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COVER:
Rest area on southbound I-95 north of Gainesville, Florida.

IN THIS ISSUE

Articles

Departments

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Rest Area Wastewater Treatment

by Byron N. Lord
and Gregory W. Hughes

Why FHWA Sponsored Research in Rest Area Sewage Treatment

- **There are more than 7,700 rest areas: 6,450 are within the Federal-aid system and 1,300 are on the Interstate Highway System.**
- **1,170 of the 7,700 rest areas have flush toilets and wastewater treatment systems.**
- **The 1972 Amendments of the Federal Water Pollution Control Act (Public Law 92-500) require maximum discharge limits under the National Pollutant Discharge Elimination Permit System (NPDES) by July 1, 1977.**
- **Characteristics—chemical, biological, and flows—of rest area systems are significantly different from municipal systems.**
- **Small remote systems present unique operational limitations.**

Introduction

Public concern for environmental quality, increased mobility because of the Interstate Highway System, and the Federal Water Pollution Control Act amendments of 1972 (Public Law [PL] 92-500) prompted the Federal Highway Administration (FHWA) to initiate a program to minimize the effect of highways on water quality. One segment of the FHWA research effort was undertaken by the Environmental Effects Laboratory, U.S. Army Corps of Engineers, Waterways Experiment Station, Vicksburg, Miss. This two-phase study[1] was to develop recommendations and guidelines for bringing existing and new rest area sanitary wastewater treatment systems into compliance with the 1977 requirements of PL 92-500.

In Phase I of the study, information was gathered to determine the conditions of existing rest area facilities. Types and sizes of existing wastewater treatment systems, operational characteristics, and design parameters were given particular attention.[2]

Phase II of the study determined and recommended those wastewater treatment systems that comply with the 1977 requirements of PL 92-500. Phase II also included the investigation of an extended aeration activated sludge package treatment plant with emphasis on the capability to meet the 1977 requirements of PL 92-500. In addition, design guidelines, criteria, and recommendations for selecting wastewater treatment systems for rest areas were prepared.

[1] The study was entitled "Sewage Treatment Methods to Meet the 1977 Requirements of Public Law 92-500, The Federal Water Pollution Control Act."

[2] "Safety Rest Area Sewage Treatment Methods; Phase I: State of the Practice, Current Technology, Interim Design Criteria and Regulations," by N. R. Francingues, G. W. Hughes, et al., Report No. FHWA-RD-76-64, *Federal Highway Administration,* Washington, D.C., July 1976.

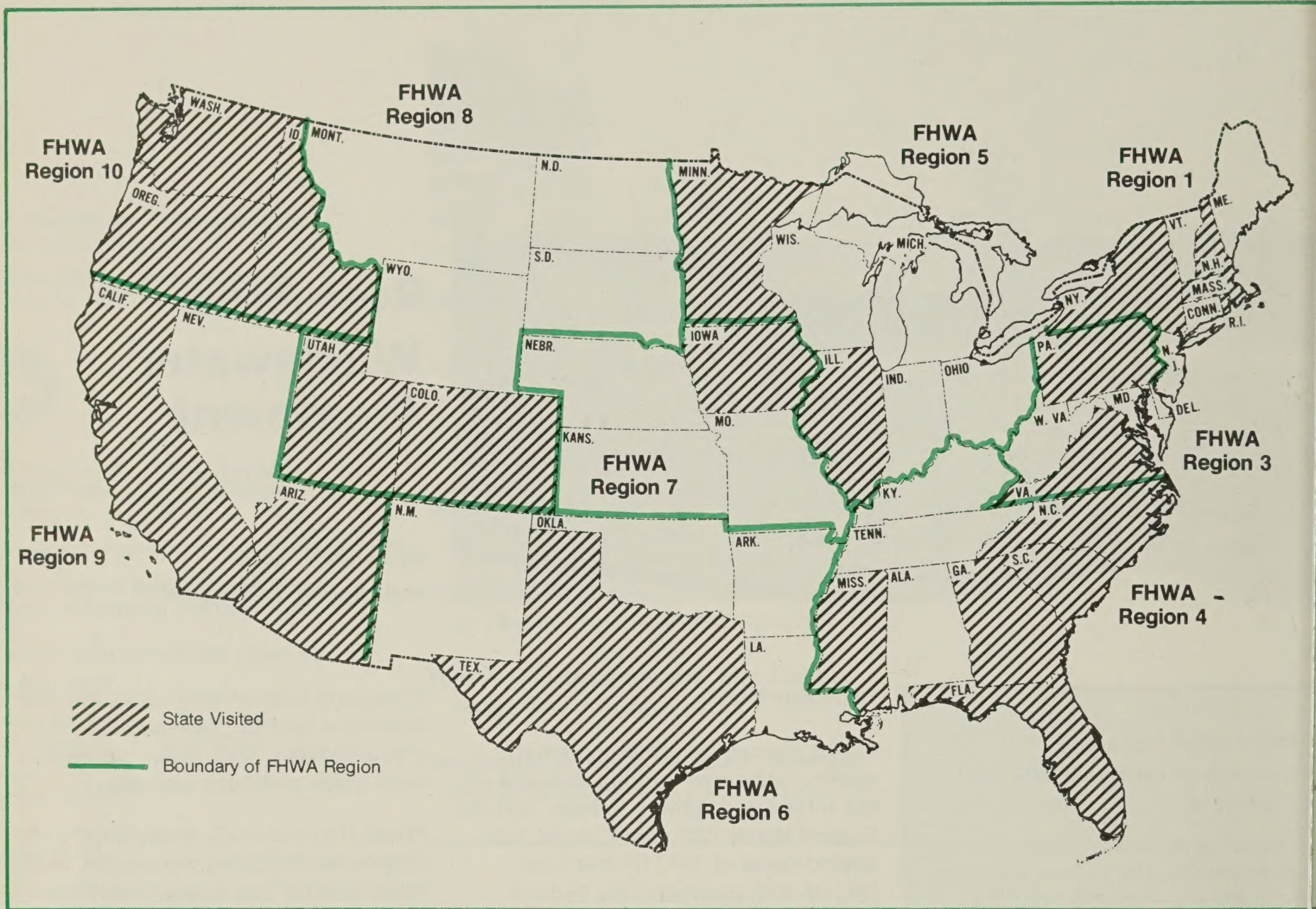

Figure 1.—States visited in Phase I of the study.

Phase I: Assessment of Treatment Systems

Phase I consisted of a literature review and field visits to 21 States (fig. 1). These field visits had a threefold purpose: (1) to study types and sizes of sewage treatment systems, particularly their operational characteristics and design parameters; (2) to evaluate the condition of existing systems in terms of their capabilities to meet the 1977 requirements; and (3) to determine current State requirements for rest area wastewater treatment. Results of Phase I were used in Phase II to develop specific design and operation guidelines.

Types of systems

In a survey of State highway agencies, the most desirable method for wastewater treatment was found to be discharge to a municipal sewage system. This method relieves the State highway department of the responsibility of constructing an onsite treatment and disposal system, reduces operation a maintenance costs, and fulfills the State's obligation to provide sewage treatment and removal of wastewate from rest areas. However, this is not very common method because most rest areas are not near municipalitie Therefore, States must supply onsite treatment facilities.

The most common onsite rest area wastewater treatment methods in us re septic tanks with leach fields or sand ilters, oxidation ponds, and extended eration activated sludge package lants.

The septic tank system is inexpensive, asy to install and maintain, and can andle small variations in flow and eriods of nonuse. However, in many ases, high groundwater, high precipi- ation, and poor soil conditions may nhibit the effectiveness of the leach ields.

Jse of oxidation ponds is limited ecause of large land requirements. Although the ponds have several ad- antages—insensitivity to organic or ydraulic shock loading, low operation nd maintenance costs, and no need or power or mechanical equipment— hey rarely achieve secondary treat- nent levels, are laden with algae in ummer, and may be accompanied by dors and groundwater pollution.

xtended aeration systems can be in- talled in narrow rights-of-way or areas f high precipitation and can have nits added to meet increased volume. hough effective, the use of extended eration systems is restricted by high nitial cost, high operation and mainte- ance costs, and the need for trained peration personnel.

.ess common treatment systems nclude various physical-chemical lants, land treatment, and evaporative agoons. In trial studies, two of the hysical-chemical systems appeared articularly promising. However, they re expensive and cannot economically eplace conventional systems unless ero discharge is required or the facil- ties are located in remote areas where vater supplies are limited. The land reatment systems are still being re- searched. They offer an advanced degree of treatment, low design costs, low operation and maintenance costs, and do not discharge to surface waters. Evaporative lagoons eliminate discharge, are inexpensive to operate and maintain, and can withstand large fluctuations in flow and shock load- ings. However, this method is limited to areas having low precipitation and high evaporation, that is, where evap- oration exceeds precipitation plus volume of wastewater. Predominant use of evaporative lagoons in these types of areas was the only instance where the study showed a relationship between the treatment system used and factors of climate, soil, precipita- tion, and normal annual temperature.

Table 1 contains a regional breakdown of rest area sewage treatment systems which were surveyed in the spring of 1975.

Table 1.—Survey of sewage treatment types
(Spring 1975)

Type of sewage treatment	Number reported in each FHWA region[1]									Total
	1	3	4	5	6	7	8	9	10	
Septic tank-leach field	11	10	16	11	24	—	11	67	30	180
Lagoon (aerobic, facultative, or evaporative)	—	2	[2]10	2	—	1	12	4	1	22
Extended aeration package plant[3]	—	23	72	—	10	—	6	5	—	116
Chemical vault holding tank	—	—	—	1	—	—	3	15	18	37
Discharge to a municipality	1	13	1	2	1	—	4	3	—	25
Recirculation-incineration	1	—	—	—	1	—	1	—	1	4
Physical-chemical	—	6	—	—	—	—	—	—	1	7
Septic tank-sand filter	1	26	4	—	—	—	—	—	1	31
Total facilities for FHWA region	14	80	93	16	36	1	37	94	51	422

[1]See figure 1 for location of FHWA regions.

[2]Used in conjunction with extended aeration plants; does not show in totals.

[3]Some extended aeration plants used in conjunction with leach fields, sand filters, or spray irrigation.

Design problems

Providing wastewater treatment and disposal facilities that can accommo- date great variations in waste flow and composition is a problem in rest area planning. In addition, most rest area treatment facilities must be sized to meet the expected usage for the design life of the area (presently 20 years). In out-of-the-way areas, inadequate supply of water is frequently a difficulty.

