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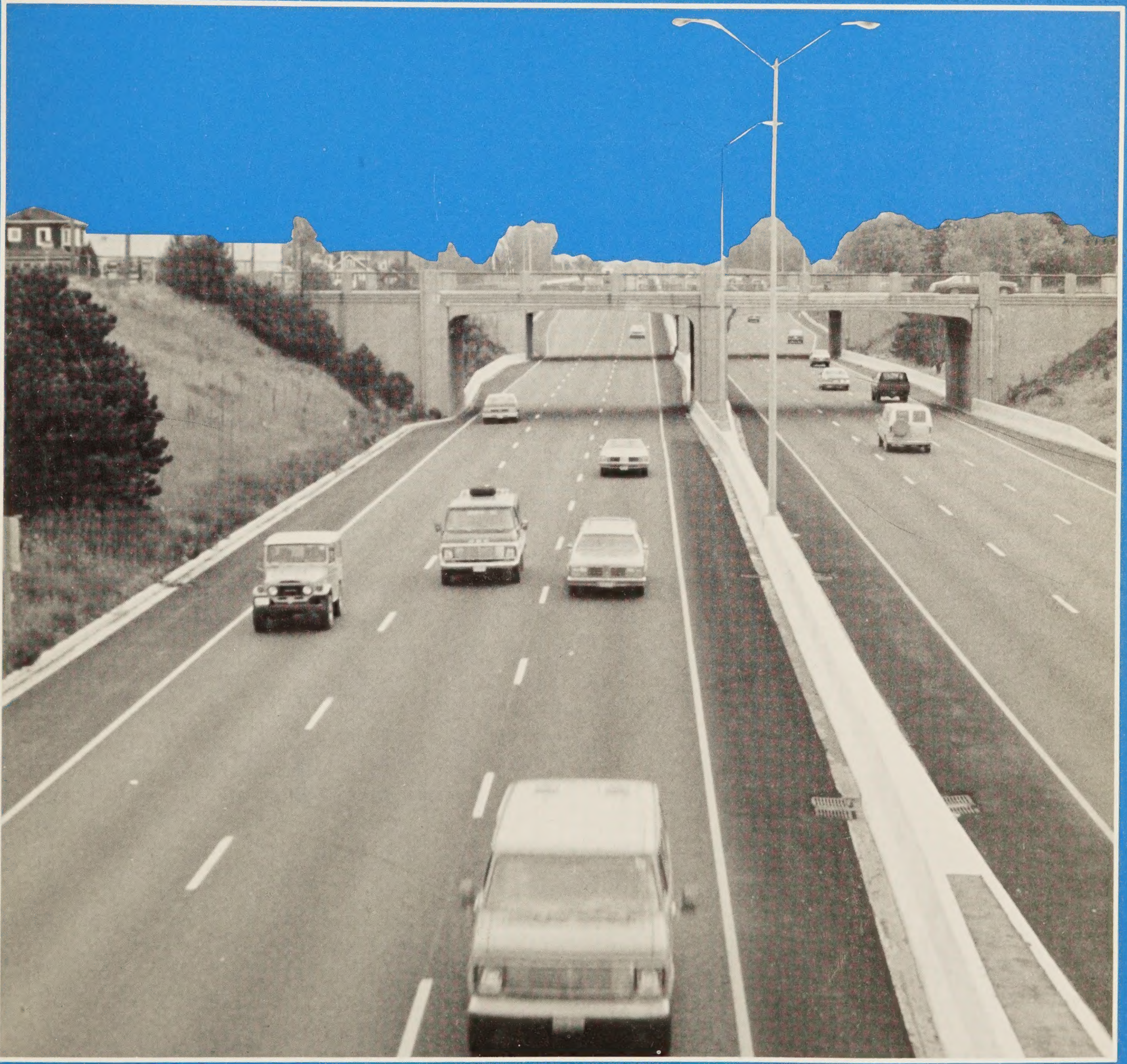

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U.S. Department of Transportation
Elizabeth Hanford Dole, *Secretary*

Federal Highway Administration
R.A. Barnhart, *Administrator*

U.S. Department of Transportation
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COVER: In an FHWA study, the effects of reduced lighting techniques on traffic operations and safety were addressed as well as the costs, potential energy savings, and legal implications of these techniques.

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David K. Phillips, *Associate Administrator*

IN THIS ISSUE

Articles

Departments

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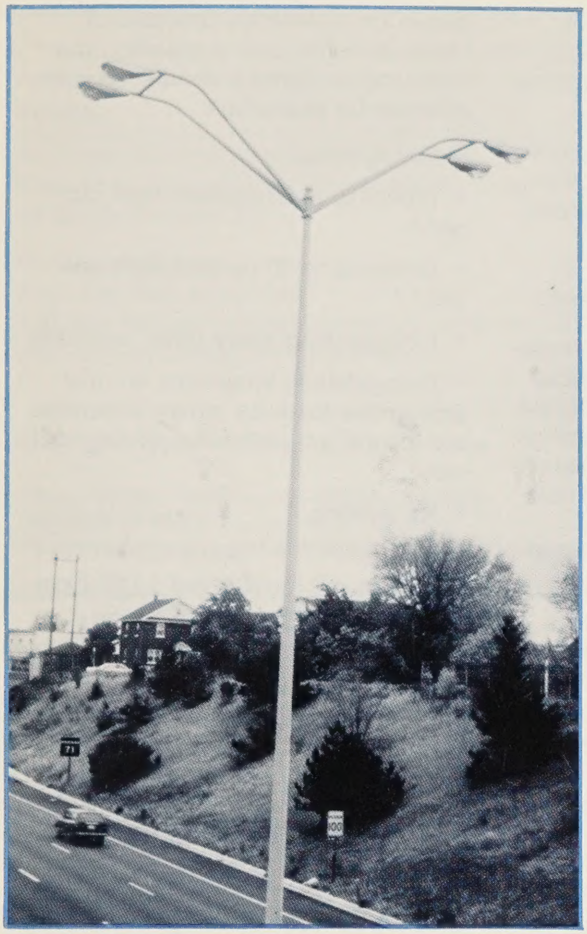

The Potential for Reduced Lighting on Roadways

by

Michael S. Janoff, Loren K. Staplin, and John B. Arens

To put the recommendations resulting from the research described in this article into proper context, the reader must be aware that although target detection may deteriorate only slightly under some of the reduced lighting conditions used in this study, the standards guiding practically all lighting designs of public roads in the United States reflect at least the minimum criteria set forth in the American National Standard Practice for Roadway Lighting (ANSI/IES RP-8, 1983). This document has been developed and is kept current by the members of the Roadway Lighting Committee of the Illuminating Engineering Society of North America and represents the national consensus of all groups having an essential interest in the provisions of this standard. The criteria, which set the minimum lighting levels considered safe for nighttime visibility and traffic conditions, are the result of ongoing work by representatives of the engineering community, various governmental bodies, academia, manufacturers, consultants, utility companies, and user groups. It is the authors' strong opinion and recommendation that, regardless of the findings demonstrated for the particular research context addressed in this study, lighting should not be reduced below ANSI minimum levels unless and until such ANSI standards may be changed.

Introduction

Over 50 percent of all motor vehicle fatalities occur in darkness even though only 25 percent of all motor vehicle travel occurs at night. (*1*)[1] This overrepresentation has been used as a justification for installing fixed lighting on many roadways. However, research to determine how such fixed lighting affects the frequency and severity of nighttime accidents appears to be mixed because accident frequencies and severities also depend on a host of geometric and traffic factors including traffic density and how it relates to the roadway's capacity, and the complexity of the driver's visual search task.[2]

[1] Italic numbers in parentheses identify references on page 42.

[2] "Road Lighting as an Accident Countermeasure," Technical Report No. 8/2, 4th draft, International Commission on Illumination, Paris, France, November 1982.

Over the past decade during periods of energy shortages, several highway agencies extinguished roadway lighting to reduce maintenance and operating costs. However, lighting frequently was restored when nighttime accidents increased or citizens pressured the agencies. (*2–4*) One fundamental problem with this reduced lighting technique was that lighting was reduced or eliminated for the entire nighttime period rather than only when traffic density was low.

Because of budget constraints, highway and lighting agencies continue to be interested in energy conservation. With the availability of modern, solid-state dimming circuitry for high intensity discharge (HID) lamps, it is possible to provide full lighting when traffic density is high and the roadway operates near capacity and to reduce lighting as the traffic density decreases. Considerable energy thus can be saved while still providing the benefits of full lighting at locations and times when driver decisionmaking is the most critical and visibility needs are the greatest.

Before such reduced lighting techniques can be implemented, however, their effects on traffic operations and safety must be quantified, their costs and energy-reduction benefits determined, and any legal implications investigated.

This article summarizes the results of a Federal Highway Administration (FHWA) study to determine if roadway lighting can be reduced or eliminated during nighttime periods when traffic density is much lower than design capacity without significantly reducing drivers' abilities to safely and effectively control their vehicles. (*5*) The study consisted of several tasks, including the following:

- A review of the literature and State and foreign experiences.
- The development of a conceptual model of visibility needs and the identification of alternative reduced lighting techniques—including no lighting, every other luminaire extinguished, luminaires on one side of a roadway extinguished, and different kinds and levels of dimming.
- The determination of the costs, potential energy savings, and legal implications of these reduced lighting techniques.
- The determination of the effect of such techniques on driver detection of simulated roadway hazards under actual traffic conditions.

The study concluded the following:

- Present technology exists for implementing all of the identified reduced lighting techniques, with benefit-cost ratios (where benefits are the savings resulting from lower energy consumption and costs are the costs of implementing the various techniques) greater than 1.0 for all but the most complex techniques.
- Driver detection performance decreased under each reduced lighting technique—minimally so for dimmed and every other extinguished and significantly so for one side extinguished and no lighting.
- Legal problems may result with the use of reduced lighting techniques.

Study recommendations include using dimming circuits, primarily for new systems, with extinguishing every other luminaire for either new or existing systems an inexpensive alternative, provided applicable lighting standards for the quality and quantity of the lighting still are met.

Selecting Reduced Lighting Techniques

A broad range of reduced lighting techniques was identified from a literature review and information obtained from worldwide lighting agencies (table 1). Various techniques were identified including extinguishing all or some of the luminaires after a designated time, installing two luminaires on each pole and extinguishing one after a designated time, and using dimming circuits to reduce the lighting by a uniform or variable amount after a designated time or as a function of traffic volume or driver visibility.

Based on preliminary analyses of costs, benefits, and availability, the following six lighting conditions were selected for evaluation:

- Full lighting.
- Dimming to 30-percent light output.[3]
- Dimming to 50-percent light output.[3 4]
- Extinguishing every other luminaire.
- Extinguishing luminaires on one side of the roadway where luminaires are located on both sides of the roadway.
- No lighting.

Effects of Reduced Lighting on Traffic Operations and Safety

Three experiments were designed and conducted to determine the effect of the six lighting conditions on traffic operations and safety. These included a pilot experiment to evaluate the extreme lighting conditions (full lighting, no lighting) on driver performance at extreme geometric (straight and level, interchange ramp) locations; a controlled field experiment to evaluate the effect of the lighting conditions at a single (mainline) geometry; and an observational validation experiment to observe the reactions of naive, unalerted motorists under three selected lighting conditions.

Pilot experiment

The experimental design, data collection, and data analysis were accomplished in the same manner as in the controlled field experiment (described in the next section). The primary result of the pilot experiment was that there was a significant decrease in driver performance when lighting was extinguished on interchange ramps but little reduction in performance when lighting was extinguished on mainline sections. Thus, the pilot experiment demonstrated that lighting should not be extinguished on interchange ramps, but the potential exists for extinguishing lighting on mainline sections. (*5*)

[3] Either uniform or variable dimming.

[4] Either dim or extinguish one of two lamps.

Table 1.—Reduced lighting techniques

Technique	Freeway/ nonfreeway	Where implemented
Every other light extinguished 1 a.m. to 4 a.m.	Freeway	Chicago, IL
Every other light extinguished after midnight or 1 a.m.	Nonfreeway	Los Angeles, CA
All lighting extinguished	Freeway	Virginia; Milwaukee, WI
One side extinguished	Freeway	Texas
Every other light extinguished	Nonfreeway	Clearwater, FL
Two-lamp luminaire—one extinguished after midnight	Freeway	Oshawa, Ontario
Time-dependent variable level (two levels)	Nonfreeway (first system) Freeway (second system)	Philadelphia, PA
Time- and volume-dependent variable level system	Freeway	Highway 401, Ontario, Canada
Time-, volume-, visibility-, and weather-dependent variable level system	Freeway	Zurich, Switzerland

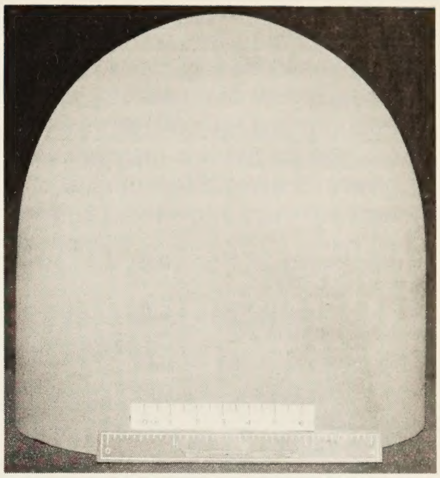

Figure 1.—Three-dimensional target.

Controlled field experiment

Data were collected in both the pilot and controlled field experiments on a 6-mi (9.7-km) portion of I-95 in the Philadelphia and Bucks County, Pennsylvania, area. This multilane, urban/suburban roadway is fully lit in Philadelphia and unlit in Bucks County. The lighting system consists of 200-W high-pressure sodium (HPS) lamps mounted 30 ft (9.1 m) at spacings of between 68 and 88 ft (20.7 and 26.8 m) in a staggered arrangement. Table 2 summarizes the photometric measurements (*6*) associated with the six different lighting conditions evaluated in this experiment.

The principal dependent measure selected for both the pilot and controlled field experiments was the distance at which drivers first could detect a simulated hazard—the target—in the roadway ahead of their vehicles.[5] It was hypothesized that as the lighting was reduced, the distance at which the driver detected the simulated hazard would decrease, and this change would serve as a surrogate measure of the relative changes in safety with the various reduced lighting techniques.

Additional dependent measures included brief subjective ratings of target visibility and responses to a questionnaire.

The target selected as a simulated hazard was a 6-in (0.15-m) styrofoam hemisphere with a 6-in (0.15-m) diameter skirt (fig. 1), used by the researchers in previous visibility and lighting studies. (*8*) This size was selected to agree with the size of the object used in the AASHTO computation of safe stopping distance. (*6*) When evaluating four of the lighting conditions, the target, painted a flat, 18-percent reflectant gray, was placed in the roadway shoulder lane at the points of maximum and minimum horizontal illuminance. When evaluating the no lighting and the one side extinguished conditions, only one target position was used because there was negligible longitudinal variability in horizontal illuminance.

The lighting on the every other extinguished, one side extinguished, no lighting, and full lighting test sections was uniform during the entire data collection period. The full and no lighting conditions did not require the lighting circuit to be changed; fuses were removed from the appropriate electrical circuits for the other two conditions.

The two dimmed conditions were obtained on the same section of roadway by installing a dimming circuit and setting the luminaire power to either 75 or 50 percent (50- or 30-percent light output, respectively).

Twenty-four subjects were tested under each of the six lighting conditions. As a control, starting points and order of presentation were distributed evenly across the six conditions. Further, for those lighting conditions that included two target positions, at random, half of the test subjects responded to the target under maximum horizontal illuminance and the other half responded to the target under minimum horizontal illuminance.

[5] This same measure has been used successfully in past lighting and visibility research to quantify the effect of visibility on a similar form of driver performance. (*7*)

Table 2.—Summary of photometric measurements

Photometric measure	Lighting condition: Full lighting	75-percent power[1]	50-percent power[2]	Every other extinguished	One side extinguished	No lighting
Horizontal illuminance—$E_{h_{ave}}$(fc)	1.15	0.58	0.35	0.58	0.04	0.01
Uniformity—($E_{h_{ave}}/E_{h_{min}}$)	4.50	4.14	4.19	14.50	—	—
Average pavement luminance—L_b(fL)	0.58	0.29	0.15	0.27	0.049	0.016
Object luminance—L_t(fL); 18-percent reflectant gray	0.23	0.17	0.12	0.18	0.10	0.03
Veiling luminance—L_v(fL)	0.39	0.19	0.07	0.25	0.14	0.01
Contrast—$(L_t - L_b)/L_b$	−0.60	−0.41	−0.20	−0.33	1.04	0.88
Visibility Index—$VI = C \times RCS_{L_b} DGF$[3]	8.78	4.43	1.66	2.93	2.82	1.57

[1] Approximately 50-percent light, relative to full lighting condition.

[2] Approximately 30-percent light, relative to full lighting condition.

[3] C = contrast; RCS = relative contrast sensitivity; DGF = disability glare factor = $\frac{L_b \times RCS_{L_{b'}}}{L_{b'} \times RCS_{L_b}}$, where $L_{b'} = \frac{L_b + L_v}{1.074}$.

Visual acuity data for each subject was obtained in a separate laboratory session using a standard Snellen chart. In addition, a contrast sensitivity screening technique was performed. Each visual performance measure was collected under two ambient light levels (target luminance)—0.4 fL (1.4 cd/m²) and 4.0 fL (13.7 cd/m²) (luminance level of plain white chart background).

Subjects were tested individually in the 1½-hour experiment which included the following:

- A simple reaction time test—a pushbutton response to a high-contrast visual stimulus (a red light emitting diode [LED] viewed against a black surround) to eliminate the effect of individual reaction time from the observed response magnitude.
- The six road trials.
- Subjective ratings of target visibility.
- A brief questionnaire on target realism and perceived safety effects of reduced lighting that was completed after the road trials were conducted.

In the road trials, the experimenter rode with the subject, and assistants were stationed out of view, slightly off the roadway shoulder. The vehicle was instrumented with a counting device linked directly to the transmission to measure the distance traveled from a designated reference point. A response button wired into the counting device was mounted in the center of the steering wheel.

During each trial, the experimenter directed the subject to pull onto the shoulder of the roadway at a reference point located at least ¼ mi (0.4 km) before the target. The experimenter then radioed the assistant to place the target in the shoulder lane. The test vehicle then re-entered the traffic stream, and the counting device was engaged. Subjects were required to drive at the 55-mi/h (89-km/h) speed limit.

When the subject first detected the target, he/she pushed the response button, which "froze" the counting device, indicating the distance traveled since the reference (radio contact) point. This display reading then was subtracted from the premeasured distance from the reference point to the target to yield the distance to the target at which the response button was pushed.

After each detection response, subjects were asked to rate target visibility on a 10-point bipolar scale.

When the six road trials were completed, each subject rated the target's realism as a roadway hazard and the perceived safety effects of implementing reduced lighting techniques.

Results of controlled field experiment

The results in table 3 show that the best (mean) detection performance was achieved under the full lighting condition, with orderly decrements in performance noted for the uniform dimming to 75-percent power, uniform dimming to 50-percent power, and every other luminaire extinguished conditions, respectively. The one side extinguished and the no lighting conditions had the poorest performance levels. The repeated measures analysis of variance of these data demonstrated highly significant main effects, even with a considerable amount of variability in subjects' performance. Further, the apparent linear pattern in the detection data was confirmed through a trend analysis.

Statistical tests demonstrated significant differences at the 0.05 level between full lighting and one side extinguished and no lighting and between full lighting and the other five conditions.

The subjective ratings of target visibility, also presented in table 3, resulted in markedly higher mean ratings for full lighting; a cluster of moderate ratings for the uniformly dimmed, every other extinguished, and no lighting conditions; and a somewhat isolated lower rating for the one side extinguished lighting condition. Also, the subjects' measured visual acuity/contrast sensitivity correlated poorly with the target detection performance.

Next, the linear regression was computed between the detection performance data and several lighting/visibility indices. The resulting coefficients are: With average horizontal illuminance, r = 0.95; with average pavement luminance, r = 0.94; and with Visibility Index values (VI)[6], r = 0.83 (figs. 2-4).

[6] See table 2.

Table 3.—Results of controlled field experiment

Lighting condition	Behavioral data: Detection distance to target — Mean	Standard deviation	Subjective data: Target visibility[1] — Mean	Standard deviation
	Feet	*Feet*		
Full lighting	287.9	190.3	7.9	2.1
75-percent power	232.6	124.8	7.1	2.1
50-percent power	223.8	97.0	7.3	2.1
Every other extinguished	204.8	92.5	6.8	2.3
One side extinguished	163.4	73.8	6.3	2.7
No lighting	163.2	44.1	7.0	2.8

[1] In rating target visibility, 1 = very poor; 10 = very good.
1 ft = 0.305 m

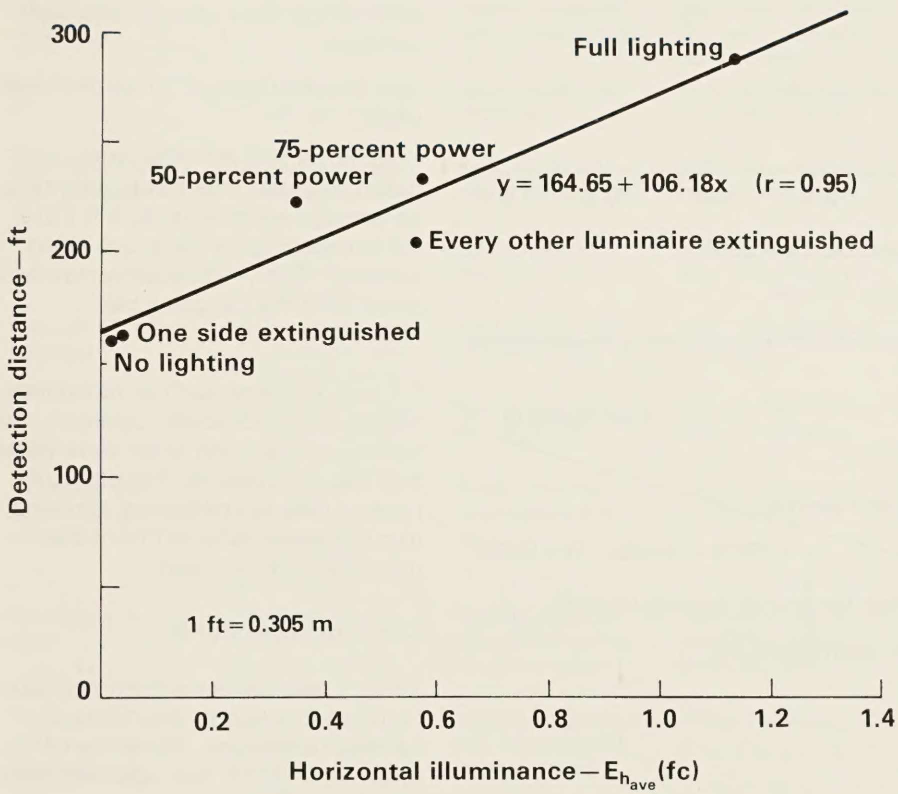

Figure 2.—Average horizontal illuminance versus detection distance.

Subjective responses to target realism and perceived safety effects of reduced lighting indicated that drivers felt that the target simulated a hazard reasonably well, felt very negative toward reducing lighting on ramps or interchanges, and felt slightly negative to ambivalent toward reducing lighting on straight, mainline roadway sections.

Observational validation experiment

An observational validation experiment using the same roadway and target as in the pilot and controlled experiments was conducted to document differences in gross driver/vehicle responses (for example, swerves/lane changes, brake actuation, and high-beam use) as a function of full lighting, uniformly dimmed lighting to 50-percent power, and no lighting. These responses were taken as indicators of a perceived hazard on the roadway. It was expected that the trend in observational data would parallel the controlled experiment data for the same lighting conditions.

Results of observational validation experiment

The results of this experiment are shown in table 4. Lateral shifts and braking reactions are included in the response totals—no high-beam use occurred.

The response percentages demonstrate a pattern corresponding to the detection measure under the same lighting conditions in the controlled field experiment. Also, the nearly identical mean vehicle velocities noted for the respective conditions indicate that the ambient lighting condition itself affected target visibility.

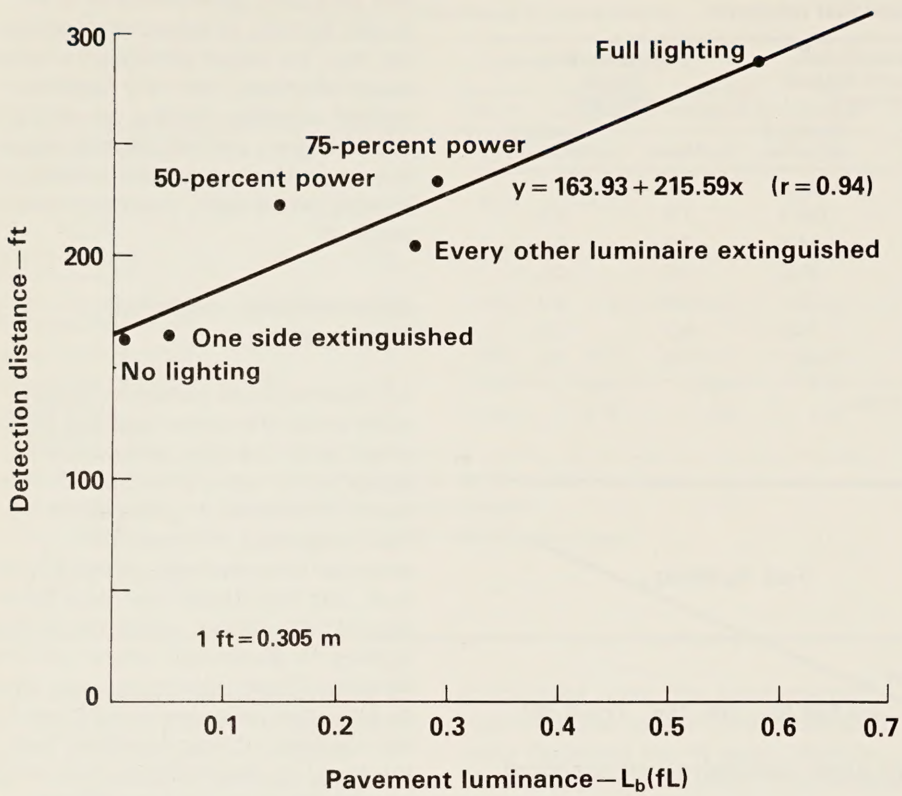

Figure 3.—Average pavement luminance versus detection distance.

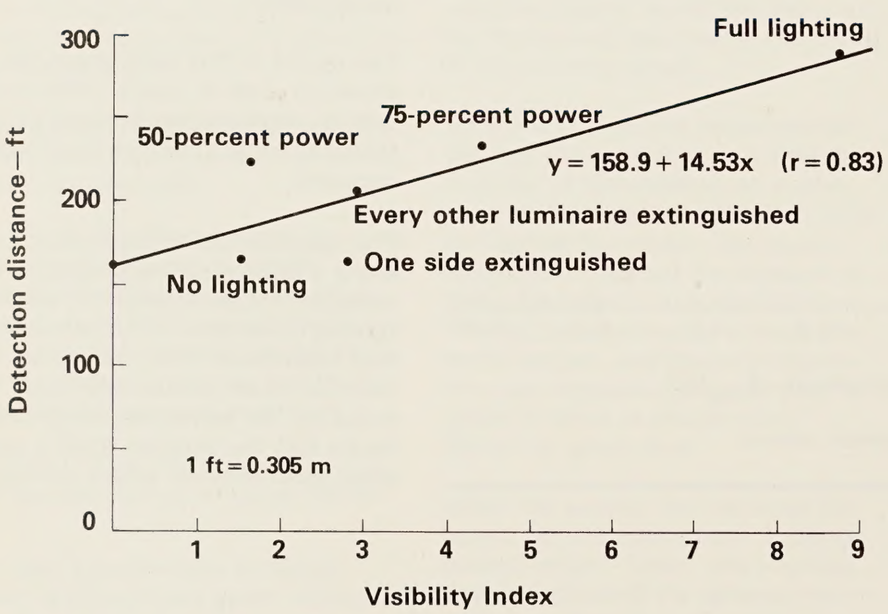

Figure 4.—Visibility Index versus detection distance.

Economic Analysis

An economic analysis of alternative reduced lighting techniques was performed to compute the costs, energy savings, and benefit-cost ratios for each technique.

The following assumptions and simplifications were made for these analyses:

- Only group-controlled lighting circuits were used for all costs.
- A cost of 6.4 cents per kWh was used.[7]
- Only one lamp (200-W HPS) was used for all systems except the two-lamp luminaire in which two 100-W HPS lamps were used to equalize wattage.
- A discount rate of 10 percent was used.
- All costs and benefits were computed on a per mile basis, assuming an average spacing of 92.5 ft (28.2 m) between lights for a two-sided system.[8] This yields approximately 57 poles per mile (35 poles per kilometer).
- Costs of items such as luminaires, timeclocks, photocells, cabinets, contactors, wiring, and labor were based on 1984 Philadelphia, Pennsylvania, costs; costs of specialized dimming circuitry were obtained from the manufacturers or users.

Costs of techniques

Table 5 lists the equipment and labor required for each of the reduced lighting techniques. For some of these techniques, two approaches are described (for example, photocell/timer or timeclock). Based on the assumptions described previously and the equipment and labor required, the costs for each of the reduced lighting techniques were developed (table 6).

[7] The figure of 6.4 cents per kWh was derived from Edison Electric Institute data and is a nationwide average for street lighting electric costs. Although this average was used in all energy cost calculations, the actual street lighting electric costs found in the United States range from about 1.5 cents to 12 cents per kWh.

[8] Derived from a brief survey of typical freeway systems in the Delaware Valley.

Table 4.—Results of observational validation experiment

Lighting condition	Sample size	Total responses[1]	Mean vehicle approach velocity
		Number (percent)	*Miles per hour*
Full lighting	72	13 (18)	58
Uniformly dimmed—50-percent power	69	9 (13)	59
No lighting	71	5 (7)	56

[1] Braking and/or lane changing.
1 mi/h = 1.6 km/h

Table 5.—Equipment and labor requirements

Technique	Requirement: Equipment	Labor	Comments
All off after midnight	(a) Photocell/timer	Replace photocell with photocell/timer	ARL watt-4 photocell; automatic reset after power failure
	(b) Timeclock, cabinet, and wiring	Install equipment (minimal)	Must be manually reset after power failure
Every other off after midnight	(a) Photocell/timer, contactor, cabinet, and wiring	Split circuit, add photocell/timer, etc.	ARL watt-4 photocell; unequal burn time
	(b) Timeclock, contactor, cabinet, and wiring	Split circuit, add timeclock, etc.	Must be manually reset after power failure
One side off after midnight	(a) Photocell/timer, contactor, cabinet, and wiring	Split circuit, add photocell/timer, etc.	ARL watt-4; unequal burn time
	(b) Timeclock, contactor, cabinet, and wiring	Split circuit, add timeclock, etc.	Must be manually reset after power failure
Two-lamp luminaire—one off after midnight	Two-headed luminaire, timer, and circuitry (Oshawa, Ontario system)	Replace luminaires plus additional wiring	Circuits to alternate luminaires
Preset light reduction	Photocell/timer, control box (Lutron), cabinet, and wiring	Replace photocell and add box, cabinet, and wiring	Timeclock also can be used as alternative to photocell/timer
Volume- and time-dependent variable light reduction	Special ballasts, dimming circuits, computer control, wiring, etc. (A. Ketvirtis, Ontario system)	Replace ballasts and add computer and volume sensing equipment	Wide Lite ballast

The costs for the all extinguished, one side extinguished, and every other luminaire extinguished techniques apply to both new and retrofitted lighting systems—all three require simple switching and timing equipment to be installed in the group-controlled circuits.

The costs for the two-lamp luminaire, uniform dimming, and variable dimming techniques apply only to new lighting systems. Costs for retrofitting probably would be 20 percent more for dimming (relatively easy to retrofit) but much higher for the two-lamp luminaire system, which would have to be modified substantially.

Potential energy savings of techniques

Based on an annual full nighttime operation of 4,200 hours (*9*) (half night = 2,100 hours) and an energy cost of 6.4 cents per kWh, the potential energy savings for each of the techniques were derived and are presented in the first column of table 7.

Benefit-cost ratios of techniques

Combining the data in table 6 with that in the first column of table 7, benefit-cost ratios were derived for each of the reduced lighting techniques, as shown in table 7. When alternative approaches were available (for example, photocell/timer or timeclock), the least expensive was used. When two or more costs were derived for a given technique (two different variable dimming methods), the average cost was used. Ranges also are noted for the averaged cases.

The most complex dimming system (variable dimming) will not prove to be cost-effective (benefit-cost ratio of at least 1.0 is cost-effective); the simpler systems will have very high benefit-cost ratios (benefit-cost ratio = 20.0).

Table 6.—Costs of reduced lighting techniques

Technique	Initial costs: Equipment	Labor	Per pole	Per mile	Annual costs per mile (10% discount)
	Dollars	*Dollars*	*Dollars*	*Dollars*	*Dollars*
All off after midnight	(a)[1] 62.00 (b) 450.00	38.00 450.00	2.00 18.00	114.00 1,026.00	11.00[2] 103.00
Every other off after midnight	(a) 300.00 (b) 450.00	300.00 450.00	12.00 36.00	684.00 1,026.00	68.00[3] 103.00
One side off after midnight	(a) 300.00 (b) 450.00	300.00 450.00	12.00 36.00	684.00 1,026.00	68.00[3] 103.00
Two-lamp luminaire—one off after midnight (new system)	—	—	—	7,650.00	765.00
Preset light reduction:					
Lutron Regulator ballast	3,000.00 per 54 lamps	1,500.00 per 54 lamps	83.00	4,731.00	473.00
Lutron Reactor ballast	3,000.00 per 30 lamps	4,000.00 per 30 lamps (disconnect capacitor from each lamp)	233.00	13,281.00	1,328.00
Volume- and time-dependent variable light reduction:					
New system in Canada	—	—	400.00	22,600.00	2,260.00
Wide Lite (retrofitted) system	500.00 per ballast plus 2,150.00 control	13,575.00	815.00	46,455.00	4,646.00

[1] (a) refers to photocell/timer and (b) refers to timeclock/contactor.

[2] For individual control, the costs are $570.00 and $296.00, respectively.

[3] For individual control, the costs are $285.00 and $182.00, respectively.

1 mi = 1.6 km

This analysis does not include the possibility of maintenance costs or energy costs increasing or the effects of reduced lighting on traffic operations and safety. In addition, three of the costs are based on new systems that have lower costs than those of retrofitted systems. Thus, the benefit-cost ratios in table 7 may be optimistic.

To compare these energy costs with the safety benefits, a sensitivity analysis was performed. On I-94 in Chicago, Illinois, the nighttime lit/unlit accident rate differential varied from 1.6 to 0.27 per million vehicle-miles (1.00 to 0.17 per million vehicle-kilometers). (*10*) The potential safety benefits for 1,000 mi (1 600 km) of lit freeway, assuming only the energy costs, were computed for an accident differential of 0.27 per million vehicle-miles (0.17 per million vehicle-kilometers).

Using the net annual energy savings per mile—$1,820 ($1,131 per kilometer) (energy savings minus cost of implementation)—resulting from extinguishing lighting after midnight, the use of lighting would have to prevent 0.46 accidents per mile (0.29 accidents per kilometer) assuming an average cost per accident of $4,000. (*11*) For 1,000 mi (1 600 km) of roadway, this equates to 455 accidents. Table 8 shows that for nighttime traffic volumes of more than 4,617 vehicles, the benefit-cost ratio is greater than 1.0. Therefore, it is cost-effective to leave the lights on when the nighttime traffic volume exceeds 4,617 vehicles or 770 vehicles per lane or 128 vehicles per lane per hour, assuming a six-lane roadway and a 6-hour lights-on-or-off consideration.

Legal Analysis

Two approaches were used to determine what legal problems might be incurred by a lighting agency using reduced lighting—a review and critique of the legal literature and a compilation and analysis of legal opinions of experts on how the laws should be interpreted.

Literature review

A review of the existing literature revealed that under both the common law and State Tort Claims Acts, the threat of liability exists for a public entity that seeks to reduce or eliminate lighting during periods of low traffic density. Any agency planning such a program should be advised that extensive scientific research and study are required. Even then, of course, a lawsuit may not be avoided, but at least the probability of a plaintiff's recovery is lessened.

Table 7.—Benefit-cost ratios of reduced lighting techniques

Technique	Annual energy savings per mile	Annual costs per mile	Average benefit-cost ratios	Range of benefit-cost ratios
	Dollars	*Dollars*		
All off after midnight	1,877.00	11.00–103.00	100	18–171
Half off (every other off or one side off) after midnight	938.00	68.00–103.00	12	9–14
Two-lamp luminaire—one off after midnight	938.00	765.00	1.2	1.2
Preset light reduction—uniform dimming (50-percent power)	938.00	473.00–1,328.00	1.0	0.7–2.0
Volume- and time-dependent variable light reduction—variable dimming (50-percent power)	938.00	2,260.00–4,646.00	0.3	0.2–0.4

1 mi = 1.6 km

Table 8.—Safety benefit-cost ratios associated with lighting 1,000 miles of roadway

Nighttime traffic volume	Expected number of accidents reduced	Benefit-cost ratios
1,000	99	0.2
2,000	197	0.4
4,000	394	0.9
4,617	455	1.0
5,000	493	1.1
10,000	986	2.2

1 mi = 1.6 km

Legal opinions

Legal opinions were obtained by contacting on the telephone and sending questionnaires to lighting engineers, consultants, and attorneys, who either worked for municipal or State agencies or were in private practice. The opinions were classified as follows:

Opinion	Percent of responses
Wait for more research	28
Definite problem	22
Defense exists	28
Liability questionable	22

Whether the respondent was a State/municipal employee or a private employee or whether the respondent was an attorney did not significantly affect the distribution of opinions.