Another problem has been the lack of adequate design criteria. Since the characteristics of rest area wastewaters were not always known, most States based their facility design on medium strength domestic wastewater. The flow was determined by assuming that a percentage of the predicted roadway average daily traffic (ADT) would enter the rest area, assigning an assumed occupancy to each vehicle, and assuming a wastewater production for each occupant. However, it was realized that the traffic, and the volume and characteristics of wastewater flow at any given facility vary hourly and seasonally and are site-specific.

Design criteria now available[3 4] shows that wastewater flow can be predicted more accurately by using the average roadway traffic for the three peak months of the year or the six peak weekends. The total daily wastewater production can be computed from this value by assuming a percentage of the ADT stopping at the rest area and assigning the wastewater flow in gallons (litres) per vehicle.

The Environmental Effects Laboratory has compiled tables giving design criteria for the FHWA regions.[5] These tables, along with knowledge of local climate, traffic, and any nearby rest areas, can be used to check the design of an existing rest area treatment facility and to aid in designing facilities more appropriate to needs.

Compliance with PL 92-500

It was required that by July 1, 1977, publicly owned treatment facilities comply with effluent limitations that reflect the application of secondary treatment for all point source wastewater discharges. Important requirements of Environmental Protection Agency (EPA) Regulations that define secondary treatment are given in table 2. All rest areas nationwide were required to achieve these effluent limitations.

Also required was achievement of any State water quality and treatment standards more stringent than the EPA standards. Water quality standards are the criteria established for lakes, streams, and ocean waters to protect these waters for beneficial uses. One goal of PL 92-500 is to make *all* waters of the United States suitable for fishing or swimming by 1983. State water quality has already been upgraded in many areas to meet this goal.

Water quality parameters that may be affected by rest area discharges include dissolved oxygen concentration, fecal or total coliform count, nitrogen concentration, and phosphorous concentration. If the discharge from the rest area will cause the receiving water quality to be degraded, then advanced wastewater treatment methods may be required in addition to secondary treatment. Many States have established effluent limitations which are more stringent than secondary treatment for discharges to certain streams due to their high quality or their particular use. Antidegradation statements, included in all State standards, protect all waters from deterioration of the existing water quality. This policy may prohibit any new discharge of pollutants into certain high quality streams.

Table 2.—Secondary treatment requirements of PL 92-500

Parameter	30-day mean	7-day mean
Biochemical oxygen demand: (5-day) (arithmetic mean)		
Influent ≥ 200 mg/L	30 mg/L	45 mg/L
Influent ≤ 200 mg/L	15 percent of influent	45 mg/L
Suspended solids: (arithmetic mean)		
Influent ≥ 200 mg/L	30 mg/L	45 mg/L
Influent ≤ 200 mg/L	15 percent of influent	45 mg/L
Fecal coliform bacteria (geometric mean)	200 per 100 ml	400 per 100 ml
pH of effluent	≥6.0, ≤9.0	≥6.0, ≤9.0

Notes: (1) These requirements represent the minimum standards that must be achieved by 1977 by publicly owned facilities.
(2) Proposed rules in the *Federal Register*, Aug. 15, 1975, eliminate the fecal coliform requirement for secondary treatment and exclude from the pH requirement systems that treat only domestic waste and do not employ chemical addition as a part of the treatment process.

[3]Reference data resulting from the following unpublished report was used as an input for the development of this design criteria: "Establishment of Roadside Rest Area Water Supply, Waste Water Carriage and Solid Waste Disposal Requirements, Vols. I and II," by R. Zaltzman et al., April 1975.

[4]"Safety Rest Area Sewage Treatment Methods; Phase I: State of the Practice, Current Technology, Interim Design Criteria and Regulations," by N. R. Francingues, G. W. Hughes, et al., Report No. FHWA-RD-76-64, *Federal Highway Administration,* Washington, D.C., July 1976.

[5]Ibid.

In order to implement the goals of PL 92-500, the National Pollutant Discharge Elimination System (NPDES) was established. All point source discharges are required to apply for an NPDES permit. The permit may be issued by States having authorization for the NPDES program or by the regional EPA administrator. The permit includes effluent limitations, monitoring and reporting requirements, operational requirements, and certain general and specific conditions.

Phase II: Design and Operating Guidelines for Treatment Systems

Phase II involved a review of information gathered in Phase I and the development of design and operating guidelines for rest area sewage treatment systems. Each treatment system included in the design and operating guidelines was either capable of meeting the 1977 requirements of PL 92-500 or could be upgraded to meet these requirements through further treatment or system optimization.

The following design guidelines are included for each treatment system: (1) process description; (2) necessary design input information; (3) state-of-the-art design procedure; (4) design output, including size of the facility and treatment efficiency; (5) a design example for each treatment system; (6) the applicability of each process in different flow ranges; and (7) the ability and ease with which each treatment system can be upgraded or modified to produce a higher quality effluent. Systems currently in use at highway rest areas have been evaluated for compliance with the effluent limitations of PL 92-500. When an existing system could not meet the requirements, methods for upgrading that system were included as part of the design guidelines.

Operating guidelines for each treatment system include the degree of skill required by the treatment system operator, the approximate daily and/or weekly time required for operation, and the minimum required operational procedures. Also included with the operation guidelines are recommended maintenance schedules, that is, when to pump out a septic tank, when and how to replace aeration equipment, and how best to disperse algal mats in facultative lagoons.

Summary

Research completed to date has shown that safety rest area wastewater treatment facilities have been overdesigned hydraulically and organic loading overestimated. In the past, designs were based on estimates of municipal wastewater characteristics and "rule of thumb" wastewater generation volumes. In addition, wastewater treatment facilities have been installed with little attention to the unique usage patterns at safety rest areas. All too often, treatment plant operators are not properly trained and do not devote enough attention to monitoring and adjusting the system.

The results of this two-phase study[6] will provide a method for estimating rest area usage patterns; develop procedures for predicting wastewater generation; establish characteristics of wastewater; identify treatment technology appropriate for rest areas; develop, as necessary, modified or special technology for meeting stringent discharge requirements; provide design and operational guidelines; and conduct a pilot workshop for highway department personnel involved in rest area wastewater treatment and disposal.

[6]Available late fall 1977.

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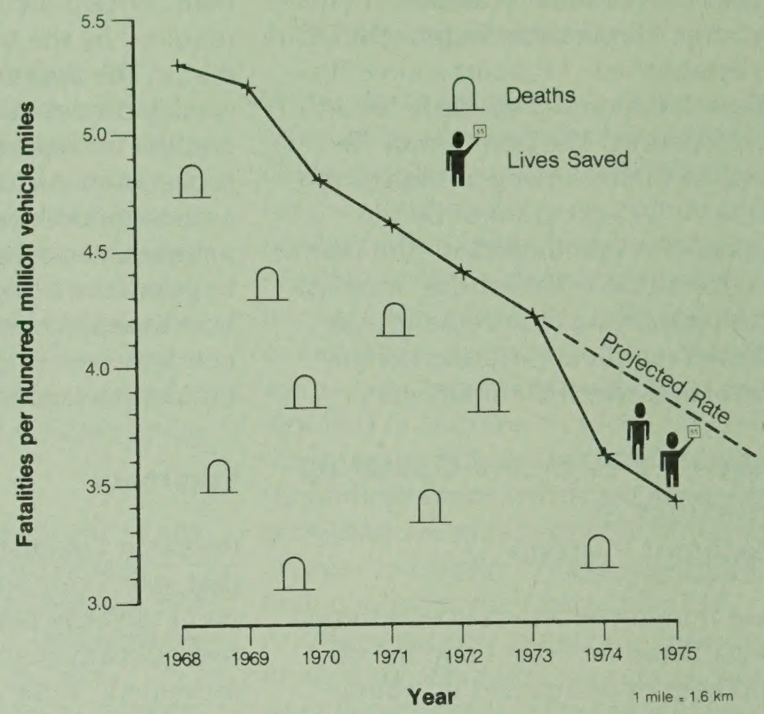
Safety Aspects of the National 55 MPH Speed Limit

by Willard J. Kemper and Stanley R. Byington



Nationwide Fatality Rate, 1968-1975

This article summarizes recent research, performed by the Pennsylvania State University, which evaluated the effect of the national 55 mph (89 km/h) speed limit on highway safety. It also contains analysis by the authors which shows that the reduction in speeds, brought about in part by the new speed limit, prevented over 4,700 fatalities and 81,000 injuries in 1974.

In the university research, fatality rates and injury rates occurring after the speed limit reduction were compared to projected rates based on pre-speed limit data. A comparison was made of nationwide data for various highway systems and for a selected representative sample of States. The study shows that, under the national 55 mph (89 km/h) speed limit, fatality rates decreased most on highways where speed limits changed most, particularly the Interstate Highway System. However, injury rates were not generally below expected rates based on past trends, except that the Interstate Highway System did show a large decrease. Additional factors studied were pedestrian fatalities, age of driver in fatal accidents, time of fatal accidents, and type of vehicle involved.

The authors' additional effort involved regression and correlation analyses of the effects of various speed measures on corresponding fatality and injury rates. Six years of data were employed in this analysis.

Introduction

The Arab oil embargo in late 1973 caused an increased emphasis on energy conservation and a realization that mandatory control would be necessary to achieve the significant fuel savings needed. Accordingly, on January 2, 1974, a 55 mph (89 km/h) speed limit was signed into law. Since that time, there have been many studies evaluating the safety effects of the 55 mph (89 km/h) speed limit. Unfortunately, such documented research had many short comings. (*1, 2, 3, 4*)[1] For example, many researchers ignored the long term trends in fatality rates, and many applied the troubling assumption that travel and fatalities are linearly related. In fact, various persons have suggeste that the 1974 drop in the fatality rate, when compared to the 1973 rate, was the result of many factors: the nationwide economic downturn, a reduction in total travel, a sh from nighttime to daytime travel, a shift from weekend to weekday travel, daylight savings time changes, and restric tions on young drivers. For the above reasons, a nationwi study was performed by the Pennsylvania State University for the Federal Highway Administration (FHWA). This stu was performed to try to overcome some of the previous re search fallacies and to consider the most relevant factors

[1]Italic numbers in parentheses identify the references on page 67.

that could possibly have had an effect on the 1974 fatality rate. The research was specifically addressed at trying to determine the true safety benefits of the 55 mph (89 km/h) speed limit. Data were collected from 17 States throughout the country. In addition, accident, speed trend, and motor vehicle travel statistics published annually by the FHWA were used.