The diversity of the responses summarized above reinforces the conclusion that tort liability will be an issue if lighting is reduced—especially below ANSI/IES recommended values.

Recommendations

Based on the results of this research, the following recommendations have been developed on the proposed use of reduced roadway lighting during periods of low traffic density:

• Uniform dimming circuits effectively reduce the energy required for roadway lighting while only resulting in a minimal, nonstatistically significant adverse effect on driver performance. Energy savings of up to 25 percent[9] can be achieved, and at an average nationwide energy cost of 6.4 cents per kWh, the benefit-cost ratio is about 1.0. For higher energy costs, the benefit-cost ratio would exceed 1.0.

The initial installation costs for such systems are, however, relatively high, and higher for existing lighting systems than for new installations.

• In urban or suburban centers where lighting levels far exceed ANSI/IES minimum requirements because of the commercial nature of the area, an inexpensive technique for conserving energy would be every other luminaire extinguished. Benefit-cost ratios for this technique range between 9 and 14 for average nationwide energy costs. Because most roadway lighting installations are designed so that extinguishing every other luminaire would result in uniformity ratios and lighting levels considerably below those suggested by AASHTO (*12*) and endorsed by FHWA, it is impossible to issue an unqualified recommendation to implement this energy conservation technique. Agencies contemplating such a step first should assure that the lighting is still in conformance with established AASHTO guidelines.

[9] Fifty percent during half of the night.

- The variable dimming circuits are not recommended because of high costs resulting in low benefit-cost ratios, and the one side extinguished technique is not recommended because of its highly significant adverse impact on driver performance. The research shows that an agency would be better off extinguishing *all* lights on a roadway rather than on one side only.

- Using two-lamp luminaires in newly designed lighting systems (where one lamp can be extinguished) can save up to 25 percent of the energy. The effect of this reduced lighting technique on driver performance is minimal (that is, the same as a dimmed circuit); however, this technique probably would be too expensive to implement in existing lighting systems.

- Reduced lighting techniques should not be implemented before 11 p.m. in most urban areas because traffic density remains relatively high until that time. Regularly scheduled sports events and other large traffic generators could change this time to still a later hour. Cities with little or no evening activity might reduce lighting earlier than 11 p.m.

To apply the preceding recommendations, an agency must be aware of the potential legal problems and ensure that lighting is still in conformance with established AASHTO guidelines and ANSI/IES minimum requirements and modify the energy side of the benefit-cost ratio computation to reflect local energy costs.

REFERENCES

(*1*) "Accident Facts," *Statistics Department, National Safety Council,* Chicago, IL, 1983.

(*2*) S.H. Young, "Turn on the Lights!" *Lighting Design and Application,* June 1982, pp. 28-32.

(*3*) S.H. Richards, "Effects of Turning Off Selected Roadway Lighting as an Energy Conservation Measure," Transportation Research Record No. 811, *Transportation Research Board,* Washington, DC, 1981.

(*4*) M.H. Hilton, "Continuous Freeway Illumination and Accidents on a Section of I-95," *Virginia Highway and Transportation Research Council,* Charlottesville, VA, August 1978.

(*5*) L.K. Staplin, M.S. Janoff, and L.E. Decina, "Reduced Lighting on Freeways During Periods of Low Traffic Density," Report No. FHWA/RD-86/018, *Federal Highway Administration,* Washington, DC, August 1985.

(*6*) Illuminating Engineering Society of North America, "American National Standard Practice for Roadway Lighting," ANSI/IES RP-8, 1983.

(*7*) V.P. Gallagher and P.G. Meguire, "Contrast Requirements of Urban Drivers," Driver Visual Needs in Night Driving, TRB Special Report No. 156, *Transportation Research Board,* Washington, DC, 1975.

(*8*) M.S. Janoff et al., "Effectiveness of Highway Arterial Lighting," Report No. FHWA-RD-77-37, *Federal Highway Administration,* Washington, DC, July 1977.

(*9*) P.C. Box, "Relationship Between Illumination and Freeway Accidents," IERL Report, Project 85-67, *Illuminating Engineering,* May/June 1971.

(*10*) P.C. Box, "Freeway Accidents and Illumination," Highway Research Record 416, *Highway Research Board,* Washington, DC, 1972.

(*11*) B.C. Kragh, T.R. Miller, and K.A. Reinert, "Accident Costs for Highway Safety Decisionmaking," *Public Roads,* vol. 50, No. 1, June 1986, pp. 15-20.

(*12*) "An Informational Guide for Roadway Lighting," *The American Association of State Highway and Transportation Officials,* 1984.

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Corrosion in Reinforced Soil Structures

by
Salem D. Ramaswamy and Albert F. DiMillio

Introduction

Soil reinforcement—the inclusion of resistant elements in an engineered soil mass to form a composite material that brings out the best features of both the soil (compressive and shear strength) and the reinforcement (tensile strength)—has emerged as one of the most significant advances in geotechnical engineering (fig. 1). Reinforced soil structures are particularly well suited for retaining walls, bridge abutments, embankments, and cut slopes (fig. 2). (*1*)[1]

A major design aspect of a reinforced soil structure requires tensile loads between the reinforcing elements and the surrounding soil to be transferred by friction and passive soil resistance. The relative contributions of friction and passive resistance depend on soil type and reinforcing elements. For example, strips and sheet reinforcing elements provide mostly frictional resistance, with some passive soil resistance provided by ribs or ridges on the surface of the reinforcement. Grid systems provide frictional resistance from the longitudinal elements and passive resistance from the interconnected transverse elements (fig. 3).

Other major design considerations are the tensile strength and durability of the reinforcing elements. Unanticipated loss of cross-sectional area because of corrosion can result in abrupt and tragic structural failure. Current design methods for reinforced soil structures rely on the calculation of material thicknesses to resist the applied loads plus an additional sacrificial thickness to counteract corrosion losses over the anticipated design life of 75 to 100 years.

This article presents an overview of corrosion problems in reinforced soil structures and describes current research by the Federal Highway Administration (FHWA) to provide solutions.

[1] Italic numbers in parentheses identify references on page 48.

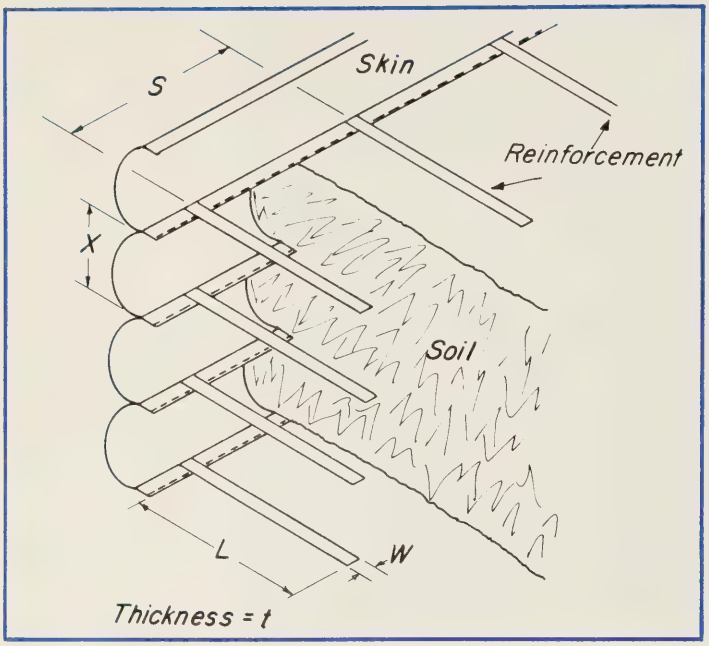

Figure 1.—Reinforced soil retaining structure.

Figure 2.—Reinforced slope.

Background

Although the use of tensile inclusions in soil for reinforcement dates back several thousand years, the modern concept of soil reinforcement was proposed by Henri Vidal in 1966 in France. (*2*) The concept was first introduced in the United States in 1969 under an FHWA demonstration project on a national forest highway in California. Since then, more than 1,000 reinforced soil structures have been built throughout the United States, and more than 5,000 have been built worldwide. (*3*)

Although corrosion of the reinforcing elements always has been a serious concern, many new structures are built each year using only the sacrificial thickness design concept without additional safeguards against corrosion. Only six corrosion failures have been reported thus far; however, it is still early in the design life of most of these structures. All of the corrosion failures occurred under severe environmental conditions such as heavy salt concentrations, organic backfill soils, or other corrosive chemical attack. For example, corrosion in a wall in Spain was accelerated when a truck accident caused corrosive chemicals to penetrate the wall. A wall in Nice, France, failed when the sulfate-inducing bacteria in the organic soil used as backfill corroded the steel reinforcements. In Cap d' Agde, France, a wall failed because the aluminum alloy reinforcements contained too much copper, which crystallized during the corrosion cycle. Also, a wall in Brunswick, Georgia, that was submerged under seawater twice daily because of ocean tides failed because of localized corrosion. The backfill material exhibited low resistivity, high chloride concentrations, and the presence of heavy metals such as iron. Figure 4 shows a cross section of a metallic strip from the Brunswick wall that was reduced from 0.08 in (2 mm) to 0.05 in (1.3 mm) because of the localized corrosion. (*4*)

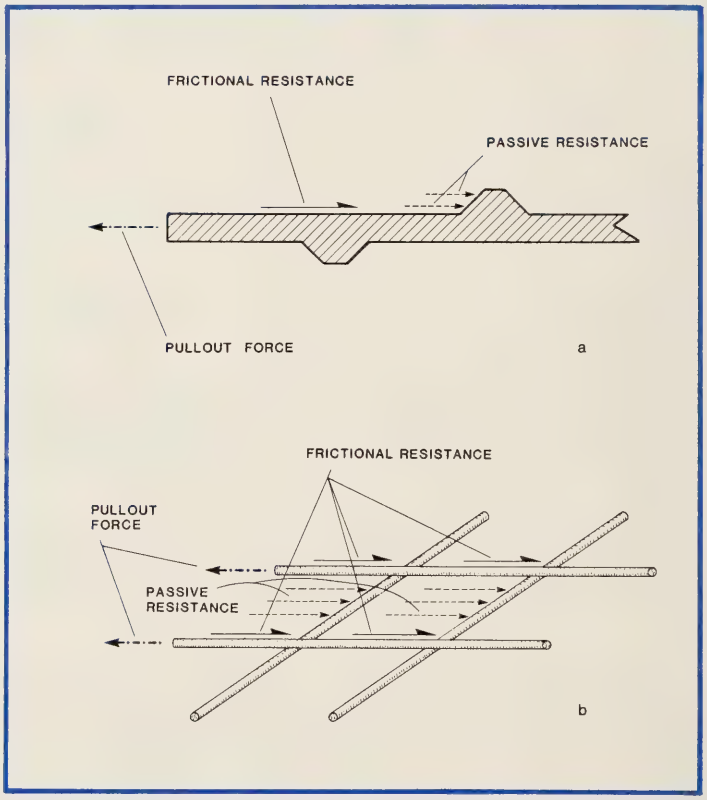

Figure 3.—Reinforcements that develop resistance by a combination of frictional and passive soil resistance. (a) Ribbed strips used in Reinforced Earth. (b) Bar mesh system.

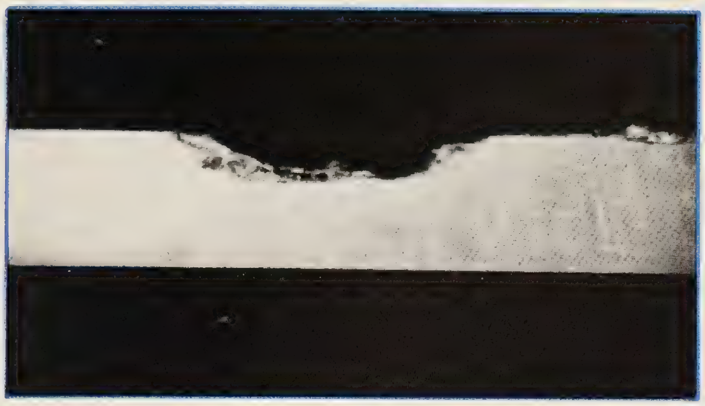

Figure 4.—Cross section of a metallic strip showing localized corrosion.

Figure 5.—Placing the major components of a reinforced soil structure.

Components of Reinforced Soil Structures

The four major components of reinforced soil structures can be seen in figure 5:

- Reinforcing elements.
- Backfill soil.
- Facing units.
- Connections between the facing units and the reinforcing elements.

Reinforcing elements

Until recently, reinforcing elements always were made of metal—mild steel with extra thick sections to allow for corrosion, stainless steel and aluminum magnesium alloys, and galvanized steel, the most commonly used reinforcing element. However, research on the performance of galvanized steel is inconclusive.

The recent emphasis on corrosion resistance has prompted the development and marketing of nonmetallic reinforcing elements made of materials such as fiberglass-reinforced plastics and geotextiles (fabrics) (fig. 6). These nonmetallic reinforcing materials do not corrode, but with time they may degrade, fracture, or be weakened by biological or environmental elements. They also are susceptible to damage from shipping, storage, and placement.

Resin-bonded epoxy coatings have been used on a few projects with some success; however, a number of problems with this method need to be resolved before it can be considered for routine application. Current research is addressing the appropriate coating thickness to minimize transportation and installation damage while maintaining sufficient flexibility. Recent research indicates that current standards for coating thicknesses (ASTM—7 to 15 mils [0.2 to 0.4 mm] and AASHTO—5 to 12 mils [0.1 to 0.3 mm]) should be increased to at least 18 mils (0.5 mm). Adhesion qualities and frictional losses associated with the use of epoxy coating also must be considered.

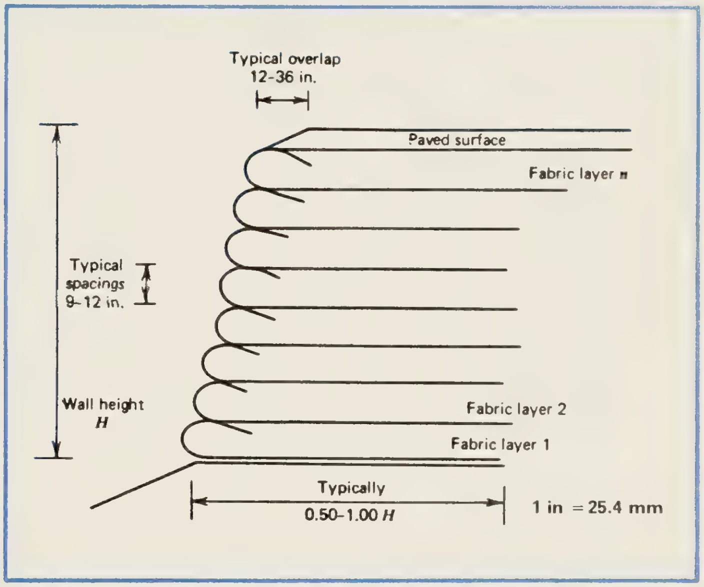

Figure 6.—Schematic of a fabric-reinforced soil wall.

Backfill soil

Backfill material usually is free-draining granular soil to provide sufficient frictional properties to maximize load transfer between the soil and reinforcement. Typical backfill specifications for reinforced soil limit the gradation, plasticity index, compaction moisture content, and organic content. FHWA specifications limit backfill to 100 percent passing the 6-in (152-mm) sieve, 75 percent passing the 3-in (76-mm) sieve, 25 percent passing the No. 200 sieve, and a plasticity index of less than 6 percent. (*5*) A prominent soil reinforcement company in France requires a minimum friction angle of 25 degrees, resistivity of more than 33 $\omega \cdot$ft (1,000 ohm-cm) for unsaturated soils and 98 $\omega \cdot$ft (3,000 ohm-cm) for saturated soils, a pH value of 5 to 10, chloride content of less than 200 ppm, sulfate ion content of less than 1,000 ppm, and no organic content. These French limits were established from research tests that showed soil corrosivity to be controlled by the degree of saturation, porosity, acidity or alkalinity, dissolved salt content, and electrical resistivity. For example, a clay backfill usually is more corrosive to buried steel than is a clean granular soil, especially if the compaction moisture content is greater than 20 percent. Organic matter and other deleterious substances are highly corrosive to buried steel.

In addition to using free-draining soil, internal drainage systems are required to prevent localized channeling of water through the reinforced soil mass, which could cause extensive localized corrosion. Plastic sheeting and/or a clay cap also should be used over the granular mass to reduce water infiltration.

Facing units

Facing units usually are made of precast reinforced concrete panels serving mainly as architectural elements but also as a barrier to prevent the granular backfill from flowing away from the reinforcements. The reinforcement in the precast concrete panels provides strength during transportation and erection and is much less vulnerable to corrosion than are the metal reinforcements buried in the soil. Because only minor forces are exerted on the facing units, their design is based on flexibility and corrosion resistance. Metal facing panels and flexible facings such as welded wire mesh, geogrids, and geotextiles also have been used; shotcrete or gunite protective systems usually are required to stabilize flexible facing units.

Connections between the facing units and the reinforcing elements

These connections usually are made of metals prone to corrosion. A galvanized strip of steel protrudes from the concrete facing panel and is bolted with galvanized bolts to the galvanized reinforcements buried in the soil mass. The corrosive environment at the connection can be severe because the space behind the facing panels allows for large concentrations of water and oxygen. Because the metal protruding from the concrete panel forms a very active and classic galvanic cell when it is connected to the metal reinforcement buried in the soil mass, similar metals should be used to connect facing units and reinforcing elements.

Corrosion Mechanism in Metals

Corrosion results from the chemical interaction between a metal and its surrounding environment. Most soil reinforcement corrosion is essentially an electrochemical process because it usually occurs in the presence of an electrolyte, such as water, containing dissolved salts and high concentrations of oxygen. A galvanic cell is formed between two poles—an anode and a cathode—that are connected by the electrolyte to form a complete circuit (fig. 7). The conductivity of the electrolyte directly controls the quantitative loss of metal ions from the anode as the current flows from the anode through the electrolyte to the cathode and back through the metal in a complete circuit.

The conductivity usually is determined indirectly by measuring the soil's resistivity because resistivity is the direct inverse of conductivity. The cathode area does not corrode because of the deposition of hydrogen or other ions that carry the current.

Attempts have been made to eliminate or counteract the galvanic cell formation using galvanizing and cathodic protection systems. The steel galvanizing process (ASTM A123) forms a zinc coating on the metal, which becomes a sacrificial anode of the galvanic cell, leaving the base metal intact. Zinc also encourages a more uniform corrosion, reducing the formation of pits and other localized corrosion.

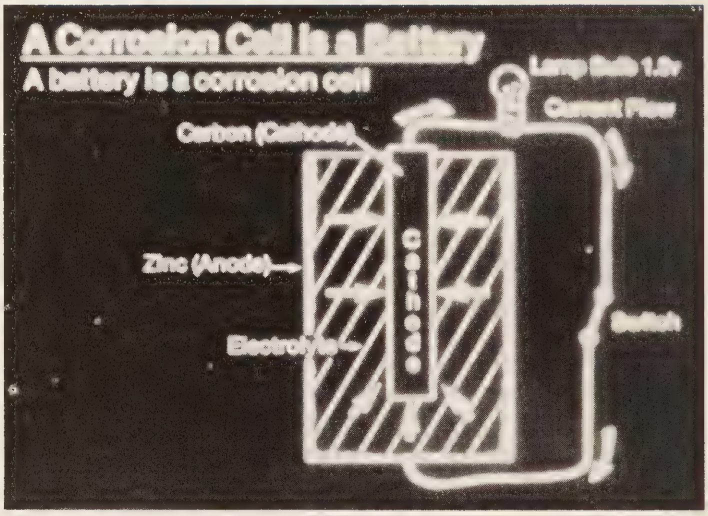

Figure 7.—Most soil reinforcement corrosion is essentially an electrochemical process.

Cathodic protection systems, on the other hand, use sacrificial aluminum anodes to divert the corrosion attack away from the steel reinforcements. Two cathodic protection systems were installed on separate structures in South Africa, and both systems failed in less than 4 years. The reasons for these failures are unknown. The systems may have been applied incorrectly, thus accelerating the corrosion, or the systems may not have been capable of reducing corrosion.

Current efforts are focused on developing a cathodic protection system for reinforced soil structures in which the cathodic protection system is connected with the entire reinforced soil mass and the connections between the reinforcing elements and the facing units. The system requires periodic maintenance and service to replace the aluminum anodes that become corroded. This system has been used successfully for protecting steel structures above ground, especially in the oil industry; however, it is not yet popular in the reinforced soil area because of the extra initial cost and recurring maintenance costs.

Another method that has not yet been applied involves encapsulating a coated or galvanized reinforcing element into a durable synthetic sheathing to produce a double protection system similar to that used to protect permanent ground anchors. (*6*) The encapsulated synthetic sheathings could be manufactured with studs, grooves, or corrugations on the exterior surface to satisfy the frictional strength requirement. The connections at the facing units could be encapsulated in the same way as the strips, straps, wires, or grids buried in the soil mass.

The major drawback to this double corrosion protection system is cost. With permanent ground anchors, the relative isolation or low density of the anchors keeps the cost down, but the high density of reinforcing elements in a reinforced soil mass has discouraged the use of double protection systems. Also, as mentioned previously, the durability of the synthetic material used for the sheathing is questionable because it may fracture or weaken with time and is susceptible to damage from shipping and placement.

Another form of corrosion, stray-current corrosion, can occur from electrical leaks into the reinforced soil mass. The stray currents enter the metal reinforcements and are discharged back into the soil at a distant point. The corrosion occurs where the current leaves the metal and enters the soil or water electrolyte.

The corrosion mechanism is a complicated, unpredictable interrelationship of many factors, some of which change with time. Because it is unlikely that a "magical alloy" that can resist aggressive corrosive attack will be developed, progress must continue to be made in predicting the corrosive environment and in developing protective coatings with a predictable service life. The rate, extent, and consequences of corrosion of reinforced soil elements also must be understood to determine the service life of a reinforced soil structure.

FHWA Corrosion Study

FHWA has initiated a research study to evaluate the potential corrosion problem in reinforced soil structures and to seek appropriate solutions. Any investigation may conclude that failure of a reinforced soil structure was caused by corrosion, because an examination of the reinforcement embedded in the soil almost always will show some signs of corrosion. (*7*) However, corrosion may not be solely responsible for the failure.

Pit-type penetrations of a buried strip or even complete severence of the strip does not necessarily lead to failure of a reinforced soil structure as it would for a buried tank or pressure vessel. Depending on the number and location of the pitting-type penetrations, the serviceability or the effective cross section resisting capacity may not have been dangerously reduced.

In the FHWA study, the corrosion and deterioration mechanisms and rates in reinforced soil structures are being evaluated through laboratory and field experiments. Specifications will be developed, and new test methods for analyzing and selecting backfill and reinforcement materials will be evaluated.

Facing panel materials, backfill soils, and reinforcing elements will be examined to determine which factors affect the design life under corrosive and deteriorating forces commonly encountered in reinforced soil environments. Chemical composition of backfill soil, kind of backfill (granular or clay), protective coating types, and stray currents will be evaluated. Exposure to seawater, roadway deicing salts, and galvanic current also will be studied, as will the potential for using cathodic protection systems and other methods currently used on highway bridge decks, underground pipelines, and piles to minimize the harmful effects of corrosion.

The long-term corrosion rate and/or durability of four field structures, both new and existing, will be monitored. Current methods for evaluating backfill materials will be studied and, if appropriate, new test methods for the analysis of pH, resistivities, sulfates, and chlorides will be developed. The effect of using lower quality backfill materials on the design life of a reinforced soil structure also will be evaluated.

Laboratory and field experiments will be conducted to develop appropriate requirements for selecting, storing, handling, and installing epoxy-coated reinforcing elements and geosynthetics (geotextiles and geogrids) for use in reinforced soil structures. The fusion-bonded epoxies will be studied to evaluate required coating thickness, bond test criteria, impact resistance, and coating flexibility. Geosynthetics will be studied to evaluate long-term creep under sustained tensile load, resistance to chemicals and solvents under stressed conditions (environmental stress cracking), construction effects, aging, ultraviolet light degradation, and probability of nonductile failure.

Based on the laboratory and field results, a detailed manual of practice containing specifications, test methods, and engineering guidelines will be developed.

Summary

Soil reinforcement has become an effective engineering solution for many highway applications. Corrosion of metal reinforcements and durability of nonmetallic reinforcements are primary concerns in selecting and designing reinforced soil structures. The exact mechanisms of corrosion and loss of durability of soil reinforcing elements are not well understood. Cathodic protection systems, galvanized metal elements, epoxy coatings, and nonmetallic materials have been used to reduce corrosion of steel reinforcement. Double protection systems, such as encapsulating coated strips with durable synthetic sheathings, also have been proposed.

Testing and evaluation are underway to develop a clear understanding of the corrosion/durability problem and to develop design and construction guidelines for reinforced soil structures.

REFERENCES

(*1*) J.K. Mitchell, W.C.B. Villet, and A.F. DiMillio, "Soil Reinforcement for Stabilization of Earth Slopes and Embankments," *Public Roads,* vol. 48, No. 3, December 1984, pp. 88-95.

(*2*) H. Vidal, "The Principle of Reinforced Earth," Highway Research Record No. 282, *Highway Research Board,* Washington, DC, 1969, pp. 1-16.

(*3*) J.K. Mitchell and J.C. Collin, "Earth Walls," Proceedings of the National Academy of Engineering, 1983 Symposium on Cutting Edge Technologies, *National Academy Press,* Washington, DC, 1984.

(*4*) S. Frondistou-Yannas, "Corrosion Susceptibility of Internally Reinforced Soil Retaining Structures," Report No. FHWA/RD-83/105, *Federal Highway Administration,* Washington, DC, January 1985.

(*5*) "Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects," FP-85, *Federal Highway Administration, U.S. Department of Transportation,* Washington, DC, 1985.

(*6*) S.D. Ramaswamy, K.Y. Yong, and S.K. Roy, "Corrosion Protection of Ground Anchors," Proceedings of 3d Asian Pacific Corrosion Control Conference, Taipei, Taiwan, June 1983.

(*7*) S.D. Ramaswamy, K.Y. Yong, and S.K. Roy, "Corrosion Aspects of Reinforced Earth," Proceedings of 3d Asian Pacific Corrosion Control Conference, Taipei, Taiwan, June 1983.

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Toward Improved Highway Safety Analysis

by
Charles P. Brinkman

Introduction

The fatality rate per 100 million vehicle-miles of travel has decreased from over 18 to under 3 (11.2 to 1.9 fatalities per 100 million vehicle-kilometers of travel) over the past 60 years (fig. 1) (*1*)[1]—an average decrease of over 3 percent per year. Although traffic safety has improved noticeably over the years, efforts to quantify the extent that various factors have contributed to this improvement have not been definitive. The dramatic decline in fatality rates results partly from advances in medical treatment and improvements to vehicles and highways; seatbelts save lives, and newer highways are safer than older ones. Part of this improvement, however, may result from factors associated with the changes in the kind of vehicle-miles of travel. For instance, the percentage of urban travel, which is associated with a fatal accident rate only one-half as great as the rural fatal accident rate, has increased.

Changes in driver demographics as the "baby boom" generation ages also may have contributed to a lower fatal accident rate as perhaps drivers are more safety-conscious today. Finally, economic factors have a short-term influence on traffic accident rates by influencing the amount of various kinds of travel. For example, teenage unemployment might reduce the amount of driving by teenagers and thus reduce traffic fatalities.

In the past, highway safety hazards were relatively easy to identify, and many of the most hazardous situations now have been improved. Despite this improvement, 44,372 persons died and more than 3,200,000 persons were injured in motor vehicle accidents in 1984. The estimated cost including property damage was over $100 billion. (*2, 3*) The economic loss amounted to approximately 2 percent of the gross national product. Today, with tight budgets and a vast highway system, available safety funds must be spent even more wisely. To ensure this, more effective means for identifying hazardous situations and more accurate estimates of the effectiveness of candidate safety improvements are needed. This article outlines some of the major obstacles in current practice to improved highway safety analysis and suggests methods for improvement.

[1] Italic numbers in parentheses identify references on page 54.

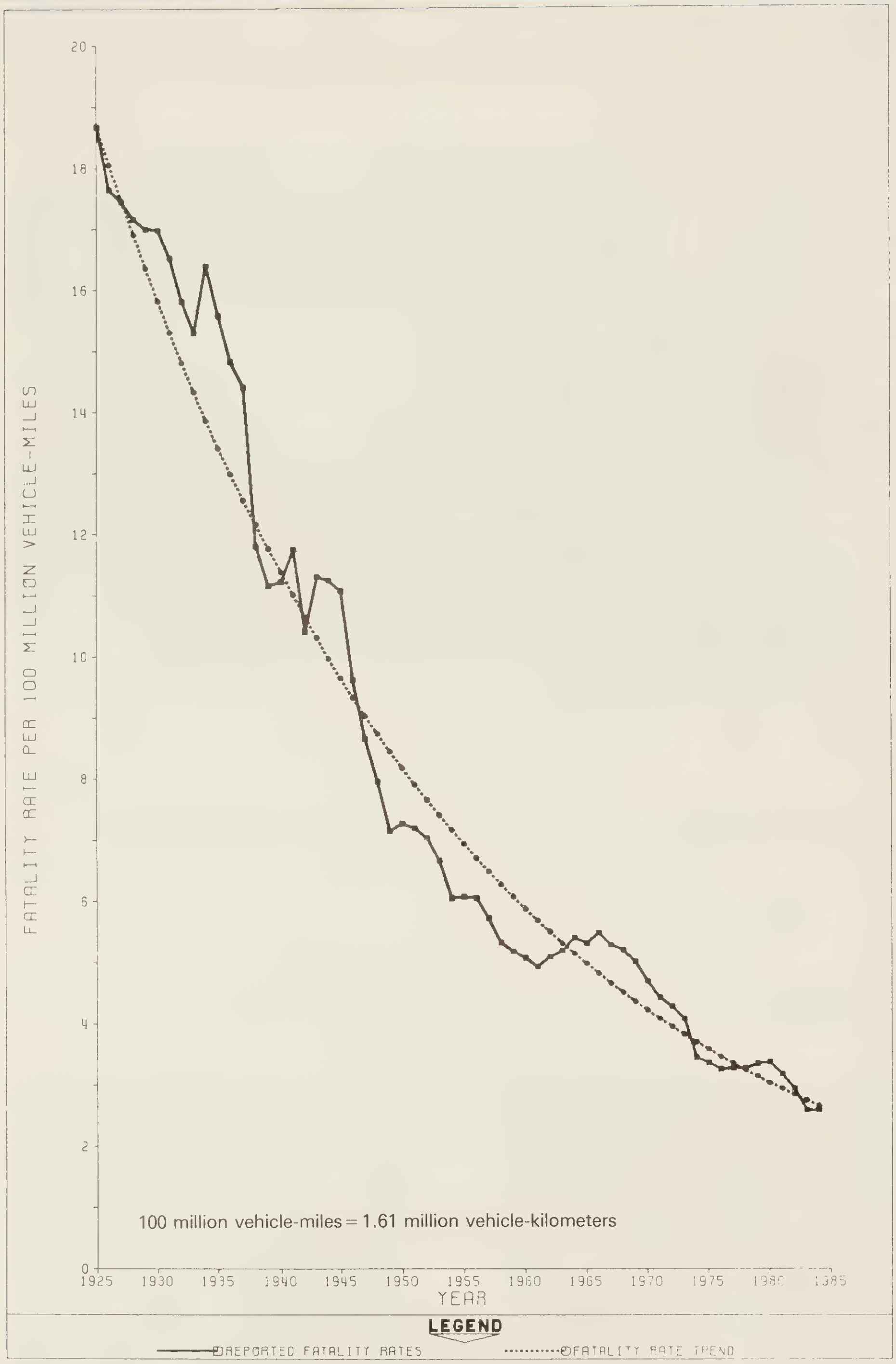

Figure 1.—U.S. motor vehicle traffic fatality rates (1925–1984). (1)

Information Needs

The ready availability of accurate and complete data is of paramount importance to improved highway safety analysis. At the national level, traffic volumes, the physical condition of the highway system, and trends in highway safety are important for establishing policy priorities and determining future needs. At the State and local levels, similar policy decisions are made. In addition, efficient day-to-day operation of the highway system demands detailed information about specific highway sites for maintenance, traffic management, and safety.

File linkage

Safety analysis uses data from a variety of sources, usually on separate files—accident and driver records, traffic volumes, highway characteristics, and traffic control devices. File linkage of these data allows great opportunity for improved highway safety analysis. Coordinated data collection avoids duplication. Carefully considering compatibility and quality reduces data collection and management costs and provides better support to the decisionmaking process.

Although file linkage unquestionably is desirable, it entails considerable obstacles. Ideally, all files should be compatible; in practice, little effort is made among the different offices of an organization responsible for data collection to ensure data compatibility. Most importantly, different location reference systems often are used, making linkage extraordinarily tedious.

Despite these obstacles, several States already have made strides in file linkage; several other States currently are merging data files with the encouragement of and, in several instances, partial financial support from the Federal Highway Administration (FHWA) and the National Highway Traffic Safety Administration.

Problems with safety information

State recordkeeping systems have severe shortcomings. Despite the wealth of data available in various files, information about the roadside and highway horizontal and vertical alignment, for example, usually does not exist in computerized form and is difficult to obtain. Highway inventory information is very limited for off-State systems, making computerized linkage of accidents to specific highway characteristics impossible.

Another problem is the quality and completeness of data. For example, not all reportable accidents get reported, particularly in urban areas where property-damage-only accidents often are not reported. In general, the more severe an accident, the more likely that accident will be reported. Also, a multi-vehicle accident is more likely to be reported than a single-vehicle accident of equal severity. Accidents are probably less likely to be reported during bad weather than during good weather.

Disregard for discrepancies in reporting particular accident types can lead to erroneous conclusions, and any safety analysis must consider the implications of these discrepancies. For example, a roadside appurtenance might reduce the severity of a majority of impacts to the extent that only minor damages and minor or no injuries result, and thus these accidents often would not be reported. The few severe accidents would likely be reported. If effectiveness were measured only by the percentage of reported accidents resulting in fatalities or severe injuries, it might be concluded that the appurtenance is ineffective. However, overall the appurtenance is effective.

The availability of complete, quality data, however, does not ensure good analysis. The nature of the highway accident problem makes data analysis and interpretation difficult. Statistical analysis is severely handicapped without sound experimental design, which usually is not considered when highway safety projects are selected for implementation. Another problem is the interrelationship of accident causal factors, confounding the establishment of strong statistical relationships. Therefore, although accident studies quantify the extent of a particular problem and provide valuable insight, they can only provide general guidance to specific safety design decisions because of their many limitations.

Safety design decisions also must be based on engineering standards and practice, physical analysis of the problem, traffic operational measures of effectiveness, and human factors research. For example, full-scale crash testing and computer simulation play an important role in examining roadway hardware performance.

Who Should Evaluate Countermeasure Effectiveness?

A thorough knowledge of experimental design, data quality, inferential statistics, and data interpretation are among the factors essential to conducting highway safety evaluations. A review of the current literature reveals that accident studies with serious design flaws and inadequate or inappropriate statistical analysis are commonplace. The studies often are inconclusive or provide conflicting conclusions, leaving users with little guidance concerning countermeasure effectiveness.

Therefore, trained professionals should determine countermeasure effectiveness instead of operations personnel who, for the most part, have neither the time nor training to conduct effectiveness evaluations. However, the practitioner is responsible for monitoring a countermeasure to ensure that it meets its objectives.

Experimental Designs

FHWA sponsored the development of an accident research manual (*4*) and presented several workshops aimed at improving accident research. The manual and workshops warned of the following dangers inherent in simple before/after evaluations:

- They do not rule out rival explanations.
- If only two time periods are examined, longer term trends go unnoticed.
- They may lead to overestimates of countermeasure effectiveness as a result of "regression to the mean." Simply stated, sites with the highest number of accidents in the past are most likely to have fewer accidents in the future even if no countermeasures are installed.

These problems can be minimized through the use of more effective experimental design or imaginative analysis. Better experimental designs include the following (*4*):

- Before/after designs with randomized control groups—This design is the strongest before/after design. Candidate locations for a given treatment are randomly assigned to either a treatment group or a control group. Controls can be imposed without using additional locations. For example, accidents during a certain period of the day, week, or year may be compared with those occurring during another period. Similarly, a specific accident type may be compared with another accident type.
- Before/after designs with comparison groups—Control locations are sought that are as similar as possible to the locations treated to diminish some of the problems inherent in simple before/after designs. An adjacent highway section often is a good control.
- Time series designs—Numerous observations of the variable of interest are made over a period of time. However, there must be sufficient data for the variable to be stable for each time period.

Simple before/after studies without controls should be discouraged. If a sound experimental design is not possible, the following should be considered: Monitoring from an operational perspective only; using measures of effectiveness (for example, traffic conflicts, speed differentials, or lateral placement) instead of accidents, to supplement accidents, or to help serve as a control; and/or using less traditional statistical methods such as Bayesian statistics. Statistical techniques, although useful, are no substitute for good experimental design.

Empirical Bayesian Statistics

Recent research sponsored by FHWA in conjunction with the National Highway Traffic Safety Administration and Transport Canada proposed using empirical Bayesian statistics for highway safety analysis. (*5*) This method combines current experience with prior experience and thus partially overcomes the lack of sufficient statistical significance in individual accident studies to conclude that a safety measure is effective. Empirical Bayesian statistics also avoid inflated estimates of safety measure effectiveness resulting from the regression-to-the-mean phenomenon.