In the research, fatality rates and injury rates since the enactment of the speed limit were compared to projected fatality and injury rates based on pre-speed limit data. This comparison was made for different highway systems in order to compare accident data on roads where the speed limit was previously over 55 mph (89 km/h) to accident data on roads where the speed limit was less than 55 mph (89 km/h). Comparisons also were made on a representative sample of the 17 States, and factors such as pedestrian fatalities, age of driver in fatal accidents, time of fatal accidents, and type of vehicle involved were considered.

The objective of this article is to summarize the university analysis and the results emanating from the study report "Safety Aspects of the National 55 MPH Speed Limit." (5) In their report, the researchers state quite strongly that "It is impossible to attach exact numerical proportions to the various causes of the reductions in fatalities" from 1973 to 1974. However, there is strong evidence to support the theory that the reduction in mean speed that occurred between 1973 and 1974 was the single most important factor contributing to the large reduction in fatalities and injuries and their respective rates during 1974. Accordingly, this hypothesis is also examined in this article and predictions are provided on how many injuries and fatalities were saved as a result of the lower travel speeds in 1974.

Nationwide Fatality, Injury, and Vehicle Mile Yearly Relationships

The energy crisis of 1973 and 1974 had a strong effect on the amount of travel done on U.S. highways and apparently on the number and severity of accidents. Figure 1 shows annual travel (measured in hundred million vehicle miles [hundred million vehicle kilometres]) on all roads in the United States from 1968 through 1975. (6) The plot clearly illustrates a definite linear trend from 1968 to 1973 followed by a decrease in travel in 1974. Interestingly, travel in 1975 increased from the 1974 value, but not at as high a rate as that from 1968 through 1973. The 1974 vehicle miles of travel was a 229 hundred million vehicle mile (368.5 hundred million vehicle kilometre) decrease from the 1973 value, but the reversal of the growth rate makes the 1974 change from its predicted value much greater.

Figure 1 also shows the annual number of highway fatalities on all roads in the United States between 1968 and 1975. (6) Prior to 1974, the annual fatality count was uniform at

Figure 1.—Nationwide travel, deaths, and death rates, 1968-1975. (6)

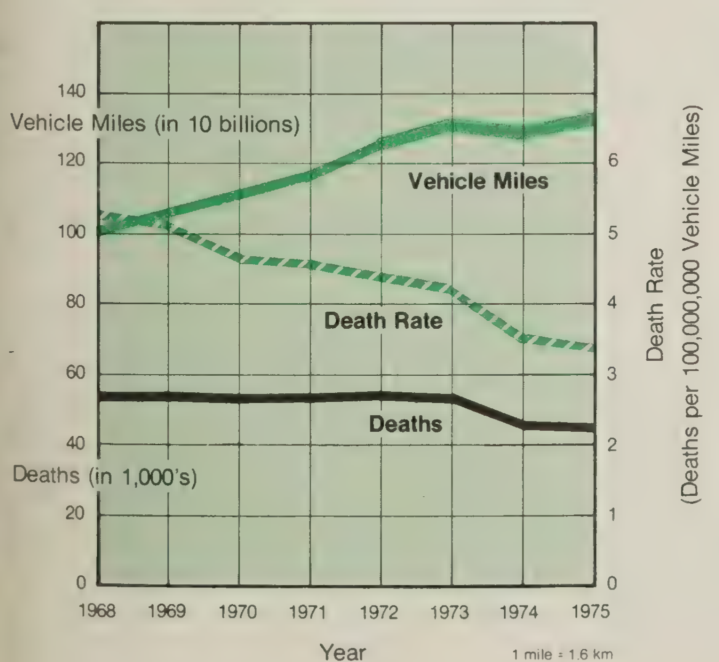


Figure 2.—Nationwide injuries and injury rates, 1968-1974. (6)

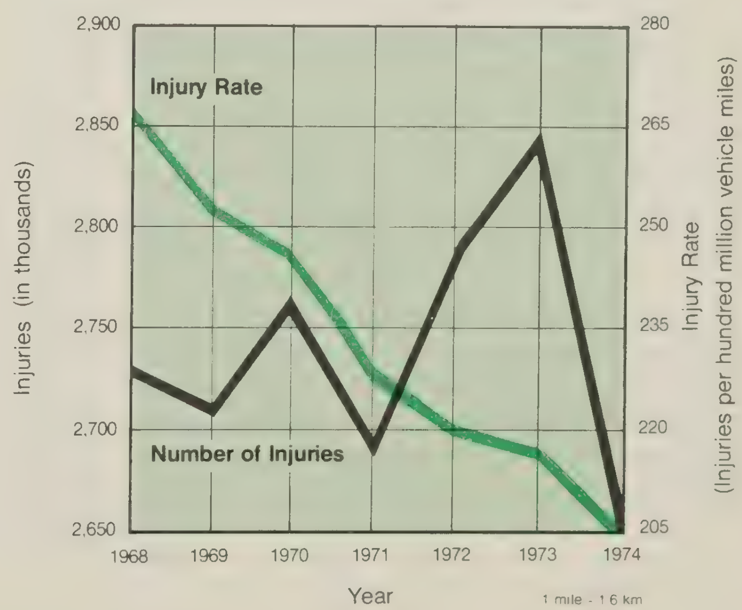


Table 1.—Average speeds of free-moving vehicles on various highway systems

Highway system	Year								
	1973			1974			1975		
	Cars	Trucks	All	Cars	Trucks	All	Cars	Trucks	All
	Mph	*Mph*	*Mph*	*Mph*	*Mph*	*Mph*	*Mph*	*Mph*	*Mph*
Rural Interstate	66.6	60.3	65.0	58.1	56.3	57.6	58.0	56.6	57.6
Urban Interstate	58.0	54.4	57.0	53.5	51.8	53.1	55.4	53.8	54.7
Rural Federal-Aid Primary	58.2	54.0	57.1	54.0	52.1	53.5	55.1	53.4	54.5

1 mph=1.6 km/h

approximately 55,000. In 1974, however, fatalities decreased to around 46,000 and dropped another 500 in 1975. Some of this decrease in fatalities was thought to be associated with reduced travel. Accordingly, fatality rates (fatalities per hundred million vehicle miles of travel [hundred million vehicle kilometres]) were calculated and are also shown in figure 1. (6) This figure shows that the death rate decreased evenly between 1968 and 1973 but, in 1974, the rate dropped sharply from 4.20 to 3.57. The 1974 drop was much greater than the annual decreases achieved before 1974. Fitting a straight line by the method of least squares to the 1968 through 1973 data produces a predicted 1974 fatality rate of 3.94. This means that the 1974 fatalities per hundred million vehicle miles [hundred million vehicle kilometres) were 0.37 lower than what might have been expected had there been no change to the system. Note that the fatality rate drop from 1974 to 1975 is at a slope approximately equal to that which occurred between 1968 and 1973.

The number of injuries and injury rate (number of injuries per hundred million vehicle miles of travel (hundred million vehicle kilometres]) in the United States between 1968 and 1974 does not exhibit the same pattern as fatalities and fatality rate (fig. 2). In fact, the Pennsylvania State University researchers concluded that, although the nationwide injuries dropped substantially in 1974 from the 1973 level, the 1974 injury rate did not appear to be significantly out of line with the decreasing trend in injury rates since 1968. The authors of this article do not agree. It appears that the injury rate started to level off, as reflected in the 1972 and 1973 rates, and as noted by the researchers, significant differences were found at the 0.05 level of significance between 1973 and 1974 injury rates on the Interstate System (both rural and urban) and on the Rural Federal-Aid Primary System.

Speed Trends

While the preceding changes in nationwide fatalities, injuries, and travel were taking place, operating speeds also changed as a result of the reduced maximum speed limit. Table 1 shows these changes for the Interstate System and the Rural Federal-Aid Primary System.

Attention should be directed to the rural Interstate highways, for virtually all of these highways had speed limits over 55 mph (89 km/h) prior to the speed limit change. In 1973, the estimated mean speed of passenger cars on the rural Interstate System was 67 mph (108 km/h), while for trucks the mean speed was 60 mph (97 km/h). In 1974, afte the speed limit change, the mean speed for passenger cars dropped to 58 mph (93 km/h) and the mean speed for trucl to 56 mph (90 km/h). Average speeds of both passenger ca and trucks remained at these levels in 1975.

Perhaps the most dramatic change in operating speeds on rural Interstates was in the percentage of passenger cars ex ceeding 65 mph (105 km/h). In 1974, this percentage dropped to approximately 10 percent from the 1973 level c 59 percent. In 1975, this percentage decreased an addition 2 percent. The percentage of cars exceeding 55 mph (89 km/h) on rural Interstates decreased from 93 percent in 1973 to 68 percent in 1974. In 1975, the percentage of cars exceeding 55 mph (89 km/h) increased slightly to 71

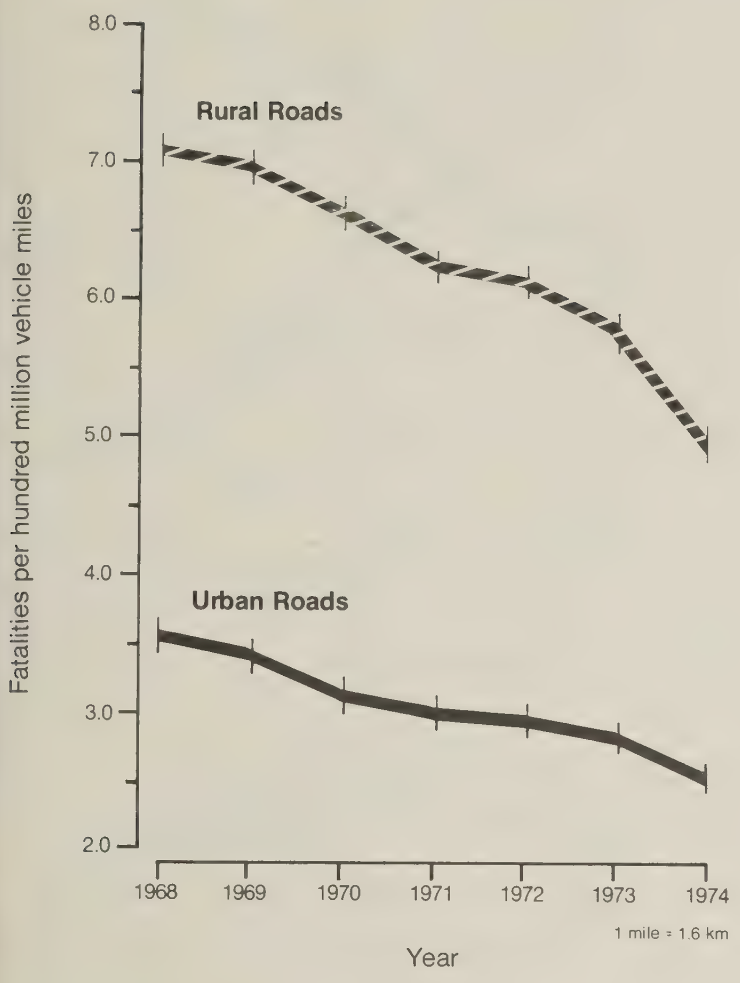


Figure 3.—U.S. fatality rate—all rural and urban roads. (6)