In the FHWA study, empirical Bayesian statistics were used to estimate the number of accidents for a site (i) by adjusting the observed accident record for that site with the accident experience of similar sites as follows:

$$m = x_i + (\bar{x}/s^2)\,(\bar{x} - x_i)$$

Where,

x_i = accident experience at each site i.
$\bar{x}$ = the mean from a group of sites similar to site i.
s^2 = the variance from a group of sites similar to site i.

The Bayesian estimates for the before-accident experience at the individual sites were summed and compared with the observed after-accident experience to determine safety measure effectiveness. The assumption is that the empirical Bayesian estimate of accident experience is a better estimate of the true safety of the sites than is the observed (sample) accident experience. However, it is not yet agreed upon as to under what circumstances Bayesian statistics should be proposed as an alternative to the use of classical statistics for evaluating countermeasure effectiveness.

Table 1 illustrates the use of this method for two-way to four-way stop conversion in San Francisco, California.

Incremental Effectiveness

Another difficulty in determining countermeasure effectiveness is the tendency to install several countermeasures at the same site at the same time. Because countermeasures often may be effective against the same kinds of accidents, their effect is not necessarily additive. Some countermeasures while ineffective alone may enhance the effectiveness of another countermeasure. Very few, if any, countermeasures eliminate or reduce all kinds of accidents. When sufficient accident data are available, the selection of countermeasures should be based on an analysis of that data categorized by kind of collision, accident severity, time of day, and environmental condition.

Table 1.—Safety measure effectiveness evaluation using Bayesian statistics

Accident type	Actual number of before accidents	Actual number of after accidents	Apparent percent effectiveness[1]	Bayesian estimate of number of accidents without countermeasure	Unbiased percent effectiveness[2]
Right-angle	129	16	88	93	83
Rear end	10	16	−60	4	−300
Left turn	14	7	50	10	30
Pedestrian	6	2	67	6	67
Injury	48	9	81	35	74
Total number of accidents[3]	172	50	71	130	62

[1] The actual number of after accidents subtracted from the actual number of before accidents divided by the actual number of before accidents.

[2] The actual number of after accidents subtracted from the Bayesian estimate of accidents divided by the Bayesian estimate of accidents.

[3] Not all accident types are shown.

A recent FHWA study concluded that it is desirable to determine the incremental effectiveness of a countermeasure used with another countermeasure. (*6*) Such determinations are particularly necessary to avoid implementing combinations of countermeasures that result in less than optimal safety improvements. The need for gradually introducing improvements at individual sites or the large number of similar sites required make such determinations difficult. Improved highway information systems eventually might overcome some of these problems; the unavailability of computerized data on countermeasure implementation (for example, date, location, and kind of countermeasure) currently prevents tapping large historical accident data bases for determining countermeasure effectiveness. Sample size requirements, however, always will place a limit on detecting small incremental differences in effectiveness.

Interpretation of Analysis

Computers have made it easy to establish mathematical relationships between traffic accidents and various factors; however, it is possible to arrive at chance relationships. Also, a strong mathematical relationship that exists between various factors does not necessarily indicate a causal relationship. (*7*) In analyzing highway accident data, a multitude of possible causative factors often are interrelated and sometimes difficult to quantify. For example, one study found that on arterial street segments, the mean accident rate is significantly lower on segments with fewer than three regulatory sign faces. The authors did not conclude, however, that regulatory signs contribute to accidents. (*8*)

Applicability of Evaluation Results

States often are reluctant to accept accident countermeasure effectiveness evaluations found in other States. If driver, vehicle, highway, environment, and accident reporting factors are accounted for properly and sample size is sufficient, effectiveness measures should be generally applicable. Indeed, the larger and perhaps even more varied the data base, the more reliable the estimates of countermeasure effectiveness. Although in an initial study on countermeasure effectiveness similar test sites might be selected carefully, subsequently the countermeasure is not likely to be applied to sites as similar as the test sites. Thus, a national estimate using data from several States actually may provide a better estimate of effectiveness over the long term than one based on data from one State.

Exact measures of countermeasure effectiveness are unrealistic. The variability between similar sites is so large that even if the average effectiveness is accurately determined, the effectiveness at a particular site is likely to be quite different from the average. Indeed, it is important to determine countermeasure effectiveness accurately enough to support the use of effective countermeasures and to avoid widespread use of accident countermeasures that are ineffective or misapplied. Engineers, however, should expect only general guidance on countermeasure effectiveness. Eventually, research should be able to provide ranges of expected effectiveness for various safety measures and factors that tend to increase or decrease this effectiveness.

Summary

A vast quantity of data potentially useful for highway safety analysis is collected regularly by the States and maintained as distinct data bases for a variety of purposes. The annual national cost of all highway, vehicle, and driver data collection and maintenance is approximately $1 billion. Safety considerations alone could not justify collecting and maintaining all this information. For a more thorough analysis, States may create comprehensive computerized safety recordkeeping systems by merging data from various data bases with their accident data. An information system created from data merged from several States would provide, at a reasonable cost, the information needed for most general safety analyses of interest to FHWA.

Despite the limitations to what can be learned from the analysis of traffic accident data, rigorous analysis using both careful experimental design and proper statistical methodology can make a vital contribution to highway safety.

REFERENCES

(*1*) "Highway Safety Performance—1984: Fatal and Injury Accident Rates on Public Roads in the United States," *Offices of Highway Safety and Highway Information Management, Federal Highway Administration,* Washington, DC, January 1986.

(*2*) "Accident Facts," *Statistics Department, National Safety Council,* Chicago, IL, 1985.

(*3*) B.C. Kragh, T.R. Miller, and K.A. Reinert, "Accident Costs for Highway Safety Decisionmaking," *Public Roads,* vol. 50, No. 1, June 1986, pp. 15-20.

(*4*) F.M. Council et al., "Accident Research Manual," Report No. FHWA/RD-80/016, *Federal Highway Administration,* Washington, DC, February 1980 (PB No. 81 155152).[2]

(*5*) E. Hauer, J. Lovell, and B.N. Persaud, "New Directions for Learning About Safety Effectiveness," Report No. FHWA/RD-86/015, *Federal Highway Administration,* Washington, DC, January 1986 (PB No. 86 180759).

(*6*) H.D. Robertson and K.S. Opiela, "Feasibility of Determining the Incremental Effectiveness of Accident Countermeasures," Report No. FHWA/RD-85/043, *Federal Highway Administration,* Washington, DC, February 1985 (PB No. 86 188984).

(*7*) P.G. Hoel, "Introduction to Mathematical Statistics," *John Wiley & Sons, Inc.,* New York, NY, 1962, pp. 164-165.

(*8*) T.K. Datta, M.R. Parker, Jr., and D.A. Randolph, "Analysis of Urban Arterial Road and Street Accident Experience," Report No. FHWA/RD-82/137, *Federal Highway Administration,* Washington, DC, September 1982, p. 92 (PB No. 85 184448).

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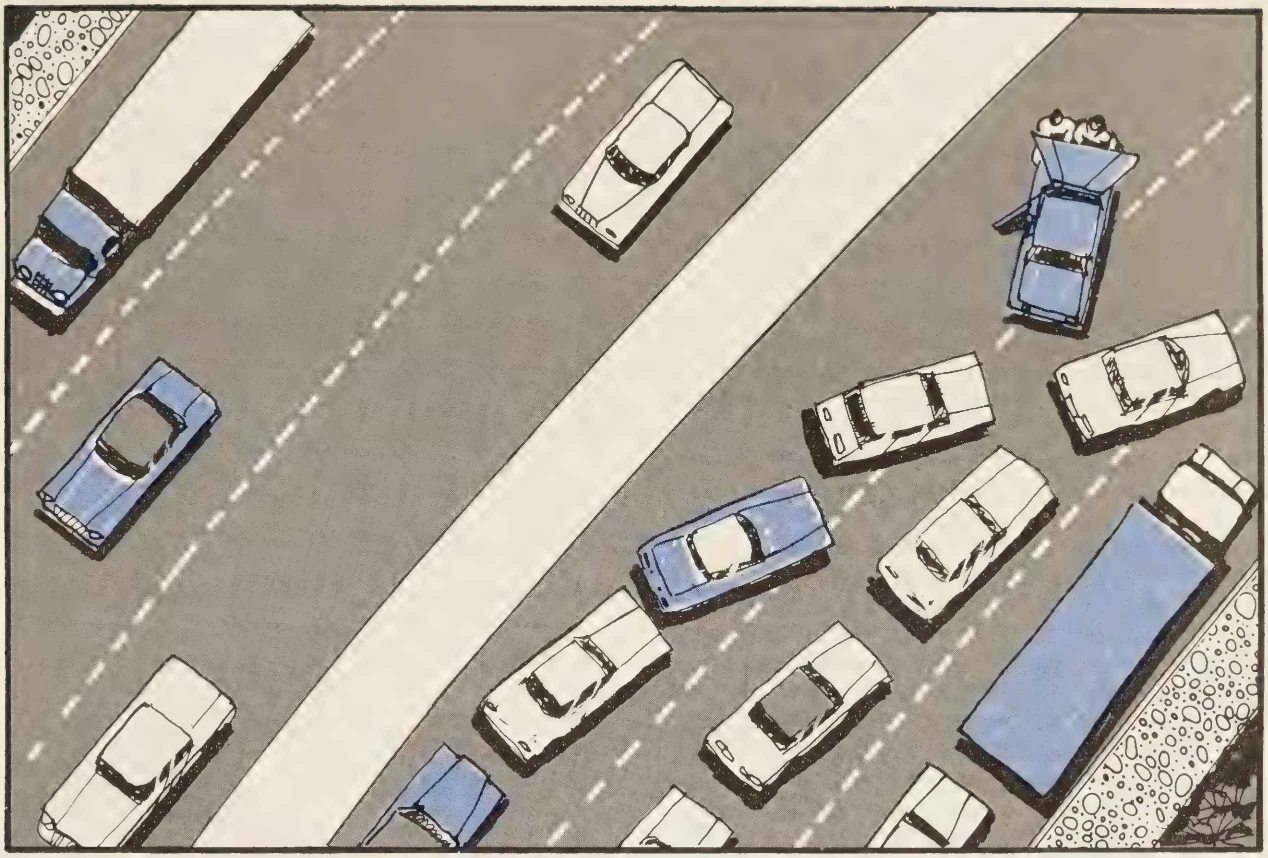
[2] Reports with PB numbers may be purchased from the National Technical Information Service, 5285 Port Royal Rd., Springfield, VA 22161.



Analytical Procedures for Estimating Freeway Traffic Congestion

by
Juan M. Morales

Introduction

A freeway incident—an accident, stalled vehicle, spilled load, or any other event that reduces the normal capacity of the roadway—causes motorist delay. Freeway incident management techniques are directed at reducing this delay, which varies with traffic volume, number of lanes, and the duration of the incident.

The Federal Highway Administration (FHWA) funded research in the late 1970's to develop guidelines and recommendations to help highway departments, police agencies, and other organizations select, plan, design, and implement low-cost measures to deal with incidents that cause freeway congestion. The research results were published in a six-volume report, which presented an overview of the nature and magnitude of the freeway incident management problem and summarized possible solutions. (*1*)[1] An analytical procedure to estimate traffic delay and congestion and assess the tradeoffs in cost-effectiveness among many alternative measures also was included in the reports. Computational examples and delay, time, and queue tables for typical conditions were provided.

This article summarizes the basic analytical procedures presented in these reports and describes a new, user-friendly microcomputer model for quickly and easily computing delay, time-to-normal flow (TNF), and maximum queue (Q_{max}) caused by freeway incidents.

[1] Italic numbers in parentheses identify references on page 61.

Representation of Incident Delay

The procedures presented in this article rely heavily on the development of a simple technique for estimating total vehicle-hours of delay. Any freeway incident can cause delay by reducing the number of vehicles that can pass the incident in a given period of time. Even vehicles removed to the freeway's shoulder will reduce capacity as motorists slow to stare at the emergency activities.

To quantify this delay, traffic volumes and incident durations can be graphically represented, as shown in figure 1. The horizontal axis is a timeline indicating the occurrence of incident-related events and the overall duration of their impact on traffic flow. The vertical axis is the cumulative traffic volume—the sum of the vehicles passing any given point on the freeway in a defined time period.

The demand flow or volume—the total number of vehicles using the freeway at a given time—is represented by the slope of L1. When an incident occurs (Time A), the reduced roadway capacity (L2) is less than the demand flow because of a lane blockage. This reduced capacity remains in effect until the incident is cleared from the freeway (Time B). At that time, the queued traffic can begin to flow at a "getaway" capacity (L3) approaching the freeway's capacity. When the last vehicle in the queue reaches the normal flow speed and traffic resumes flowing at the demand volume (Time C), the effects of the incident are over.

The getaway capacity, or the rate at which vehicles can depart a standing queue, is, in some cases, less than the typical capacity rate (under ideal conditions) of 2,000 passenger cars per hour per lane (pcphpl). Various observations of freeway getaway rates range from as low as 1,500 pcphpl to as high as 2,000 pcphpl. (*2-4*) Local driving characteristics have a major influence on this range. The analytical procedures described in this article assume the getaway capacity to be equal to the freeway's capacity.

Factors Affecting Incident Duration

A number of factors determine the magnitude of incident-caused delay, which is represented by the shaded area in figure 1. Only some of these factors can be influenced by freeway incident management techniques. Other factors, such as the freeway's capacity and demand flow, generally are fixed by external environmental circumstances such as the number of lanes and time of day. Unless an incident occurs just before or at the end of a peak period or traffic is diverted during an incident, the demand flow rate is assumed to remain constant for the duration of the incident.

Two factors that can be influenced by incident management techniques are the reduced capacity past the incident and the incident's total duration. Effective onsite traffic management techniques optimize use of whatever freeway capacity remains after the incident. Graphically, this is represented in figure 2 by an increase of the slope of the reduced roadway capacity L2 to create an improved flow rate L2′.

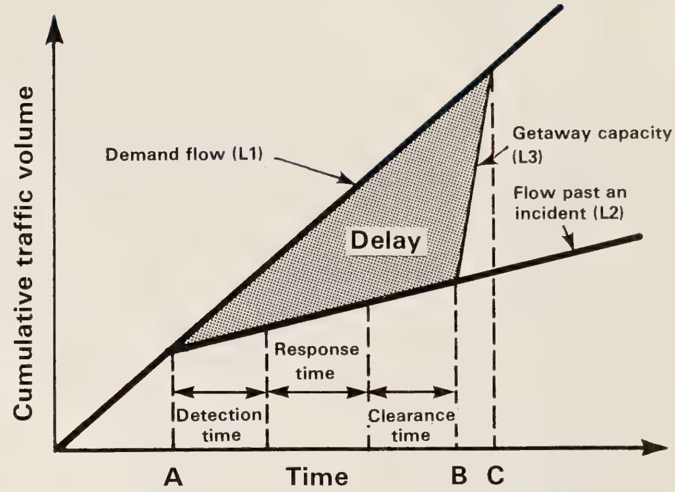


Figure 1.—Quantifying delay caused by a freeway incident.

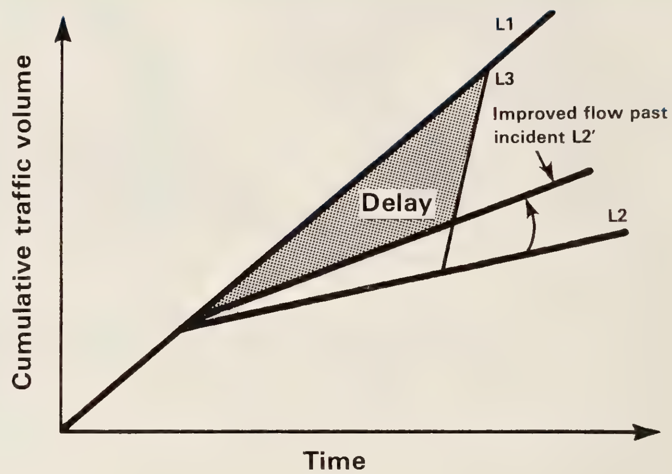
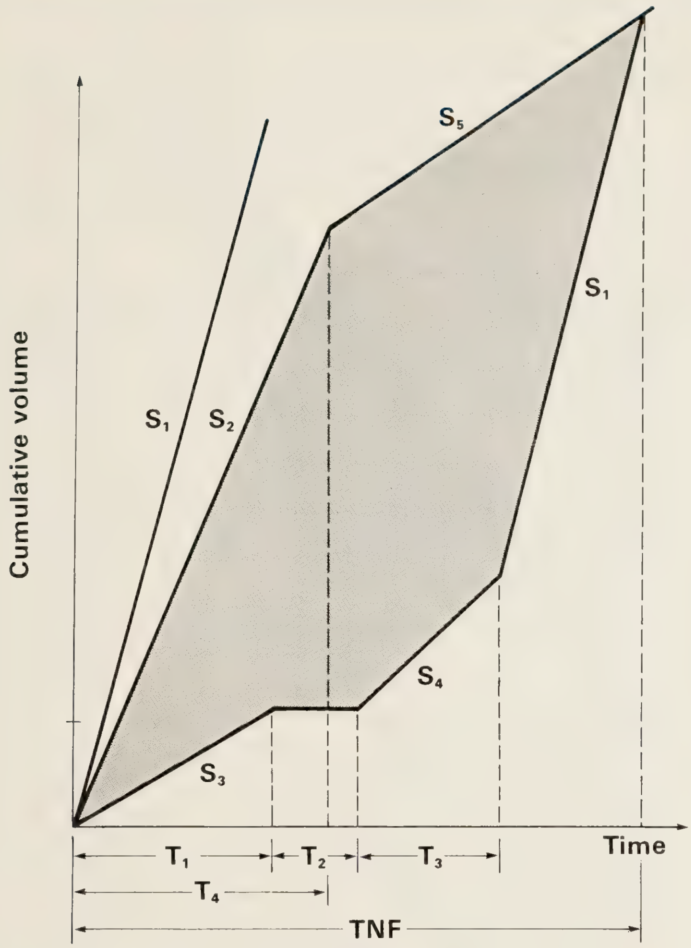


Figure 2.—Delay reduction caused by increasing flow past the incident.

Another factor influencing total delay is the time from the moment the incident occurs to the time it is cleared from the freeway. This time interval AB can be expressed as the sum of the detection, response, and clearance times as shown in figure 1. Obviously, minimizing any of these times through efficient incident management will result in less total delay.



S_1 = Capacity flow rate of the freeway, veh/hr.
S_2 = Initial demand flow rate, veh/hr.
S_3 = Initial bottleneck flow rate, veh/hr.
S_4 = Adjusted bottleneck flow rate, veh/hr.
S_5 = Revised demand flow rate, veh/hr.
T_1 = Incident duration until first change, min.
T_2 = Duration of total closure, min.
T_3 = Incident duration under adjusted flow, min.
T_4 = Elapsed time under initial demand, min.
TNF = Total elapsed time until normal flow resumed, min.

Figure 3.—General condition diagram.