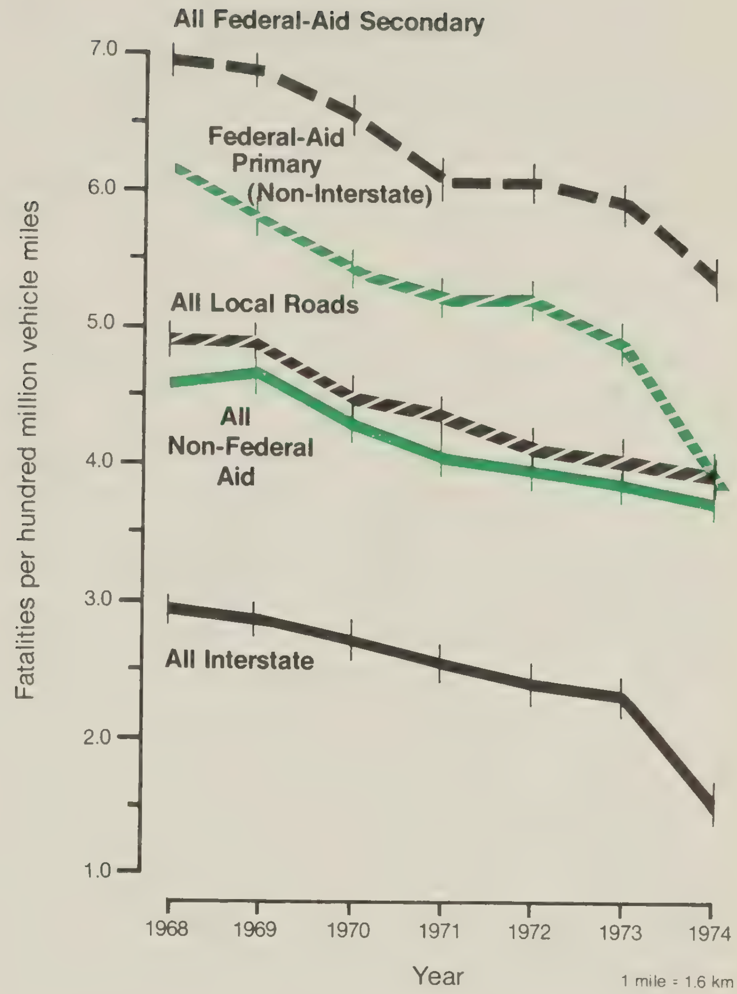


Figure 4.—U.S. fatality rate by type of road system. (6)

percent. Although speeds were generally above the 55 mph (89 km/h) speed limit, they were much more uniform than before the lower speed limit. In 1975, approximately 63 percent of passenger cars traveled between 55 and 65 mph (89 and 105 km/h). Trucks had the same speed trends in 1974 and 1975 as passenger cars. (*7*)

Fatality and Injury Rates by Highway Type

Figures 3 and 4 are plots of the fatality rates from 1968 through 1974 for different highway systems in the United States. Examination of these plots reveals a generally decreasing trend in fatality rates since 1968. For most of the highway systems, the 1974 decrease in fatality rate is larger than the annual decreases occurring between 1968 and 1973. Furthermore, there is a tendency for the highway systems with a larger proportion of high speed roads to show a larger decrease in fatality rates in 1974. For example, in figure 3, there is a much larger decrease for rural roads than for urban roads. Furthermore, figure 4 indicates large decreases in fatality rates on Interstate, Federal-aid primary, and Federal-aid secondary highways; but the decrease in fatality rates on non-Federal-aid and local roads is not unusual when compared to the decreases taking place between 1968 and 1973. Assuming that the pre-1974 trends would have continued had there been no energy crisis, the difference between the actual and predicted rates might reasonably be attributed to the lower speed limit and other energy crisis induced changes.

Nonfatal injuries per hundred million vehicle miles (hundred million vehicle kilometres) of travel did not have the same overall decrease in 1974 as did the fatality rate. Figures 5 and 6 are plots of the injury rates on different highway systems in the United States from 1968 through 1974. During that time, the injury rate was generally decreasing. On many of the highway systems, the 1974 decrease in injury rate does not appear much sharper than the typical decreases from 1968 through 1973. A notable exception is the injury rate on Interstate highways (fig. 6) which dropped sharply in 1974. The decrease in injury rate on all rural roads (fig. 5) is also larger than the previous annual decreases. These results seem to support the safety benefits of the 55 mph (89 km/h) speed limit.

Statewide Fatalities and Injuries

Fatality and injury data for a sample of 17 States[2] offering a wide range of traffic and demographic conditions were analyzed. The main reason for analyzing individual States was to determine if fatality rate decreases might be related to specific highway, demographic, or speed limit enforcement policies. However, no pattern was detected in differences between predicted rates (based on past trends) and actual rates in the sampled States. For example, although Colorado, Massachusetts, Michigan, Washington, and Wyoming had 1974 fatality rates which were above their predicted rates, these States were like other sampled States in terms of enforcement and compliance of the 55 mph (89 km/h) speed limit. Twelve States had lower actual fatality rates than predicted rates.

Statewide annual injury rates were considered in the same manner as fatality rates. Nine of the States had 1974 injury rates below the predicted rates. However, the differences between actual and predicted injury rates were relatively small for all of the States.

Pedestrians

A speed reduction from 65 mph (105 km/h) to 55 mph (89 km/h) would not be presumed to contribute substantially to the survival of any pedestrian struck by a vehicle. Pedestrian accidents were therefore used as a control on the effects of the speed limit. The value of this analysis was further enhanced by consideration of urban pedestrian accidents, which are difficult indeed to relate to the speed limit imposed in 1974. The change in these statistics in 1974 is noteworthy. Every sampled State showed a decrease in pedestrian fatalities between 1973 and 1974. For the entire sample, pedestrian fatalities dropped 16.5 percent in 1974. Urban/rural splits of pedestrian fatalities for 1971 through 1974 were available for Michigan, New York, North Carolina, Ohio, Virginia, and Washington. Urban pedestrian fatalities for the six States dropped 17.8 percent from 1973 to 1974 while rural pedestrian fatalities decreased 10.7 percent. Curiously, rural pedestrian fatalities decreased more between 1972 and 1973—14.3 percent. Since urban

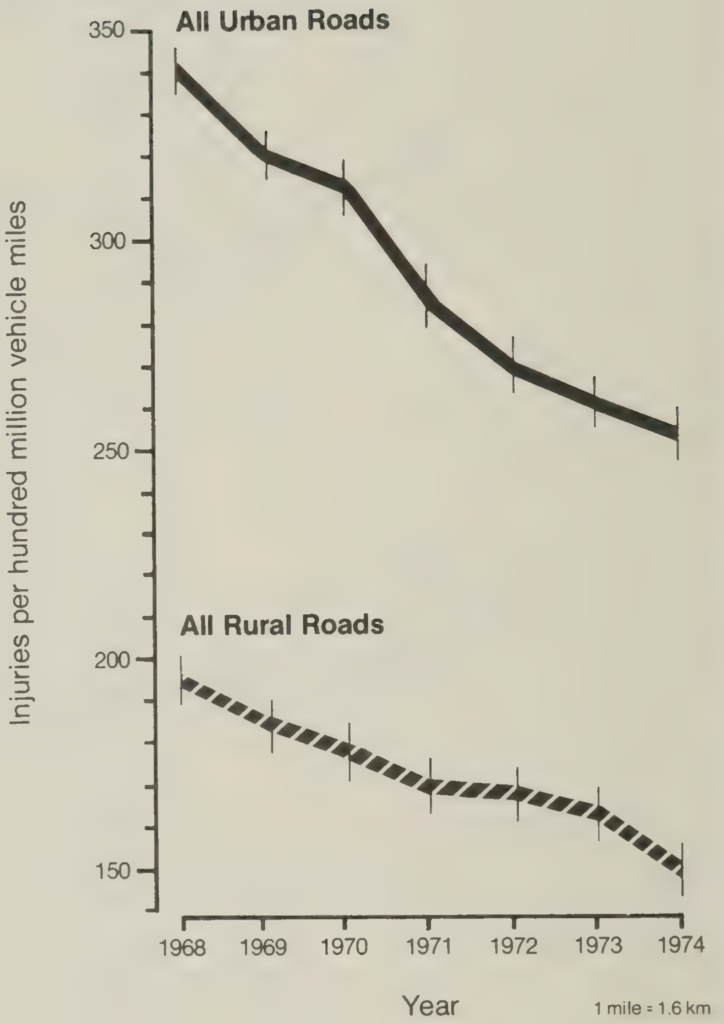


Figure 5.—U.S. injury rate—all rural and urban roads. (6)

[2]Sampled States were Arizona, California, Colorado, Georgia, Massachusetts, Michigan, Minnesota, Mississippi, Missouri, New York, North Carolina, Ohio, Pennsylvania, Texas, Virginia, Washington, and Wyoming.

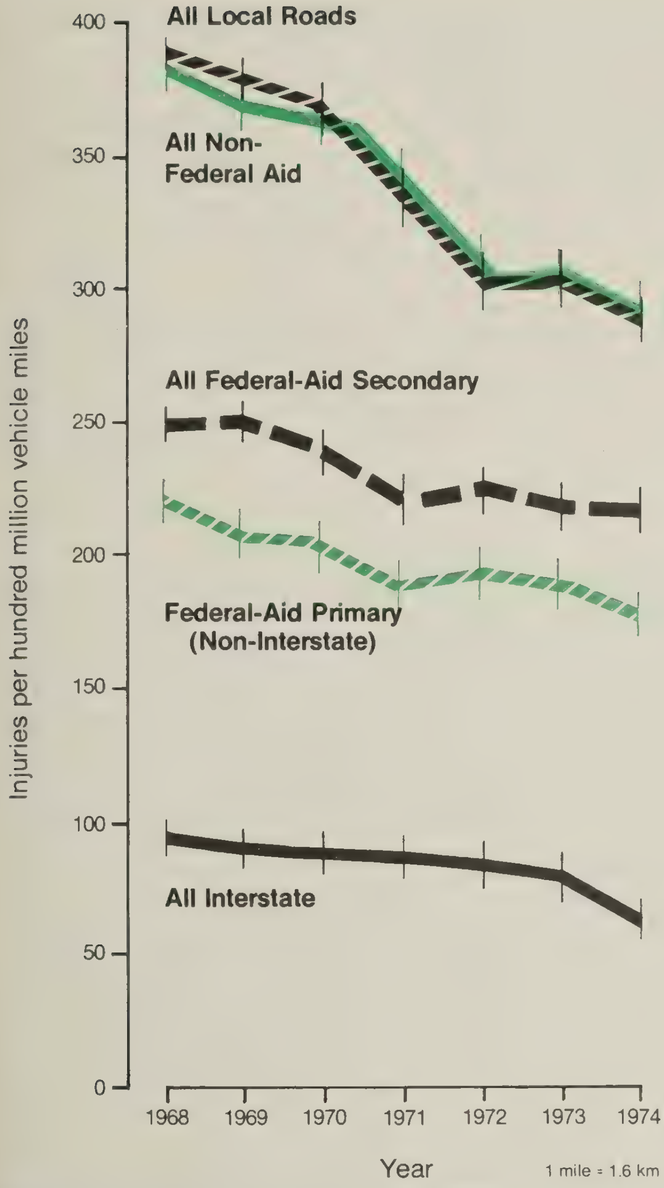


Figure 6.—U.S. injury rate by type of road system. (6)

fatalities should not have been affected by the speed limit change, these data seem to support the argument that conditions other than reduced driving speeds were influencing the fatality rate.

Time of Accident

Much speculation has been offered on the possibility of changes in the times when accidents occurred. The fuel shortage caused service stations to close on Sunday and at night during the first part of 1974. This would suggest that potential reduced travel during these time periods would perhaps lead to reduced fatalities. Additionally, the extended period of daylight savings time slightly altered the daylight hours. To examine the effects of these situations, fatal accident data for 16 States were analyzed for possible changes in the time-of-day and day-of-week fatal accidents.

In the time-of-day analysis, data for the 24-hour day were grouped in six equal 4-hour periods beginning with 10 p.m. to 2 a.m. Most sample States were found to have decreases in the number of fatal accidents during all time periods. However, the decrease in fatal accidents was less between 6 p.m. and 2 a.m. (particularly between 10 p.m. and 2 a.m.) than during other time periods. When a further comparison was made of those States in the sample which formerly had different day and night speed limits with those which did not, all of the six States with different day and night speed limits had a higher percentage of nighttime fatal accidents in 1974 than in any of the previous 3 years. However, this result was also found in 5 of the 11 States which formerly did not have different speed limits for day and night.

Sixteen States provided information on fatal accidents categorized by the day of the week. No major changes were found in the day-of-the-week distribution of accidents. Early in 1974, fatal accidents may have been reduced more on the weekends due to reduced travel, but this does not appear to be the case when considering an entire year's data.

Age of Driver

Thirteen States provided information regarding the age of drivers involved in fatal accidents. Ages were broken into four groups: 19 years or less, 20 to 34 years, 35 to 54 years, and 55 years or more. Involvement-in-accident rates for the two youngest groups exceeded the expected rate based on prior trends, but the rate for the two older groups was below expected values. Such differences could be due either to better speed compliance or reduced travel by the older drivers.

Vehicle Type in Fatal Accidents

Thirteen States also provided data regarding the type of vehicle involved in fatal accidents. Due to the wide spectrum of classification schemes, the analysis was restricted to three vehicle categories: passenger cars, all forms of trucks, and motorcycles. Analysis of the data showed that fatal accidents involved a greater proportion of trucks in 1974 than in previous years. However, the percentage of trucks involved in accidents has been rising for the past several years.

Study Summary

Employing up to 6 years of travel, speed, and traffic data from a representative sample of 17 States, the researchers made the following conclusions:

- Fatalities per hundred million vehicle miles (hundred million vehicle kilometres) have been reduced by enactment of the speed limit; the largest reduction is an over 30 percent decrease on the rural Interstate System.
- In 1974, individual statewide fatality rates were found to be consistent with nationwide data even when individual demographic, traffic, and speed limit enforcement characteristics were accounted for among the sampled States.
- Fatal accidents decreased in number during all times of the day in 1974, but generally decreased more during daylight hours than at nighttime.
- There was no significant change in the day-of-the-week distribution of fatal accidents in 1974 compared to the preceding 3 years.
- The number of drivers over 34 years of age who were involved in fatal accidents decreased during 1974 as compared to those involved in similar accidents from 1971 to 1973. This may reflect better speed compliance or reduced travel by older drivers.
- Although fatal accidents involved a greater proportion of trucks in 1974 than in previous years, this followed a trend of the past few years.
- Examined as a whole, the 1974 fatal accident record indicates a substantial change from the projected trend, and this change was definitely due in part to the national speed limit and to decreasing travel speeds.

Since the researchers did not numerically estimate the effects of the new speed limit on fatalities and injuries, the authors performed additional analysis using the same sources of national data employed by the researchers. The form of the analysis and its results are described below.

Estimated Safety Effects of Speed Reductions

Many factors have been cited which may have contributed to the 1974 nationwide drop in fatality rate. However, only two appear plausible in terms of previous research findings: reduction in mean speed and speed variance. Research by Solomon, Fee, and others has consistently shown that traveling at speeds greater than the mean travel speed increases the chance of being involved in an accident. (*8, 9*) If one is involved in an accident, the higher the speed, the more likely the accident will be fatal. Further, Heckard et al., have shown that factors related to day-of-week travel, time-of-day, the national economy, and less use of the roads by young drivers are unlikely contributors to the reduced 1974 fatality rate. (*5*) In addition, the suggested factor of reduced total travel can be at least partially discredited because the 1975 fatality rate was lower than the 1974 rate, even though total travel in 1975 exceeded 1974 travel.

The above facts, together with the findings that the largest drops in 1974 fatality rates occurred on the higher speed road systems, encouraged the authors to examine additional possible relationships between speed measures and changes in fatality and accident rates prior to and after the 55 mph (89 km/h) speed limit.

Using 1970-1975 data from "Fatal and Injury Accident Rates" (*6*) and "Traffic Speed Trends," (*7*) several regression and correlation analyses were made to determine the relationships between the following pairs of variables on a highway system basis:

- Mean travel speed and fatality rate.
- Mean travel speed and fatal accident rate.
- Mean speed change between 2 years and associated change in fatality rates.

Similar analyses were also made using injury and injury accident rates in lieu of fatality and fatal accident rates.

Six years of data were employed to (1) obtain a reasonable balance of points between higher travel speeds (pre-55 mph [89 km/h] limit) and lower travel speeds (post-55 mph [89 km/h] limit), (2) account for long term trends in rates, and (3) discount possible significant effects of vehicle and highway safety standards established before and after the speed limit change. The first two analyses (mean travel speed and fatality rate, and mean travel speed and fatal accident rate) involved only six pairs of data points, one for each year. The third analysis (mean speed change between 2 years and associated change in fatality rates) involved data for a pair of years and was run twice: once using only adjacent year pairs (5 data points) and once using all possible pairs of the 6 years of data (15 points).

Results of these regression and correlation analyses are summarized in tables 2-4. Table 2 contains the results of the correlation analysis. Analysis was made for only those highway systems where Heckard et al., found significant differences between 1973 and 1974 fatality and injury rates. (5) Very high correlation indexes (R^2) were found in the correlation analyses for all highway systems except the rural State secondary system. Therefore, results for this system do not appear in tables 3 and 4. As could be expected, there were few differences in R^2 between the fatality and fatal accident rate correlations by highway system. Somewhat surprising is the fact that the 5-point speed change correlations are higher than those for the 15-point analysis. This is probably the result of the longer term interaction effect of other factors (such as vehicle and highway safety improvement programs) when comparisons are made between data from years more than 1 year apart.

Table 2.—Correlation analysis results for nationwide fatalities and injuries

(Per 100 million vehicle miles of travel)

Highway system	Correlation indexes (R^2)			
	Mean speed		Speed change	
	Fatality rate[1]	Fatal accident rate[1]	Fatality rate	
Rural Interstate	0.82	0.87	[2]0.96	[3]0.75
Rural Federal-Aid Primary	.83	.82	.93	.77
Rural Federal-Aid Secondary	.23	.25	.17	.08
Urban Interstate	.65	.64	.82	.70
	Injury rate	Injury accident rate	Injury rate	
Rural Interstate	.93	.95	.95	.90
Rural Federal-Aid Primary	.69	.72	.68	.49
Urban Interstate	.61	.74	.95	.68

1 mile=1.6 km

[1]Based on average speeds by highway system as taken from "Traffic Speed Trends" reports (six data points; one for each year 1970-75). (7)

[2]Values in this column are based on changes in average speeds between adjacent years from 1970 to 1975 (5 data points).

[3]Values in this column are based on changes in average speeds between all years from 1970 to 1975 (15 data points).

Table 3.—Regression equations and confidence limits for determining effects of speed limit changes (1974-1973) on fatality and injury rates

Rates by system	Regression equation[1]	1974-1973 speed difference (x)	1974-1973 rate difference (per 100 million vehicle miles) (y)	
			Lower limit[2]	Upper limit[2]
		Mph		
Fatality rate:				
Rural Interstate	Y=0.130X - 0.461	-7.4	-1.20	-1.65
Rural Federal-Aid Primary	Y=0.339X - 0.546	-3.6	-1.46	-2.07
Urban Interstate	Y=0.083X - 0.390	-3.9	-0.47	-0.96
Rural Interstate	Y=0.130X - 0.461	[3]-10.0	-1.34	-2.18
Injury rate:				
Rural Interstate	Y=2.495X - 4.302	-7.4	-20.3	-25.3
Rural Federal-Aid Primary	Y=3.669X - 9.757	-3.6	-17.4	-28.9
Urban Interstate	Y=4.717X - 9.934	-3.9	-19.9	-32.8
Rural Interstate	Y=2.495X - 4.302	[3]-10.0	-24.6	-33.9

1 mph = 1.6 km/h

[1]Based on 15-point speed change regressions (see table 2).