Procedures for Estimating Delay

Delay can be estimated for a variety of incident management situations from a general condition diagram (fig. 3). From this diagram, the following equation for computing delay can be derived:

$$
\begin{aligned}
\text{Total delay} = [&T_1^2(S_1 - S_3)\ (S_5 - S_3) + T_2^2S_1S_5 \\
&+ T_3^2(S_1 - S_4)\ (S_5 - S_4) \\
&- T_4^2(S_1 - S_2)\ (S_2 - S_5) \\
&+ 2T_1T_2S_1(S_5 - S_3) \\
&+ 2T_1T_3\ (S_1 - S_4)\ (S_5 - S_3) \\
&+ 2T_1T_4(S_1 - S_3)\ (S_2 - S_5) \\
&+ 2T_2S_5(S_1 - S_4) \\
&+ 2T_2T_4S_1(S_2 - S_5) \\
&+ 2T_3T_4(S_1 - S_4)\ (S_2 - S_5)]/2(S_1 - S_5).
\end{aligned}
$$

Similarly, an expression for the TNF can be written as follows:

$$TNF = [T_1(S_1 - S_3) + T_2S_1 + T_3(S_1 - S_4) + T_4(S_2 - S_5)]/(S_1 - S_5)$$

The general equation to compute the maximum queue, Q_{max}, is somewhat more complicated and is indicated as follows:

$$Q_{max} = T_aS_2 + T_bS_5 - T_cS_3 - T_dS_4 - T_eS_1.$$

T_a, T_b, T_c, T_d, and T_e are functions of the conditions being considered and vary accordingly. However, by definition queue is the algebraic difference between the demand flow L1 and the bottleneck flow L2 at a specific time (figs. 1 and 2). Therefore, Q_{max} can be obtained graphically by computing the maximum difference between L1 and L2.

It should be noted that these expressions do not apply to every imaginable delay condition and should be used carefully. Four specific delay conditions (fig. 4), obtained from the general diagram (fig. 3) for estimating vehicle-hours of delay, are typical. In these conditions, either the demand flow rate or the reduced flow rate changes because of varying incident circumstances.

In Condition 1—simple blockage—the number of vehicles that would have gone through a point if the incident had not occurred (the demand flow) is indicated by S_2. The actual number of vehicles going through this point at the reduced flow rate is shown as S_3. The duration of the incident, from the time of occurrence until the time of clearance, is represented by the time interval T_1. After the incident has been cleared, the queue of vehicles delayed by the incident will move past the point at a getaway capacity S_1 (assumed to be equal to the freeway capacity). Traffic will continue to flow at this rate until all queued vehicles have gone through—at TNF.

The shaded area shown in Condition 1 represents the total vehicle-hours of delay for all the vehicles affected by this incident. Delay will be accumulated whenever the reduced flow rate S_3 is lower than the demand flow rate S_2.

Condition 2 is similar to Condition 1 but includes a short-term closure on the affected freeway. The time interval T_2 indicates that the freeway is completely closed and that no vehicles can go through the incident point. Vehicle-hours of delay continue to accumulate as more vehicles join the queue forming behind the closure.

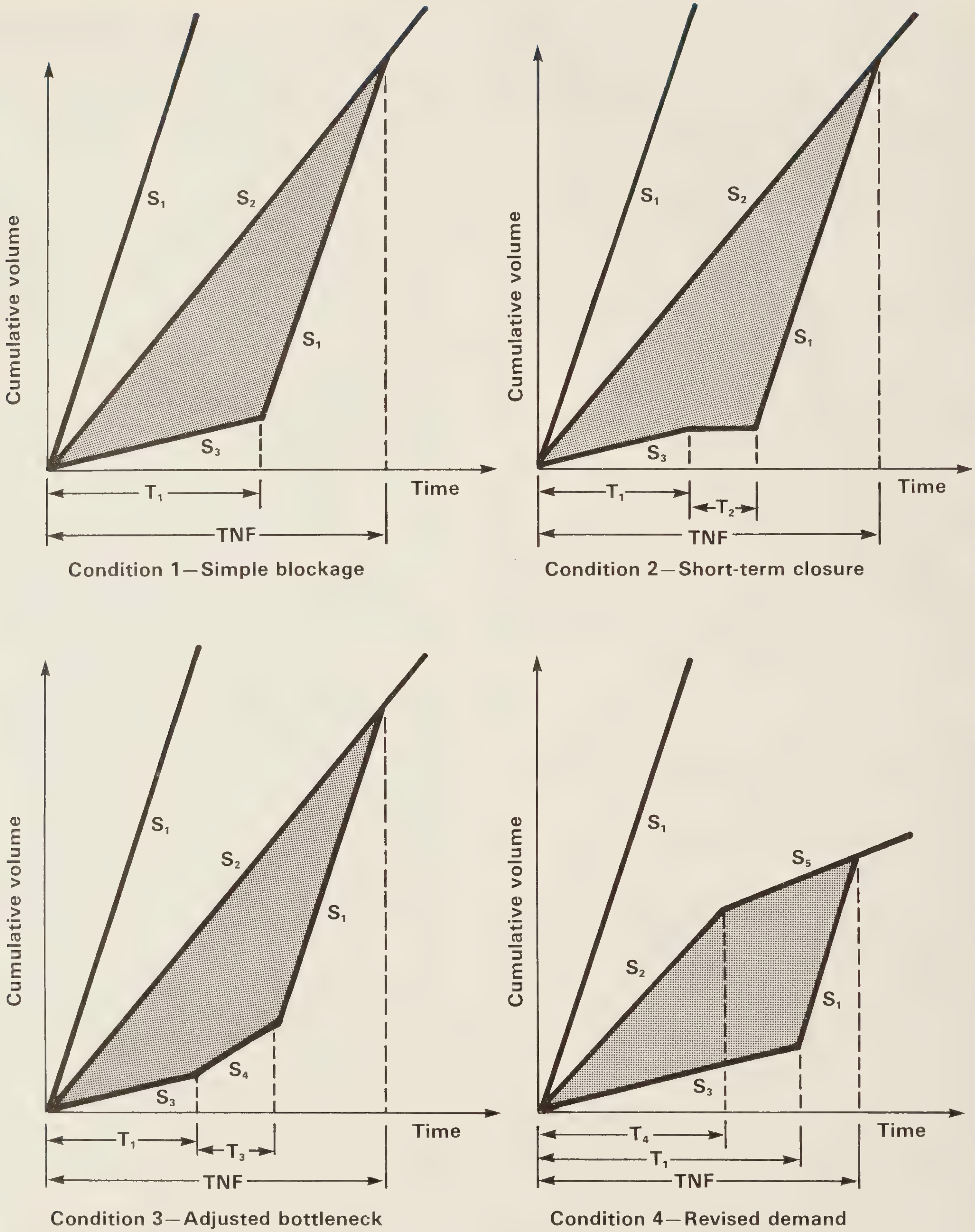


Figure 4.—Delay conditions.

Condition 3 is similar to Condition 2 except at the time interval T_1 the bottleneck flow is adjusted, and the onsite flow rate is increased for a period before total clearance by improving the flow of traffic through effective traffic management (such as police officers directing traffic) or by reopening lanes previously blocked by debris and wreckage. The time interval T_3 indicates how long this improved flow rate S_4 is in effect before the getaway capacity S_1 can be attained.

Condition 4 is created when the demand flow rate S_2 is reduced during the incident. This condition typically is caused by natural or artificial upstream traffic diversion or by typical fluctuations in traffic volumes, such as those that occur at the end of a peak period. The demand flow rate drops from S_2 to S_5 at time T_4.

With the appropriate substitutions, these four conditions can, of course, be derived from the general equations. For example, under Condition 1, $T_2 = T_3 = T_4 = 0$, $S_4 = S_3$, and $S_5 = S_2$.

Application of Procedures

Total delay is a function of three variables: Remaining capacity, traffic demand, and incident duration. At least three and up to five flow rates (depending on delay conditions) must be known or estimated to calculate delay. Some of these flow rates can be measured easily in the field for particular freeway sections. Average volumes based on historic data also can be used. Table 1 presents typical capacity flow rates S_1 and bottleneck capacity flow rates S_3 for both in-lane and shoulder incidents for freeways of two, three, and four lanes. (*1*)

Once the necessary flows and durations are known, total delay, TNF, and Q_{max} are computed by solving the general equations presented previously or by using the interactive spreadsheet. The spreadsheet uses LOTUS 1-2-3 running on an IBM-compatible microcomputer with at least 128k of memory. The program interactively guides the user through a series of screens to enter the required data (flows and incident durations) and computes the total delay, TNF, and Q_{max}. In addition, the delay condition being specified is graphically displayed. The results and graph can be printed as well.

Individual entries can be changed to determine the hypothetical effect of variations in traffic demand and/or incident duration.

Table 1.—Typical flow rates (veh/hr)

Number of lanes in one direction	Freeway capacity (S_1)	Bottleneck capacity: One lane blocked (S_3)	Bottleneck capacity: Shoulder blocked (S_3)
2	3,700	1,300	3,000
3	5,550	2,700	4,600
4	7,400	4,300	6,300

Consider the following example:

At 8:15 a.m., a three-lane freeway with a capacity of 5,550 veh/hr carries a demand flow of 4,500 veh/hr. At this time, an accident occurs and a vehicle blocks one lane, which creates a bottleneck flow of 2,700 veh/hr. It takes 25 minutes for the incident management crew to learn of the incident and arrive at the site. While the vehicle is removed, the entire freeway is closed for 5 minutes. Once the vehicle is removed, the bottleneck flow improves to 3,500 veh/hr for 10 minutes before reaching its getaway capacity (5,550 veh/hr). Hourly volumes indicate that a decrease in the demand flow (to 2,800 veh/hr) is expected at 9 a.m.

This example uses all the variables needed to compute delay and TNF as expressed in the general equations:

- S_1—the capacity flow rate of the freeway—is 5,550 veh/hr.
- S_2—the demand flow at the time of the incident—is 4,500 veh/hr.
- S_3—the initial bottleneck flow rate—is 2,700 veh/hr, which remains in effect during the 25 minutes it took the incident management crew to learn of the incident and arrive at the site (T_1).
- S_4—the adjusted bottleneck flow rate—is 3,500 veh/hr, which lasts 10 minutes (T_3).
- S_5—the revised demand—is 2,800 veh/hr, which is expected to occur at 9 a.m., or 45 minutes (T_4) after the incident.
- The entire freeway is closed for 5 minutes (T_2).

Under simpler conditions, some of these variables do not apply and are substituted with zeros.

The data are entered in the microcomputer model, and the results are obtained (fig. 5). The total delay caused by this incident is 803 vehicle-hours, and it would take 71 minutes for the effects of the incident to dissipate.

If it took only 15 minutes for the incident management crew to learn of the incident and arrive at the site, T_1 would be changed to 15 minutes (fig. 6). Total delay then would be reduced to 572 vehicle-hours, and TNF would be reduced to 61 minutes.

Summary

This article describes analytical procedures to estimate delay, TNF, and the Q_{max} caused by freeway incidents and discusses the availability of an interactive LOTUS 1-2-3 spreadsheet for fast computations. This microcomputer tool easily can be used to estimate the impact of planned incidents (that is, lane closures during construction or maintenance operations) and the consequences of freeway incidents for immediately determining the optimum traffic control strategy.

This spreadsheet can be obtained, without charge, by mailing an IBM-formatted, 5 ¼-in floppy disk to:

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Federal Highway Administration
Traffic Safety Research Division, HSR-30
6300 Georgetown Pike
McLean, VA 22101-2296
Telephone (703) 285-2499

A METHOD FOR CALCULATING DELAY, TIME AND QUEUE FOR TRADE-OFF ANALYSES

Any Place, U.S.A.

Number of Lanes: 3

Capacity flow rate of the facility, veh/hr............S1 =	5550
Initial demand flow rate, veh/hr......................S2 =	4500
Initial bottleneck flow rate, veh/hr..................S3 =	2700
Adjusted bottleneck flow rate, veh/hour...............S4 =	3500
Revised demand flow rate, veh/hr......................S5 =	2800
Incident duration until first change, min.............T1 =	25
Duration of total closure, min........................T2 =	5
Incident duration under adjusted flow, min............T3 =	10
Elapsed time under initial demand, min................T4 =	45

RESULTS:	
Total Delay, veh-hrs =	803.4
Time to Normal Flow (TNF), min =	71.3
Maximum extent of queue, veh =	1375
Maximum length of queue, miles =	2.60

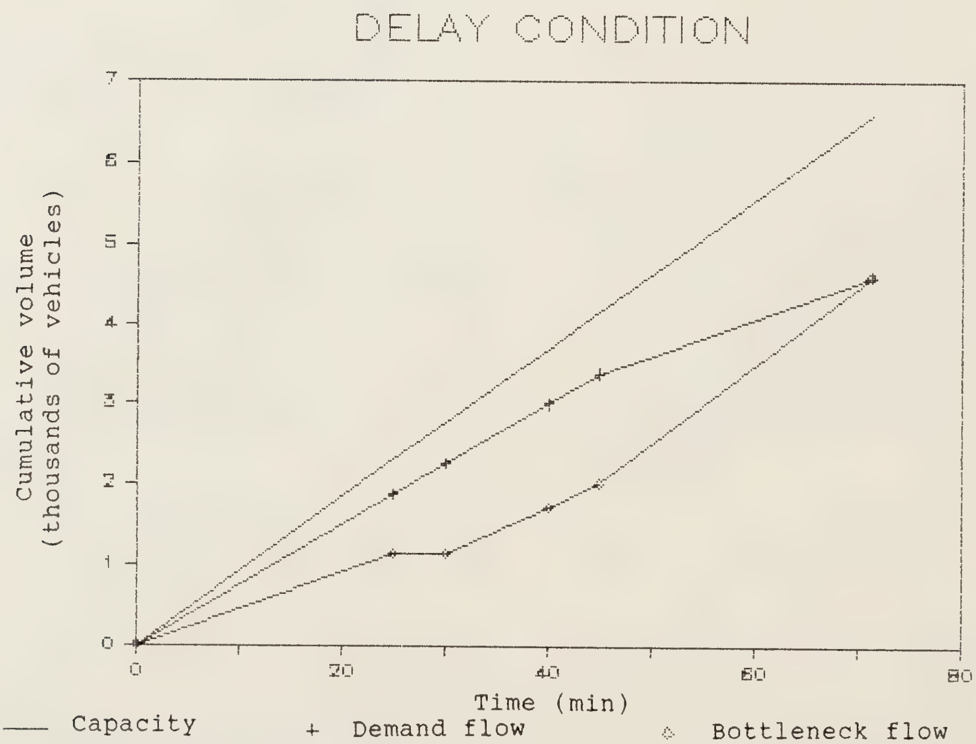

Figure 5.—Sample printout—25-minute detection and arrival time.

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A METHOD FOR CALCULATING DELAY, TIME AND QUEUE FOR TRADE-OFF ANALYSES
======================================================================

Any Place, U.S.A.
                                        Number of Lanes:           3

 Capacity flow rate of the facility, veh/hr............S1 =       5550
 Initial demand flow rate, veh/hr......................S2 =       4500
 Initial bottleneck flow rate, veh/hr..................S3 =       2700
 Adjusted bottleneck flow rate, veh/hour...............S4 =       3500
 Revised demand flow rate, veh/hr......................S5 =       2800
 Incident duration until first change, min.............T1 =         15
 Duration of total closure, min........................T2 =          5
 Incident duration under adjusted flow, min............T3 =         10
 Elapsed time under initial demand, min................T4 =         45

          RESULTS:                  Total Delay, veh-hrs =       572.5
                           Time to Normal Flow (TNF), min =       60.9
                             Maximum extent of queue, veh =       1242
                           Maximum length of queue, miles =       2.35
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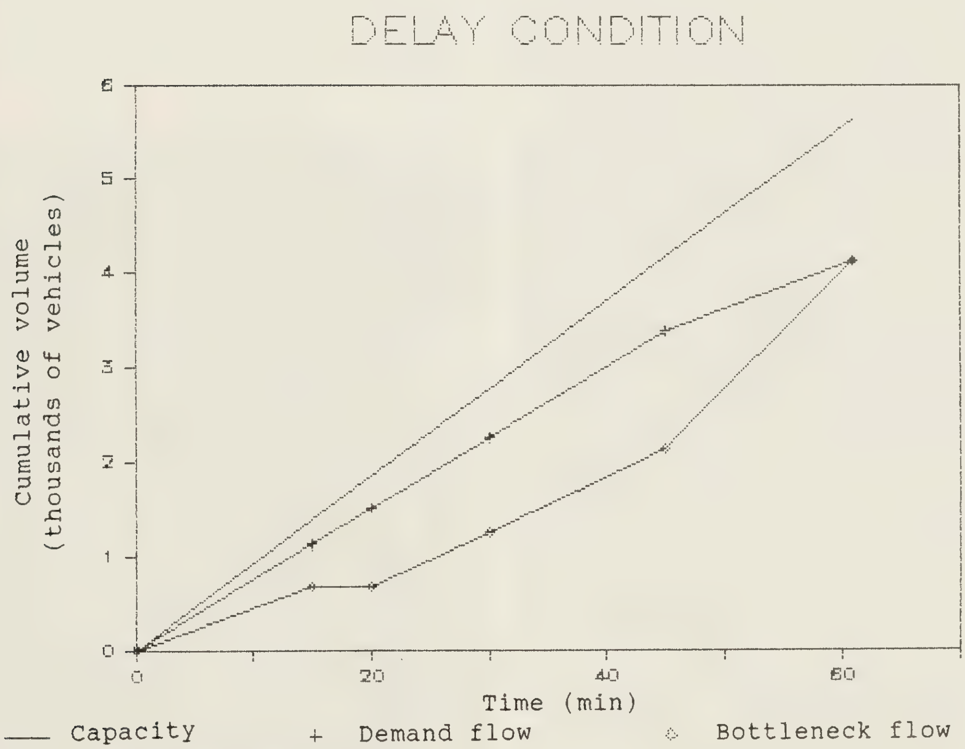

Figure 6.—Sample printout—15-minute detection and arrival time.

REFERENCES

(*1*) "Alternative Surveillance Concepts and Methods for Freeway Incident Management," Report Nos. FHWA-RD-77-58/63, *Federal Highway Administration,* Washington, DC, March 1978.

(*2*) J.A. Lindley and S.C. Tignor, "Getaway Flow Rates for Freeway Incident and Geometric Bottlenecks," *Public Roads,* vol. 43, No. 1, June 1979, pp. 1-7.

(*3*) J. Duke, J. Shofer, and A. May, "A Statistical Analysis of Speed-Density Hypotheses," Highway Research Record 154, *Transportation Research Board,* Washington, DC, 1967.

(*4*) "Highway Capacity Manual—Special Report 209," *Transportation Research Board,* Washington, DC, 1985, p. 6-7.