[2]95 percent confidence limits around prediction fatality rate change for speed change between 1973 and 1974 and shown in col. 3.

[3]If speed had dropped to 55 mph (89 km/h) rather than to 57.6 mph (93 km/h).

The correlations shown in table 2 are the result of simple linear regressions run on the previously mentioned pairs of variables. The regression equations used in predicting the fatality and injury savings tabulated in table 4 are shown in table 3. Table 3 also contains the 95 percent confidence intervals around the estimated fatality and injury rate changes for the observed 1973 to 1974 highway system speed changes. All data in tables 3 and 4 reflect the 15-point speed change regression analysis since these analyses provided tighter confidence limits of the estimated fatality and injury savings. Although correlations were lower for these analyses, the additional number of data points (15 as compared to 5) produced a tighter confidence range.

The estimated fatality and injury savings shown in table 4 are the result of multiplying the fatality and injury rate confidence limit values shown in table 3 times the vehicle miles (kilometres) of travel for each highway system as taken from "Fatal and Injury Accident Rates." (6) The table shows that on a conservative basis, and with 95 percent confidence, the speed changes which occurred on the Interstate System and the Rural Federal-Aid Primary System between 1973 and 1974 led to a savings of over 4,700 fatalities and 81,000 injuries. Savings could have been as high as 7,000 lives and 123,700 injuries.

As also shown in table 4, it might be concluded that, had

Table 4.—Fatality and injury savings due to speed reductions

Highway system	1974 mean speed	Speed reduction from 1973	1974 vehicle miles	Predicted savings			
				Fatalities		Injuries	
				Low	High	Low	High
	Mph	*Mph*	*Millions*				
Rural Interstate	57.6	7.4	104,408	1,253	1,723	21,194	26,415
Rural Federal-Aid Primary	53.5	3.6	205,734	3,004	4,259	35,798	59,457
Urban Interstate	53.1	3.9	109,647	515	1,053	24,342	37,828
Total				4,772	7,035	81,334	123,700
Rural Interstate[1]		10.0	104,408	1,399	2,278	25,684	35,394

1 mph = 1.6 km/h

[1]If speed had dropped to 55 mph (89 km/h) rather than 57.6 mph (93 km/h).

the mean speed been reduced on the rural Interstate System to 55 mph (89 km/h) in 1974 rather than to 57.6 mph (93 km/h), an additional 140 to 550 fatalities and 4,500 to 9,000 injuries would have been saved. This conclusion is somewhat risky, however, as the analysis involves extrapolation beyond the observed range of speed changes. Still, it is reasonably certain that the regression function exists over a wider range of speed change values than those observed, for Solomon earlier showed that the fatality rate continues to drop at travel speeds below 55 mph (89 km/h). (*8*)

In conclusion, it can be stated that decreasing travel speed definitely causes a reduction in fatalities and injuries. The 55 mph (89 km/h) speed limit—aided by driver education, increased enforcement, and drivers' desire to save fuel—probably caused most of the observed speed reductions during 1974, which saved a minimum of 4,700 lives and 81,000 injuries. These figures could have been further increased if average speeds on the rural Interstate System had been reduced to 55 mph (89 km/h). It appears desirable to maintain such a limit, because speeds were more uniform in 1974 than before the new speed limit; and a proportion of the observed changes in fatality and injury rates between 1973 and 1974 was definitely caused by the national speed limit of 55 mph (89 km/h).

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The Legibility of

by Donald A. Gordon and Joseph A. Boyle

Introduction

Symbolic signs require less space than equivalent word message signs; one symbol can often convey an entire message. As an example, a single "Parking Prohibited" symbol may take the place of the nine-letter "No Parking" message. If symbol and letter legibilities were equal, the symbol sign might be one-ninth the size of the word sign. Or, if symbol and letter signs are of equal size, the symbol sign would be seen much farther away. Although economy of space is an important advantage of symbolic signs, very little study has been made of the distances at which highway symbols can actually be read.

The present study is concerned with the legibility of symbolic parking signs proposed by the Texas State Highway Department for use in Dallas, Tex. At the request of the Office of Traffic Operations, Federal Highway Administration (FHWA), it was agreed to test these signs. The signs tested include both symbols and word messages. The legibilities of these design elements were determined and compared with the legibility of Snellen letters.[1]

Test Site Description

Testing took place at FHWA's Fairbank Highway Research Station, McLean, Va. Markings were laid out on the floor at 1-ft (0.3 m) intervals, from 20 to 120 ft (6.1 to 36.6 m) from the test signs. For closer testing, stations ranged from 2.5 to 21.5 ft (0.8 to 6.6 m) in a 1:1.2 geometric ratio. These stations provided equal visual angle increments. The signs and acuity charts were displayed at eye height against a 30 in by 20 in (762 mm by 508 mm) white cardboard background. A uniform 36 foot-Lamberts (123.3 candela per metre^2) background luminance was provided by incandescent flood lamps on both sides of the signs.

The Dallas parking signs appear in several sizes. In urban areas, the symbol will appear on a 1-ft (0.3 m) square panel. In rural locations, sign width will range from 1 to 4 ft (0.3 to 1.2 m). The word messages are on panels attached to the signs below the symbols. Because these sizes and their corresponding legibility distances were too large for convenient laboratory testing, reduced replicas were tested. The legibility distances found in the laboratory were multiplied by scale factors to obtain the legibility distances of full-size signs.

The seven parking signs included in the study are shown in figure 1. The red

[1]The Snellen E test shows a standard eye chart E arranged on lines of diminishing size. The E faces randomly up, right, down, and left on each line.

symbol with a diagonal slash is used to show that parking is not permitted. A circular green symbol without a slash signifies that the driver may park. The written messages below the symbol state the specific conditions of parking. In addition to the standard "parking prohibited" and "parking permitted" symbols, a red circle without a slash and a prohibition symbol with a wider and longer slash were tested (fig. 2). The symbol without a slash functioned as a catch test to insure that subjects made a shape discrimination. The symbol with the wider slash was included as a possible improvement over the conventional symbol.

A Snellen "E" Eye Chart was employed for testing visual acuity. The chart is a series of letter E's facing up, down, right, or left in decreasing sizes. The subject is asked to state the direction of each letter. To include drivers with corrected vision better than 20/20, the test was administered at a 30-ft (9.1 m) distance.

Test Procedures

Thirty-four members of the Traffic Systems Division, Offices of Research and Development, FHWA, served as subjects in the experiment. They viewed the signs binocularly (with both eyes), using eye glasses if required for driving. The group may be considered as representative of the on-the-road drivers, who by law must drive with normal corrected vision.

Symbol tests

The symbol legibilities were tested first. The symbols were explained as follows:

> In this part of the study, you will tell us whether you can park or not. (Show subject the small-sized signs.) This sign with a slash means that you can't park. The sign without the slash means that you can park. We also have green signs.

The subject was taken to the 120-ft (36.6 m) position. The subject moved closer to the sign until he or she could judge whether or not parking was permitted. When a correct judgment had been made, the experimenter recorded the distance to the sign, and the subject walked back to the 120-ft (36.6 m) station. In the meantime, the second experimenter, who was at the sign, put the next symbol in place. Testing then continued.

The four symbols were presented in random order, but no symbol was presented twice in succession. Each symbol was presented 3 times, to give a total of 12 symbol presentations.

The sign messages

Legibility of messages on the bottom of each sign was tested next. The signs had a total of 17 message lines (table 1). The subject was given the following instructions:

> The next problem is to read the messages on these signs. (Show subject large-sized signs.) This is the message you must read. We will move in, one station at a time, until you can read the message. You may not be able to read the entire message at a station. Read as much as you can.

The experimenter recorded the station at which each line of the message could be completely read. The subject kept moving closer until all the lines of the message were correctly read. He then moved back to the 11th station (21.5 ft [6.5 m]) and attempted to read the message on the next sign. The procedure continued until all seven signs had been read.

The visual acuity test

After the sign message test, the visual acuity (Snellen E) test was given.

Table 1.—Message line identification

Sign	Line	Message
1	1	Parking 10 Minute Limit
2	2	No Parking Any Time
3	3	No Parking or Standing
	4	Of Vehicles
	5	Here to Corner
4	6	No Parking
	7	Out of State
	8	Vehicle Inspection
5	9	No Parking or Standing
	10	Of Vehicles
	11	6:30–9:30 A.M.
6	12	Parallel Parking Only
7	13	No Parking or Standing
	14	Of Vehicles
	15	6:30–9:30 A.M.
	16	3:30–6:30 P.M.
	17	Except Saturday and Sunday

Subjects were tested binocularly with corrected vision, that is, they used both eyes and wore glasses if necessary for driving. Acuity scores were therefore higher than those found with the usual uncorrected vision testing. Instructions for the acuity test were as follows:

> The letter E is in different positions: up, down, right, and left. Name the way the open end of the E faces. Stand so that your toe touches this line. (Put subject at 30-ft [9.1 m] distance.) Now read the top line.

A 95 percent probability level was adapted in acuity testing. To pass a line on the test, the subject was required to equal a performance attained only 1 in 20 times by chance alone. The E may appear in any one of four positions; hence, the chance of correctly guessing its position is 25 percent. The probability of correctly guessing two consecutive orientations is 1 in 16. The probabilities are therefore given by cumulating the following binomial expansion:

$$(1/4A+3/4B)^N$$

Where,
A = Success,
B = Failure, and
N = Number of letters on a line.

The expansion of the expression indicates that the subject fails at the 0.05 chance level if more than two items are wrong on a six-item line, or if more than one item is wrong on a four-item line. (There are no five-item lines on the chart.)

A person who correctly reads the 20/40 line of an acuity test and fails the next line is said to be able to read at 20 ft (6.1 m) what a normally sighted person can read at 40 ft (12.2 m). To measure the improved acuity of subjects who viewed the chart with corrected vision, the test was administered at a 30-ft (9.1 m) rather than a 20-ft (6.1 m) distance. The subject was therefore credited with better vision than that actually indicated on the eye chart. If a subject read the 20/15 line, he or she was graded 20/10. He or she was said to be able to read at 20 ft (6.1 m) what a normally sighted person would just be able to read at 10 ft (3 m). A 20/20 score was classified as 20/13. Similar corrections were made in grading the other line scores.

Determination of legibility distances

To limit the influence of atypically large or small scores, the legibility distance of a symbol was considered to be the median of the three distances at which a subject identified the symbol. Word message legibility was the furthest distance at which the message line was completely read. Each subject provided 4 symbol distance scores, 17 message line legibility distances (from the 7 signs), and an acuity score. These scores (symbols, word messages, and Snellen E) were directly compared in one phase of the analysis by being converted to equivalent visual angles. (Visual angle is the angle formed by the height of a letter or symbol as reckoned from the observer's eye.)

Figure 1.—Signs used in the study.

Results and Discussion

Legibility distances

Group legibility distance results are summarized in table 2. The table shows the mean (of individual medians) and 94th percentile distances at which each of the 4 symbols and each of the 17 word message lines were identified by the group. The 94th percentile distance is the distance at which 94 percent of the persons in the experimental group read the message or symbol. If a sign were designed for the average (mean) driver, half the driving population would be unable to read it. Hence, the equipment is usually designed for the 95th percentile person. Since 34 subjects were tested, there is no precise 95th percentile person, but performance of the third poorest subject precisely marks the 94th percentile level. Hence the 94th percentile level was used in the tables.

Table 3 shows predicted legibility distances of full-size signs. Foot square (0.3 m square) signs are linearly 6.56 times as large as the signs tested in the laboratory and can therefore be seen at 6.56 times the distance. Other scale factors were used to give the predicted visibility distances of the 2-, 3-, and 4-ft (0.6, 0.9, and 1.2 m) wide signs.

Large disparities in legibility distances appear among the elements of each sign (tables 2–4). For example, the green permissive symbol of the first, 1-ft (0.3 m) sign is read at a mean-predicted distance of 365 ft (111.3 m) (table 4). The word message "Parking 10 Minute Limit" appearing below on the same sign is read at a mean-estimated distance of 72 ft (21.9 m). The symbol

Figure 2.—Symbols tested in study. Subjects were shown entire sign but asked to identify only the symbol.

may therefore be read 5.1 times farther away than the word message, that is, 293 ft (89.3 m) farther from the sign. On the average, the symbols were identified at 6.14 times the reading distance of messages. The 94th percentile observer identified the symbols at 5.17 times the message distance. All legibility differences between word lines and their associated symbol (either no parking or parking) fall beyond the 0.01 t-test level of significance.

Word messages on the same sign may also differ in legibility. The "No Parking or Standing" message of the third sign is read at about twice the distance of "Of Vehicles" on the same sign (table 1, lines 3 and 4). The message of the larger size is read at the greater distance. Differences in legibility, associated with variation in letter size, are also found in the fourth, fifth, and seventh signs. These legibility differences are significant at the 0.01 level of significance.

Differences in legibility distances among the same (repeated) lines may seem puzzling. For example, the line "Of Vehicles" appeared on lines 4 and 10 in table 2. These lines differed in size.

The legibility differences found among parts of the same sign violate good signing practice. The driver should not have to come 212 ft (64.6 m) closer, on the average, after having identified a symbol, to read the fine print on a sign (table 4). All signs could be improved by using capital letters, shorter messages, and larger, more uniform size print, as prescribed in the Manual on Uniform Traffic Control Devices.(*1*)[2]

Effect of increasing slash size

As shown in table 2, the green permissive symbol is identified in the laboratory at a mean distance of 55.6 ft (16.9 m), the red prohibitive symbol at 33.25 ft (10.1 m). When the size of the slash is increased and protrudes beyond the circle, legibility distance more than

[2]Italic numbers in parentheses identify the references on page 73.

Table 2.—Legibility test results

Test	Distance		Visual angle	
	Group mean	94th percentile	Group mean	94th percentile
	Feet	*Feet*	*Minutes*	*Minutes*
Symbol:				
Ⓟ Red	54.1	32.0	9.9	16.3
Ⓟ̸ Red	74.6	32.0	7.0	16.3
Ⓟ Green	55.6	38.0	9.4	13.8
Ⓟ̸ Red	33.25	21.5	15.7	24.3
Mean	—	—	10.5	17.7
Message line:				
1	11.03	8.63	4.4	5.6
2	12.57	10.38	3.2	3.9
3	8.46	7.21	3.8	4.5
4	4.66	3.46	4.3	5.8
5	7.84	6.0	4.1	5.3
6	10.44	7.21	3.1	4.5
7	4.13	3.46	4.2	5.0
8	6.71	4.17	4.8	7.7
9	8.73	6.0	3.9	5.6
10	5.41	4.17	3.4	4.5
11	4.93	3.46	4.8	6.8
12	9.25	7.21	5.2	6.7
13	8.59	5.0	3.7	6.4
14	5.38	4.17	3.8	4.9
15	4.88	3.46	4.9	6.9
16	4.54	3.46	5.2	6.9
17	4.63	4.17	4.4	4.9
Mean	—	—	4.19	5.64
Acuity:	20/13	20/20	3.3	5.0

1 ft=0.305 m

Table 3.—Predicted legibility distances of Dallas Signs

Symbol or line	12 in x 12 in sign		24 in x 24 in sign		36 in x 36 in sign		48 in x 48 in sign	
	Group mean	94th percentile	Group mean	94th percentile	Group mean	94th percentile	Group mean	94th percentile
	Feet	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*
Symbol:								
Ⓟ Red	348	209	696	420	1,045	630	1,393	839
Ⓟ̸ Red	480	209	978	420	1,468	630	1,957	839
Ⓟ Green	365	249	729	498	1,094	748	1,458	997
Ⓟ̸ Red	218	141	436	282	654	423	872	564
Message line:								
1	72	57	145	113	217	170	289	226
2	82	68	165	136	247	204	330	272
3	55	47	111	95	166	142	222	189
4	31	23	61	45	92	68	122	91
5	51	39	103	79	154	118	206	157
6	68	47	137	95	205	142	274	189
7	27	23	54	45	81	68	108	91
8	44	27	88	55	132	82	176	109
9	57	39	114	79	172	118	229	157
10	35	27	71	55	106	82	142	109
11	32	23	65	45	97	68	129	91
12	61	47	121	95	182	142	243	189
13	56	33	113	66	168	98	225	131
14	35	27	71	55	106	82	141	109
15	32	23	64	45	96	68	128	91
16	30	23	59	45	89	68	119	91
17	30	27	61	55	91	82	121	109

1 ft=0.305 m
1 in=25.4 mm

doubles to 74.6 ft (22.7 m). The exaggerated slash shown in figure 2 may not be the best final choice. Dewar has shown that a slash may obscure the letter P (for parking). (2) It is clear from these results, however, that an emphasized slash would aid symbol identification. An extended slash can be designed so as not to obscure the center P of the symbol. This idea warrants further study.

Legibility of equal-sized symbols, letters, and word messages

The legibility of symbols, letters, and word messages may be directly compared in terms of threshold visual angle. As indicated in table 2, the average of the four symbol thresholds is 10.5 minutes of visual angle; average message threshold is 4.19 minutes; and the Snellen E threshold is 3.3 minutes.

The 94th percentile results show the same ordering of symbol, message, and acuity thresholds, although threshold angles are all larger. It appears that word messages and the acuity E can be recognized at two to three times the distance of symbols of equal visual angle or size. It is possible that changes in symbol design, such as lengthening the slash, could raise legibility toward the limits imposed by visual acuity, for objects of that size.

The factorial nature of sign legibility

To what extent can the legibilities of symbols and word messages be predicted from Snellen acuity scores? The question is of some practical importance; sign legibilities could be conveniently calculated if they were highly correlated with drivers' (known) acuity.

The product-moment correlations of symbols, word messages, and visual acuity scores indicate the extent to which subjects who did relatively well (or poorly) on one test also did well (or poorly) on the others. Averages of the correlations of special interest are given in table 5. The squared correlation coefficients indicate the percentage of total variance that would theoretically be shared by tests with the listed correlation.

Table 5 shows that the four symbols correlated highly with each other (average intercorrelation 0.67). The word messages show a somewhat lower intercorrelation (0.43). Symbol scores show very little relationship to word line scores (average correlation 0.13) or to acuity scores (average correlation

0.19).[3] There is no overlap between the symbol intercorrelations and the correlations of symbol scores with word line or acuity scores. The lowest symbol intercorrelation (0.39) was higher than the highest correlation of symbols with word line (0.35) or with acuity scores (0.30). Evidently, symbol identification taps different abilities than visual acuity or the reading of word messages. Consideration might possibly be given to including symbol recognition tests in the driving license acuity test.

The average correlation between consecutive word lines on the same sign is shown in table 5. If a sign had four lines, this figure was obtained by averaging the correlations between lines 1 and 2, 2 and 3, and 3 and 4. The average correlation between consecutive lines (0.60) is higher than that among all word message scores (0.43). The subject may be able to decipher a particular line if he "gets the drift" of the entire sign.

Several word lines appear repeatedly on different signs. "No Parking or Standing" appears on three signs as does "Of Vehicles." The time statement "6:30–9:30 A.M." appears on two signs.

[3] A closer relationship between acuity and symbol identification would be expected in a group tested with unaided vision. In the present study, acuity range was severely restricted by the permitted use of corrective glasses.

The average intercorrelations of these repeated lines (0.62) is higher than the average intercorrelations of word messages (0.43). Evidently, there is indication of a subject-message line interaction. If a subject finds a particular line easy on one message, he may tend to find the same (repeated) line easy on other messages.