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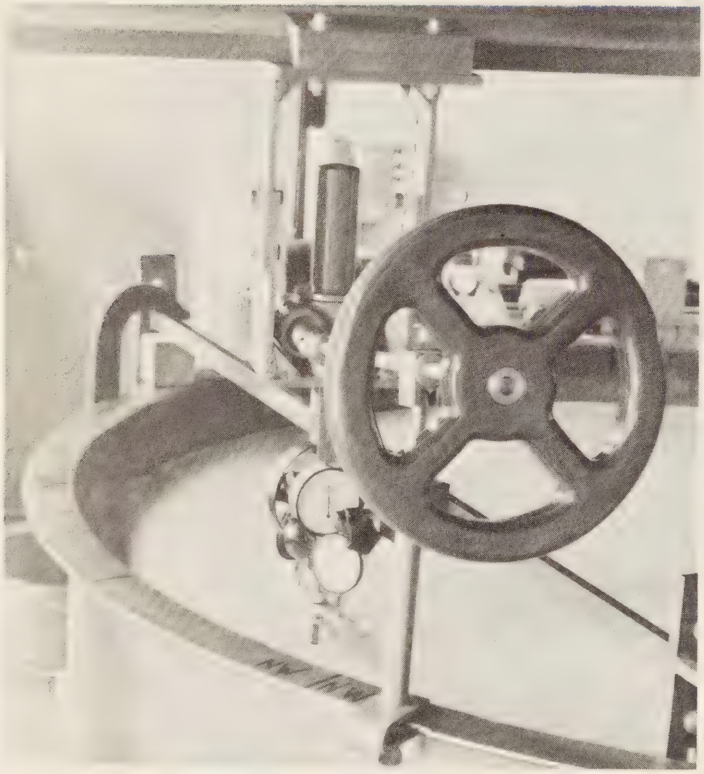
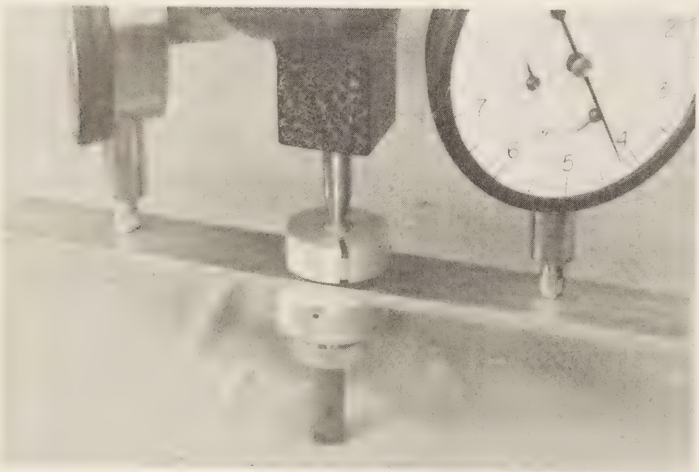
Soil Mechanics Laboratory and Bridge Foundation Test Facility at the Turner-Fairbank Highway Research Center

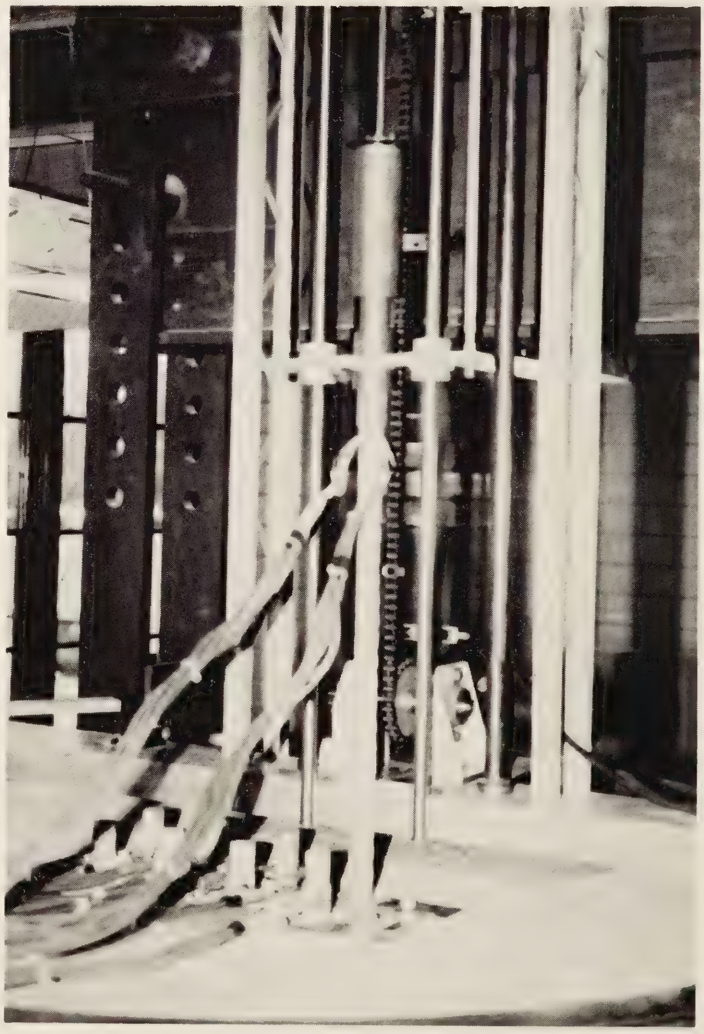
The laboratories and outdoor testing facilities at the Turner-Fairbank Highway Research Center in McLean, Virginia, enhance greatly the potential scope and quality of the Federal Highway Administration in-house research program.



Screw-actuator for applying vertical loads to a model timber pile in sand.



Closeup of load test on a model timber pile in sand.



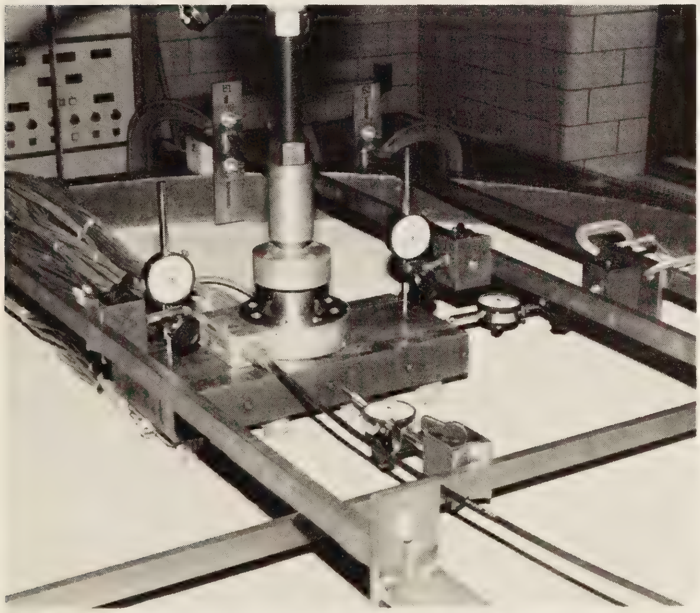
Driving one of eight fully instrumented model timber piles in sand.

The **Soil Mechanics Laboratory** is equipped to prepare soil specimens and to conduct soil mechanics tests (triaxial shear, direct shear, laboratory vane shear, incremental load, and controlled gradient consolidation) and subgrade pavement tests (California bearing ratio and Hveem stabilometer). To support staff research on small-scale piles and pile groups, the laboratory contains scaled-down pile drivers, in situ soil testing equipment, test tanks in which various foundation soils are simulated, and load test apparatus to test small-scale (1/10 to 1/100 actual size) pile groups to optimize pile foundations for highway structures.

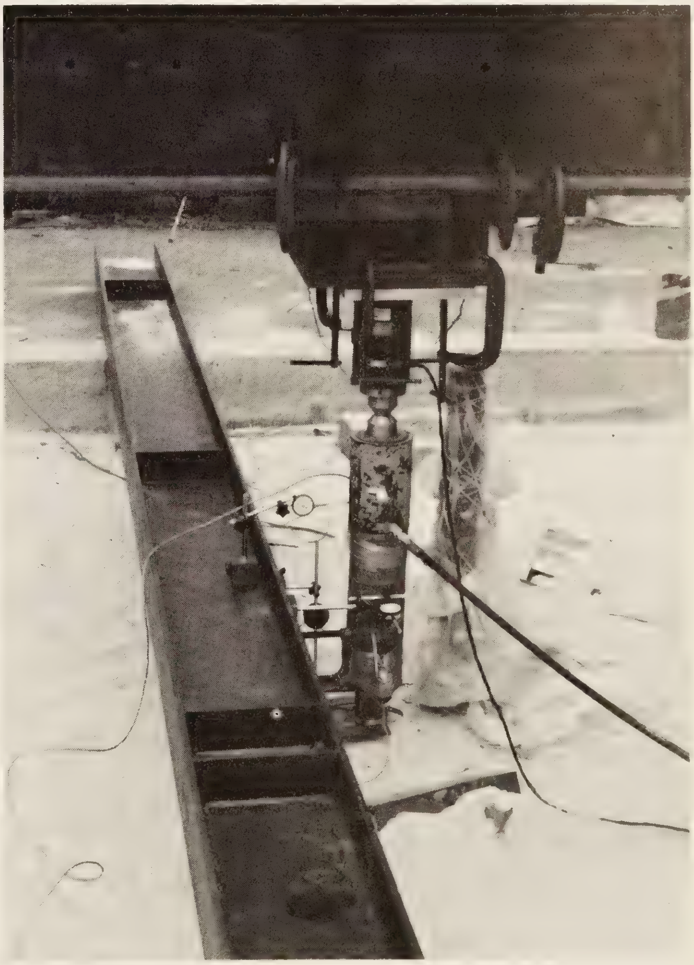
Load test on a one-quarter scale fully instrumented model timber pile in an outdoor sand test pit.

The **Bridge Foundation Test Facility** is an outdoor foundation testing facility for evaluating new design and construction concepts for spread footings and pile foundations. The test facility provides a valuable link between the small-scale testing program in the Soil Mechanics Laboratory and full-scale tests performed at actual bridge sites and other field locations. The two large test pits (20 ft x 20 ft x 20 ft [6.1 m x 6.1 m x 6.1 m]) and adjacent load test reaction systems can accommodate model sizes up to one-half scale and different soil conditions with full instrumentation and monitoring systems.

Current research in the laboratory tanks and outdoor pits involves driving and load testing model timber piles in sand. In 1987, the research in the laboratory tanks and outdoor pits will involve driving and load testing model metal piles in clay.



Load test on a fully instrumented model timber pile group in sand.



Closeup of load test on a one-quarter scale model timber pile in an outdoor sand test pit.

Recent Research Reports You Should Know About

The following are brief descriptions of selected reports recently published by the Federal Highway Administration, Offices of Research, Development, and Technology (RD&T). The Office of Engineering and Highway Operations Research and Development (R&D) includes the Structures Division, Pavements Division, and Materials Division. The Office of Safety and Traffic Operations R&D includes the Traffic Systems Division, Safety Design Division, and Traffic Safety Research Division. 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 and address requests to:

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

Electrically Conductive Polymer Concrete Overlays, Report No. FHWA/RD-84/033

by Structures Division

The development of a built-up, electrically conductive polymer concrete overlay and a premixed, electrically conductive polymer concrete mortar for use on bridge decks and other concrete members, in conjunction with cathodic protection systems, is discussed in this report. The research was divided into two phases. In the first phase, 18 commercially available resin systems and 16 conductive filler

systems were evaluated as were various parameters affecting the electrical resistivity properties of both the built-up overlay and premixed mortar systems. In the second phase, the physical and mechanical properties of the most promising overlay and premixed mortar systems were evaluated. Also, the performance characteristics of the selected overlay systems, when incorporated into active cathodic protection systems, were evaluated. The report includes recommendations regarding the further development and use of both the built-up, electrically conductive overlay and the premixed, electrically conductive mortar systems.

The report may be purchased from NTIS (PB No. 86 114873).

Evaluation of Cathodic Protection (CP) Criteria for the Rehabilitation of Bridge Decks, Report No. FHWA/RD-83/048

by Structures Division

When a bridge deck was replaced because of corrosion of the reinforcing steel from deicing salts, sections of the deck were salvaged to evaluate various cathodic protection systems. Also evaluated were the half-cell potential and constant current values obtained from the beginning of the Tafel slope on the E log I curve. The effectiveness of each criterion was monitored by electrical resistance probes (bars) that changed their electrical resistance when affected by corrosion. The probes were embedded in salt-contaminated concrete at the level of the top and bottom mat of reinforcing steel. The cathodic protection currents were applied to the concrete by cutting longitudinal anode grooves 3/8 in (9.5 mm) wide and 0.5 in (12.7 mm) deep on 1-ft (0.3-m) center-to-center spacing. The grooves contained an electrically conductive strand of carbon filament and were backfilled with electrically conductive polymer concrete. Measurements were made to determine the current distribution when the anodes were electrically energized on a 1-ft (0.3-m) center-to-center spacing. Procedures are given in the report for obtaining an E log I curve using an oscilloscope so that the measurements are not adversely affected by turning the current on and off to obtain polarized potentials.

The report may be purchased from NTIS (PB No. 86 113701).

Ice-Melting Characteristics of Calcium Magnesium Acetate, Report No. FHWA/RD-86/005

by Materials Division

This report presents pertinent chemical and physical properties of calcium magnesium acetate (CMA). Calcium-magnesium ratios varying from 100 percent calcium acetate to 100 percent magnesium acetate were compared to determine the optimum composition of CMA for road deicing.

In a 12-month study involving wetting/drying tests with 11 ratios of calcium to magnesium and 5 pH's, it was found that CMA at pH's above 7.0 and at any calcium-magnesium ratio produced no more harm to portland cement concrete than did sodium chloride. In addition, with compositions containing magnesium at levels equal to or greater than calcium, the damage to concrete was significantly reduced.

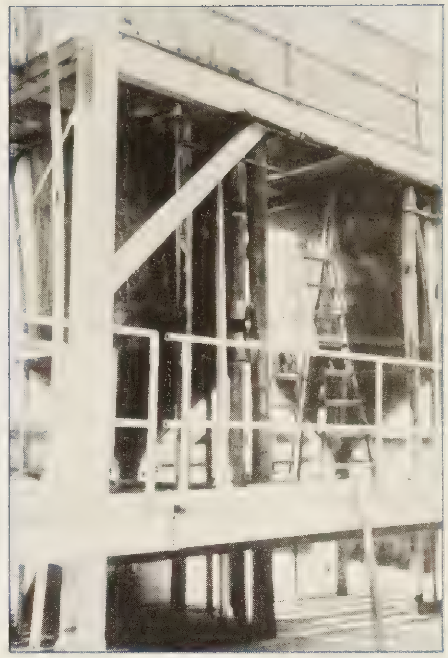
The lowest eutectic temperature (freezing point) was determined to occur with 3 mole fractions calcium to 7 mole fractions magnesium. The optimum pH for CMA was determined to be at least pH 7, with pH 9 the likely upper limit. The optimum calcium-magnesium ratio was determined to be 3Ca/7Mg $\pm$ 1 unit.

The report may be purchased from NTIS (PB No. 86 142742).

Preparation of High-Quality Calcium Magnesium Acetate Using a Pilot Plant Process, Report No. FHWA/RD-86/006

by Materials Division

In the research discussed in this report, a pilot plant-size drum pelletizer was used to produce over 200 lb (90 kg) of calcium magnesium acetate (CMA) from unslaked dolomitic lime and acetic acid. Data from the single-step operation are directly scalable to industrial units having 125 ton/hr (113 Mg/hr) nominal capacity.

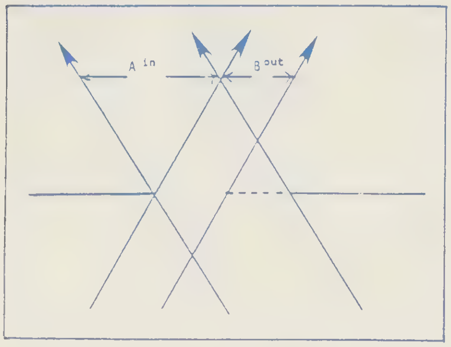


Particle-size distribution of pea-size CMA product pellets is relatively narrow. Attrition resistance is comparable to, or better than, that of commercial rock salt deicer. Product solution pH is 7.4, and at least 0.2 mols of hydration water are retained per mol of calcium acetate. The CMA produced exhibits superior ice-melting characteristics.

Capital cost is estimated at $3 million for an 80,000 ton/yr (72.6 Gg/yr) plant. At a stream factor of 0.5, total operating cost is 5.5 cents/lb (12 cents/kg) CMA beyond the acetic acid input cost.

The report may be purchased from NTIS (PB No. 86 141082).

Directional Weighting for Maximal Bandwidth Arterial Signal Optimization Programs, Volume I—Technical Report, Report No. FHWA/RD-86/020, and Volume II—Description of the Algorithm, Report No. FHWA/RD-86/022



by Traffic Systems Division

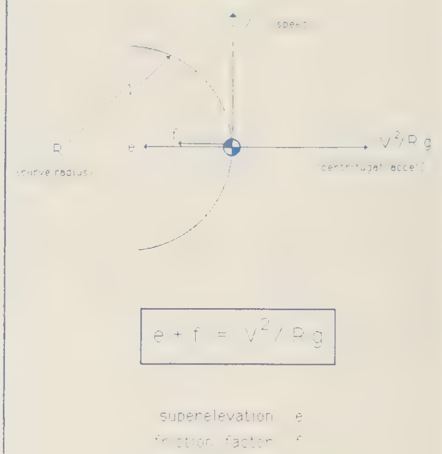
The bandwidth progression concept has been used for approximately 60 years to calculate optimal offsets in arterial signal systems. Recent studies conducted by FHWA indicated that proportioning the total two-way bandwidth in the ratio of volume distribution does not necessarily provide the lowest systemwide delay. These reports describe a study that developed an algorithm for determining the best directional weighting for arterial bandwidth optimization programs such as MAXBAND or PASSER II. This algorithm will help improve design and operation decisions, especially with heavily directional arterial street movements during peak hours.

The reports may be purchased from NTIS (PB Nos. 86 116928 and 86 116910).

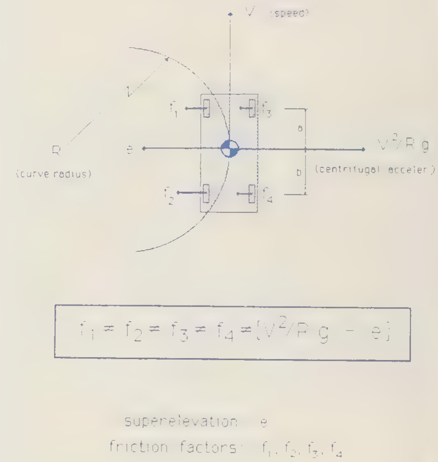
Side Friction for Superelevation on Horizontal Curves, Executive Summary, Report No. FHWA/RD-86/024, Final Technical Report, Report No. FHWA/RD-86/025, and Appendixes, Report No. FHWA/RD-86/026

by Safety Design Division

These reports present the results of a study that addressed the adequacy of point-mass representations in predicting friction requirements for vehicles operating along superelevated curves. The study focused on this issue and related questions by combining computer analysis and full-scale vehicle testing. Simple-to-use models for predicting the friction factor requirements at individual wheel locations first were developed and applied to the steady-turning condition. An existing comprehensive computer model for predicting transient or nonsteady maneuvering situations also was used to analyze friction demand while maneuvering along superelevated curves. Highway tests then were performed using two passenger cars and a five-axle tractor-semitrailer to collect representative test data and assist in validating the predictions of the computer models. Finally, a sensitivity analysis was performed to illustrate the relative importance of and interactions among various vehicle parameters and highway geometrics in influencing side friction requirements.



Point-Mass Model



Two-Axle Model

The reports may be purchased from NTIS (PB Nos. 86 114907, 86 114915, and 86 127123).

Test and Evaluation of Eccentric Loader BCT Guardrail Terminals, Volume 1—Research Report, Report No. FHWA/RD-86/009, and Volume 2—Appendixes, Report No. FHWA/RD-86/010

by Safety Design Division

These reports discuss the test and evaluation of two W-beam guardrail terminal systems known as eccentric loader breakaway cable terminals (BCT). The two terminals, while quite similar, are characterized by a 4-ft (1.2-m) flare offset and a 1.5-ft (0.5-m) flare offset. Both designs were crash tested successfully using the four-test terminal matrix described in National Cooperative Highway Research Program Report 230.

Volume 1 contains a summary of the tests, design drawings for the terminal systems, and recommendations for the implementation of these designs.

Volume 2 contains the full-scale crash test reports and describes design concepts conceived during the project that were not evaluated by crash tests.

The reports may be purchased from NTIS (PB Nos. 86 123692 and 86 123700).

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

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

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

Federal Highway Administration
RD&T Report Center, HRD-11
6300 Georgetown Pike
McLean, VA 22101-2296
Telephone: 703-285-2144

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

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

State-of-the-Practice Retroreflective Measurement Devices for Pavement Markings, Report No. FHWA-TS-84-222

by Office of Implementation

This report identifies the current use and availability of instruments for measuring the retroreflectivity of pavement marking as well as associated standards for and causes

of retroreflection from pavement markings. Tests of existing instruments were evaluated and equipment designs were compared.

The report may be purchased from NTIS (PB No. 86 165297).

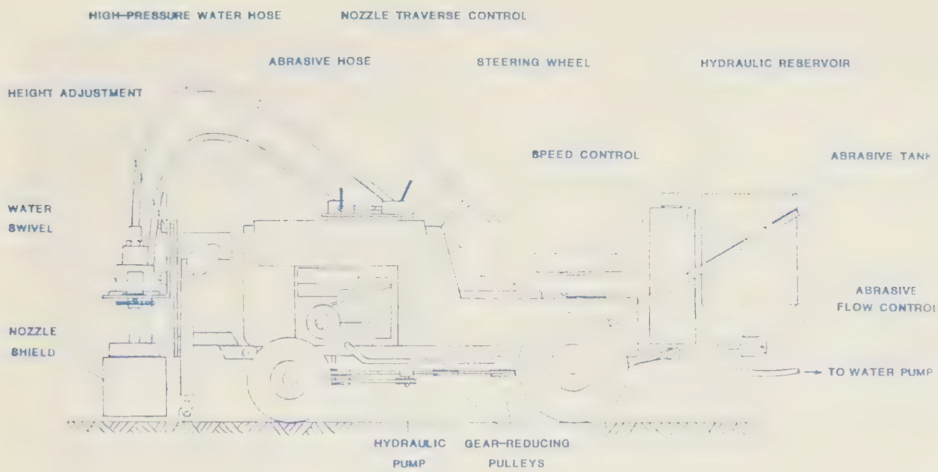
Asphalt Pavement Rutting—Western States, Report No. FHWA-TS-84-211

by Office of Implementation

A series of workshops were conducted in late 1983 to develop a set of guidelines to assist the Western Association of State Highway and Transportation Officials (WASHTO) States in preventing or reducing rutting in asphalt pavements. Representatives from most of the WASHTO States participated and provided input. It was agreed that because so many variables contributed to asphalt rutting, it is unlikely that a single cause or solution could be identified. Nevertheless, it was the consensus of the States that the highest short-term payoffs could be achieved by improving and strengthening State procedures in mixture design, materials, and construction practices. Recommendations for aggregate acceptance, paving asphalt cements, mixture design criteria, compaction control, traffic data, and information exchange were developed. In general, it was felt that rutting could be prevented or reduced if most or all of the recommendations were adopted. This report reflects the discussions at the workshops and summarizes the recommendations.

ASPHALT PAVEMENT RUTTING
WESTERN STATES

The report may be purchased from NTIS (PB No. 86 156460).



Concrete Removal With Abrasion Jet, Report No. FHWA-TS-85-221

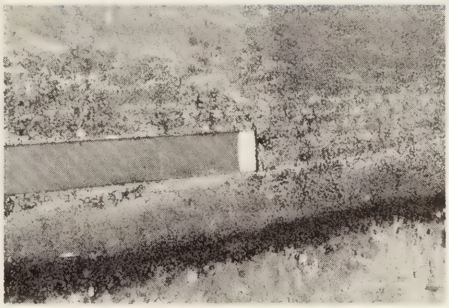
by Office of Implementation

An abrasive waterjet system was designed and built to demonstrate the system's ability to remove bridge deck concrete. The removal system consisted of two rotating abrasive nozzles mounted on the front of a garden tractor. A hydraulic system was used to move the nozzles across the front of the tractor. The abrasive tank was mounted on the rear of the tractor. The pump system was a trailer-mounted 120 hp (89.5 kW) triplex pump capable of delivering 9 gal/min (34 L/min) at a maximum pressure of 20,000 psi (138 MPa).

The field testing phase of this project was not as complete as planned, and a few system components did not perform as well as desired. The rate of removal by this rotating nozzle abrasive system is relatively low in comparison with conventional concrete removal methods.

The report may be purchased from NTIS (PB No. 86 164704).

Recessed Reflective Markers Feasibility Study, Report No. FHWA-TS-86-207



by Office of Implementation

Conventional glass bead traffic stripes generally provide poor delineation under wet-night conditions. Many States, especially in the north, have complemented the stripes with reflectors positioned adjacent to or in line with the normal striping. Reflectors, placed in recessed grooves, have shown low susceptibility to damage from snowplows.

Manually installing reflectors after the pavement has been placed requires cutting the pavement to provide the groove. Approximately 45 percent of the installation cost is for the diamond cutting blades required to cut the pavement. This report documents the results of an FHWA study that tested and evaluated candidate systems for forming the grooves during paving operations. Recommendations are included for field testing and additional research needs.

The report may be purchased from NTIS (PB No. 86 164696).

Longitudinal Edge Drains in Rigid Pavement Systems, Report No. FHWA-TS-86-208

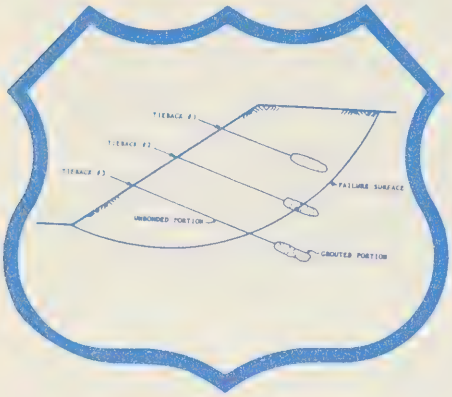
by Office of Implementation

This report documents the findings of a four-State study of longitudinal edge drain systems used in rigid pavements. Design philosophies and criteria, construction practices, and field performance comparisons are discussed, as well as where edge drains may not be advisable.

Trench drains and drainable asphalt concrete layers were found to be cost-effective in terms of their original cost and in their ability to remove water. Performance comparisons with 7-year-old pavements without drainage provisions indicate that edge drains can extend pavement life. Maintenance of the edge drain system contributed to pavement performance.

The report may be purchased from NTIS (PB No. 86 168317).

PCSTABL4 Users Manual, Report No. FHWA-TS-85-229



by Office of Implementation

This report describes the operation of the two-dimensional, limiting equilibrium, slope stability microcomputer program PCSTABL4. A short review of the program's capabilities and hardware and software requirements are presented, as are instructions for operating the program on a microcomputer. Three example problems are included to familiarize the user with the program.

The report and program may be purchased from the School of Civil Engineering, Purdue University, West Lafayette, Indiana 47907.

Tolerable Movement Criteria for Highway Bridges, Report No. FHWA-TS-85-228

by Office of Implementation

This report summarizes a comprehensive research program on the tolerable movement of highway bridges that was begun in 1978 and completed in 1983. This research included a state-of-the-art assessment of tolerable bridge movements based on a literature review; an appraisal of existing design specifications and practice; the collection and analysis of field data on foundation movements, structural damage, and the tolerance to movements for 314 bridges in the United States and Canada; and an appraisal of the reliability of the methods currently used for settlement prediction. Also included were a series of analytical studies to evaluate the effect of different magnitudes and rates of differential movement on the potential level of distress produced in a wide variety of steel and concrete bridge structures of different span lengths and stiffnesses and the development of a methodology for the design of

bridges and their foundations that embodies a rational set of criteria for tolerable bridge movements.

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

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, DC 20418.

FCP Category 1—Highway Design and Operation for Safety

FCP Project 1A: Traffic and Safety Control Devices

Title: Stop Signs Used With Flashing Traffic Signals. (FCP No. 31A2204)
Objective: Install blank-out stop signs at two locations in Orlando, Florida. Choose two similar signal locations without blank-out stop signs to serve as a control. Collect and analyze accident record data at these four locations.
Performing Organization: Development Assistance Corporation, Washington, DC 20001
Expected Completion Date: March 1987
Estimated Cost: $24,000 (FHWA Administrative Contract)

Title: Manual on Uniform Traffic Control Devices (MUTCD) Research Data Base. (FCP No. 21A3174)
Objective: Operate, maintain, and increase the computer data base developed on research related to traffic control devices.
Performing Organization: Federal Highway Administration, McLean, VA 22101
Expected Completion Date: January 1988
Estimated Cost: $8,000 (FHWA Staff Research)

FCP Project 1K: Accident and Countermeasure Analysis

Title: Application of Bayesian Statistics to Highway Safety Analysis. (FCP No. 21K3055)
Objective: Demonstrate the applicability of the empirical Bayesian statistics to highway safety analysis.
Performing Organization: Federal Highway Administration, McLean, VA 22101
Expected Completion Date: June 1987
Estimated Cost: $7,000 (FHWA Staff Research)

FCP Category 4—Pavement Design, Construction, and Management

FCP Project 4A: Pavement Management Strategies

Title: Pavement Materials and Design. (FCP No. 24A1011)
Objective: Determine the structural properties of pavement materials for pavement management research. Develop materials test and pavement analysis procedures by evaluating indirect tensile tests; analyzing data obtained from previous staff work (deflection profiles of the east and west pits at four different temperatures and three different loads); developing a pavement performance data base; developing mechanistic damage models for inverse solutions; and determining the effects of mix design and compaction on permanent deformation.
Performing Organization: Federal Highway Administration, McLean, VA 22101
Expected Completion Date: September 1987
Estimated Cost: $35,000 (FHWA Staff Research)

Title: Development of a Pavement Design and Rehabilitation Guide for Puerto Rico. (FCP No. 44A1502)
Objective: Collect and analyze data. Select test sections, collect historical data, and then conduct field and laboratory tests. Change the current pavement design guide based on data analysis. Adapt rehabilitation techniques to local conditions, and conduct a workshop.
Performing Organization: University of Puerto Rico, Mayaguez, PR 11111
Funding Agency: Puerto Rico Department of Transportation and Public Works
Expected Completion Date: March 1988
Estimated Cost: $76,185 (HP&R)

Title: Microcomputer Analysis for Project Level Pavement Management Systems for Rigid and Composite Pavements. (FCP No. 44A2332)
Objective: Develop a decision support system for project level pavement management to enable engineers to make the optimum selection of pavement strategies for individual highway segments. Develop a life-cycle cost analysis procedure for rigid and composite pavements to enable engineers to specify measures to rank alternatives and set criteria to manipulate the ranks.
Performing Organization: University of Maryland, College Park, MD 20742
Funding Agency: Maryland State Highway Administration
Expected Completion Date: March 1987
Estimated Cost: $50,000 (HP&R)

Title: Monitoring of Pavement Rehabilitation Techniques. (FCP No. 44A2392)
Objective: Monitor pavement rehabilitation techniques used in the State of Maryland, and examine other rehabilitation techniques used nationwide. Identify and recommend for use specific rehabilitation techniques that are most cost-effective for a particular kind of pavement distress.
Performing Organization: Maryland State Highway Administration, Baltimore, MD 21211
Expected Completion Date: March 1991
Estimated Cost: $17,300 (HP&R)

Title: Pavement Deflection Evaluation. (FCP No. 44A3132)
Objective: Establish, maintain, and collect statewide deflection data base (rigid, flexible, composite, and other pavement sections). Use data for model development, evaluation, verification, and modification of the design and for construction quality control of materials.
Performing Organization: University of Kentucky, Lexington, KY 40506
Funding Agency: Kentucky Transportation Cabinet
Expected Completion Date: December 1990
Estimated Cost: $25,000 (HP&R)

Title: Laboratory and Field Evaluations and Correlations of Properties of Pavement Components. (FCP No. 44A3462)
Objective: Correlate back-calculated material properties from field deflection tests. Evaluate deflection models, and study how the values change with time. Verify and modify design practices based on study results, which will give values of layer material properties. Develop acceptance tests based on moduli.
Performing Organization: University of Kentucky, Lexington, KY 40506
Funding Agency: Kentucky Transportation Cabinet
Expected Completion Date: April 1992
Estimated Cost: $20,215 (HP&R)

FCP Project 4B: Design and Rehabilitation of Rigid Pavements

Title: Portland Cement Concrete Pavement Performance and Rehabilitation. (FCP No. 44B1353)
Objective: Model selected existing urban portland cement concrete pavements in Washington State to define failure modes, estimate deterioration rates, and develop optimum rehabilitation alternatives and timing. Perform extensive in situ sampling, instrumentation, nondestructive testing, and analysis.
Performing Organization: Washington State Department of Transportation, Olympia, WA 98504
Expected Completion Date: December 1987
Estimated Cost: $200,000 (HP&R)

FCP Project 4C: Design and Rehabilitation of Flexible Pavements

Title: Fine Aggregate Shape and Surface Texture. (FCP No. 44C2124)
Objective: Evaluate the influence of variation in fine aggregate shape and surface texture on the properties of asphalt mixtures. Evaluate the ability of a series of test methods to predict the effect of these fine aggregate properties on the characteristics of the mix. Correlate these test results with standard Marshall mix design results and field performance.
Performing Organization: Western Technologies, Inc., Phoenix, AZ 85036
Funding Agency: Arizona Department of Transportation
Expected Completion Date: May 1987
Estimated Cost: $89,000 (HP&R)

Title: Evaluation of Bituminous Pavement With Polypropylene Fiber. (FCP No. 44C6265)
Objective: Evaluate the performance of a bituminous overlay of a pavement containing 0.3 percent polypropylene fiber.
Performing Organization: Mississippi State Highway Department, Jackson, MS 39215
Expected Completion Date: December 1990
Estimated Cost: $7,400 (HP&R)

☆U.S. Government Printing Office: 1986—491-825/40004

FCP Category 5—Structural Design and Hydraulics

FCP Project 5A: Bridge Loading and Design Criteria

Title: Post Delineator Mechanical Fatigue Behavior. (FCP No. 45A2242)
Objective: Review the literature related to the fatigue of flexible delineation posts, visit manufacturers, investigate long-term fatigue properties, establish performance specifications, and develop an automated performance testing system.
Performing Organization: Ohio University, Athens, OH 45701
Funding Agency: Ohio Department of Transportation
Expected Completion Date: January 1988
Estimated Cost: $81,415 (HP&R)

Title: Use of Welded Mesh in Reinforced Concrete Bridge Decks. (FCP No. 45A3272)
Objective: Develop design guidelines for using welded steel mesh in reinforced concrete bridge decks.
Performing Organization: University of Maryland, College Park, MD 20742
Funding Agency: Maryland State Highway Administration
Expected Completion Date: April 1989
Estimated Cost: $116,383 (HP&R)

Title: Distribution of Post-Tensioning Forces Prior to Grouting Tendons. (FCP No. 45A4092)
Objective: Review current facts and practices and test specimens to define the relationship and simplify the design of post-tension applications. Determine whether simplification of design calculations is justified.
Performing Organization: University of Texas at Austin, Austin, TX 78712
Funding Agency: Texas State Department of Highways and Public Transportation
Expected Completion Date: August 1987
Estimated Cost: $36,000 (HP&R)

Title: Grid Analysis of Wide, Short-Span, Continuous Curved Box Girders Using Curved Member Properties. (FCP No. 45A4122)
Objective: Develop an analysis method that enables a designer to analyze wide, short-span, continuous curved box girders by synthesizing the bridge as an assemblage of beams and slabs. Develop a computer program to perform grid analysis while incorporating curved member properties.
Performing Organization: Mississippi State University, State College, MS 39762
Funding Agency: Mississippi State Highway Department
Expected Completion Date: October 1987
Estimated Cost: $31,200 (HP&R)

Title: Evaluation of Cast-in-Place Concrete Inserts. (FCP No. 45A4132)
Objective: Perform structural tests on two common kinds and four brands of cast-in-place concrete inserts. Develop comprehensive guidelines for designers to follow when inserts are required.
Performing Organization: California Department of Transportation, Sacramento, CA 95809
Expected Completion Date: April 1989
Estimated Cost: $17,000 (HP&R)

FCP Project 5K: Bridge Rehabilitation Technology

Title: Development of Ultrasonic Techniques to Establish Flaw Size. (FCP No. 45K1252)
Objective: Study thoroughly some of the most promising techniques (such as "time-of-flight," tandem transducers, and flaw area-transducer area-misorientation angle) that are feasible for shop and field use. Fabricate test pieces with flaws whose size, type, and orientation will be determined by x ray from different directions or by sectioning.
Performing Organization: California Department of Transportation, Sacramento, CA 95809
Expected Completion Date: December 1988
Estimated Cost: $5,080 (HP&R)

FCP Project 5Z: Implementation of Structural and Hydraulics R&D

Title: Drilled Shaft Manual. (FCP No. 35ZP248)
Objective: Revise and update a previous implementation package, FHWA-IP-77-21, to reflect research and information available since 1977. Add chapters on construction techniques and specifications. Develop a workshop package and a slide-tape package that will summarize the advantages of drilled shaft foundations and encourage their consideration for a variety of foundation needs.
Performing Organization: Association of Drilled Shaft Contractors, Garland, TX 75041
Expected Completion Date: January 1988
Estimated Cost: $60,000 (FHWA Administrative Contract)

FCP Category 0—Other New Studies

Title: Conversion of Travel and Facilities Data Processing From Mainframe to Microcomputer. (FCP No. 40PN046)
Objective: Research, identify, purchase, and install microcomputer hardware and software for collecting, assembling, manipulating, and printing data in standard report form.
Performing Organization: Arizona Department of Transportation, Phoenix, AZ 85007
Expected Completion Date: March 1987
Estimated Cost: $25,000 (HP&R)

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