Summary and Findings

This study is concerned with the legibility of symbolic signs proposed for use in Dallas, Tex. Thirty-four experimental subjects viewed the signs and an acuity chart, binocularly with corrected vision. Findings of the study were as follows:

- Legibility distances of different elements of the signs differed markedly. On the average, symbols were identified at over 5 times the distance of word messages. Messages with large lettering were identified at about twice the distance of messages with small lettering on the same sign. These disparities in legibility may be reduced by shortening messages, employing capital letters, and using larger, more uniform size print, as prescribed in the Manual on Uniform Traffic Control Devices. (*1*)
- Symbol threshold visual angle averaged 10.5 minutes, compared to 4.19 minutes for letter lines, and 3.3 minutes for the Snellen E Chart. At maximum seeing distance, symbols would have to be more than twice the size to be identified. These results suggest that symbol legibility can be considerably increased by improving the symbol design.
- The conventional "parking prohibited" symbol is difficult to read. An experimental symbol with a large slash which extended beyond the circle gave double the legibility distance of the conventional symbol.
- The legibilities of the four symbols intercorrelated highly, but showed low correlation with visual acuity and word message legibilities. The findings suggest that abilities other than visual acuity are involved in identifying parking symbols.
- Predicted legibility distances are given in table 3 for signs ranging from 1 to 4 ft (0.3 to 1.2 m) in size. These projections are given for the average driver and for the 94th percentile (poorly sighted) driver.

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(2) R. E. Dewar, "The Slash Obscures the Symbol on Prohibitive Traffic Signs," *Human Factors*, vol. 18, No. 3, 1976, pp. 253–259.

Table 4.—Symbol versus printed message legibility distances

Sign	Mean: Symbol legibility distance (A)	Mean: Message legibility distance (B)	A/B	A–B	94th percentile: Symbol legibility distance (C)	94th percentile: Message legibility distance (D)	C/D	C–D
	Feet	*Feet*		*Feet*	*Feet*	*Feet*		*Feet*
1	365	72	5.1	293	249	57	4.4	192
2	218	82	2.7	136	141	68	2.1	73
3	218	31	7.0	187	141	23	6.1	118
4	218	27	8.1	191	141	23	6.1	118
5	218	32	6.8	186	141	23	6.1	118
6	365	61	6.0	304	249	47	5.3	202
7	218	30	7.3	188	141	23	6.1	118
Mean	—	—	6.14	212.1	—	—	5.17	134.1

1 ft=0.305 m

Table 5.—Averages of the correlations among tests

Tests correlated	Average of correlations (r)	r^2
Symbols with other symbols	0.67	0.45
Symbols with acuity scores	0.19	0.03
Symbols with word messages	0.13	0.02
Word messages with other word messages	0.43	0.18
Word messages with acuity scores	0.48	0.23
Consecutive word lines on same sign	0.60	0.36
Word lines repeated on other signs	0.62	0.38

FHWA Skid Measurement Test Centers

by Henry C. Huckins

Introduction

In 1970 the Federal Highway Administration (FHWA) initiated a program to improve and standardize the measurement of pavement skid resistance on the nation's highways. (*1*)[1] [2] The achievement of the program objectives aided State highway departments and Federal agencies in conducting pavement skid inventories that are needed for highway safety programs.

As the national program to standardize skid resistance measurement progressed, it provided for the establishment of national area reference skid measurement systems, the development of evaluation and calibration procedures, and the development of specifications and field testing procedure guidelines for inventory skid measurement.

The program also provided for the establishment of three field centers to test, evaluate, and calibrate the skid measurement systems operated by State highway departments and other governmental agencies. After a short time, it became apparent that only two centers were necessary to service the existing population of skid resistance measurement systems in the nation.

Area Test Centers

The Central Field Test and Evaluation Center (CFTEC) is located at the Research Annex of the Texas Transportation Institute, College Station, Tex.; and the Eastern Field Test and Evaluation Center (EFTEC) is located at the Ohio Transportation Research Center, East Liberty, Ohio. The CFTEC is operated by the Texas Transportation Institute and the EFTEC by Ohio State University. The centers have conducted routine tests, evaluations, and calibrations on over 100 inventory skid measurement systems. There are presently 20 additional requests for calibration schedules.

The centers have the necessary facilities and personnel to perform the static and dynamic tests, evaluations, and calibrations of the inventory skid measurement system and its subsystems.

Each center has an area reference skid measurement system (fig. 1) which has been evaluated and calibrated against the National Reference System (NRS) of the National Bureau of Standards. A relationship between the measurements of each State's inventory skid system and the NRS is established through these area reference systems (ARS). In this manner standardization of skid measurements is achieved on a national basis.

As part of the facilities, each center has five Primary Reference Surfaces (PRS), 15 ft (4.6 m) wide and 520 ft (158.5 m) long to serve as the reference material from which the accepted reference level of skid resistance values is determined during the evaluation and calibration process. These surfaces were constructed with selected materials in order to produce a resulting range of skid numbers (SN) from 20 to 80. All of the surfaces were placed by one contractor.[3] The only traffic allowed on the PRS is the area reference system

[1] Italic numbers in parentheses identify the references on page 79.

[2] "Development of Skid Trailer Specifications and Regional Field Test and Evaluation Centers," by L. M. Cook. Paper presented at the Conference on Skid Resistant Surface Courses, Baton Rouge, La., October 1972.

[3] "Construction of Primary Reference Surfaces for FHWA Field Test Centers—Final Report," by Adhesive Engineering Company. Unpublished report, Oct. 15, 1975.

and the inventory skid systems being calibrated. Much care is exercised in protecting the integrity of the surfaces. Figure 2 shows the PRS at the EFTEC.

Equipment at the centers includes items such as air bearing force plates to calibrate the skid measuring transducers and balance the vertical wheel load; static water measurement equipment to measure the flow rate and distribution of the water emerging from the nozzle; precision air pressure gages to calibrate the gages used by the inventory crew; and a variety of electronic test instruments to service equipment such as recorders, velocity generator-indicators, and system electrical circuits. Many replacement components common to all skid systems are kept on the shelf and readily available for fast skid system service.

Center personnel include experts in electronics, mechanics, data analysis, and report writing. These people are well acquainted with the process of evaluation and calibration of inventory skid systems and the transmission of valuable information to the visiting crews.

Evaluation and Calibration Procedures

The calibration process begins with a request by a State through the appropriate FHWA Division Office to schedule its skid measurement system into a center. After the center receives the request, the director contacts the State to arrange a convenient time for system calibration. Generally, 2 weeks are necessary for a system to be calibrated. The center sends the State a brochure which tells what to expect from a visit to a center and lists necessary items to bring.

On the first day at a center, the inventory crew is given an orientation by the director who outlines the calibration procedures and the safety regulations of the center and states what is expected of the crew.

Pictures are taken of the skid measurement system and instrumentation. The system's components (for example, truck model and trailer manufacturer) are documented and certain important measurements are made. This documentation is filed to be used as a basis for comparison on the next calibration visit.

Following the documentation, the first correlation ("as arrived") measurements between the State skid system and the ARS are conducted on the PRS. Each skid system completes 108 runs—36 runs each at speeds of 20, 40, and 60 mph (32, 64, and 97 km/h). The data from these tests are reduced to an average SN for each pavement surface at the three speeds (table 1), and standard deviations, which represent the system's precision, are statistically determined for each speed set (table 2). Linear regression equations are also derived from the data. These equations statistically relate measurements of the State skid system data to those that would have been determined by the National Reference Standard had it been there at the same time. In this process, the State skid data is standardized on a national basis. A typical graphic plot of these linear regression equations is shown in figure 3.

The results of the first correlation may be used to standardize the State system's previous skid number data. This data now can be related to the National Reference Standard

Figure 1.—Area reference skid measurement system in use at the centers.

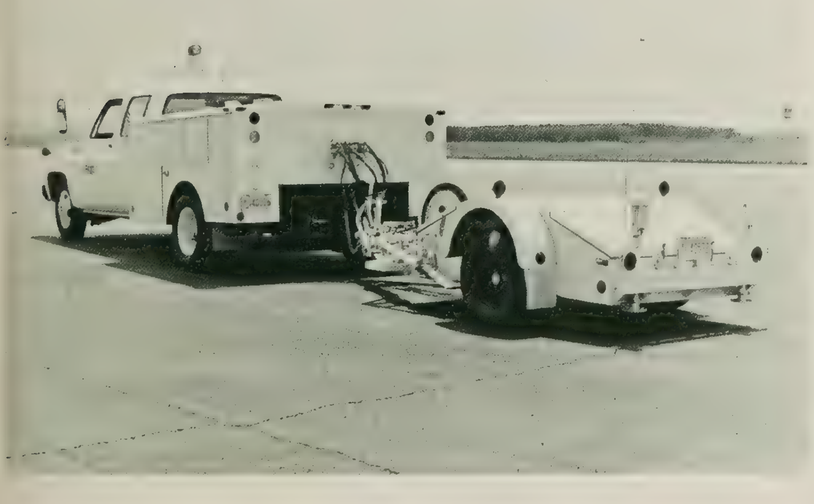

Figure 2.—The primary reference surfaces.

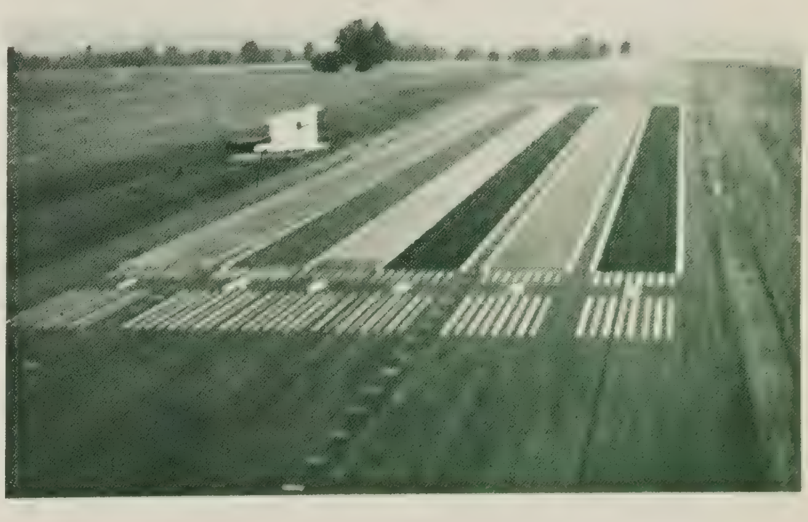

Table 1.—Results of first dynamic correlation

Speed	PRS	ARS skid number	State skid number
Mph			
20	1	45.9	43.4
	2	61.1	57.5
	3	64.2	61.0
40	1	37.2	35.4
	2	54.1	53.8
	3	53.1	54.1
60	1	28.7	3[illegible].9
	2	52.8	53.9
	3	48.1	53.1

1 mph=1.6 km/h

through the center ARS. In the same manner, results of a second correlation ("departing") can be used to standardize skid measurement data obtained after leaving a center. Figures 4 and 5 show a second correlation example.

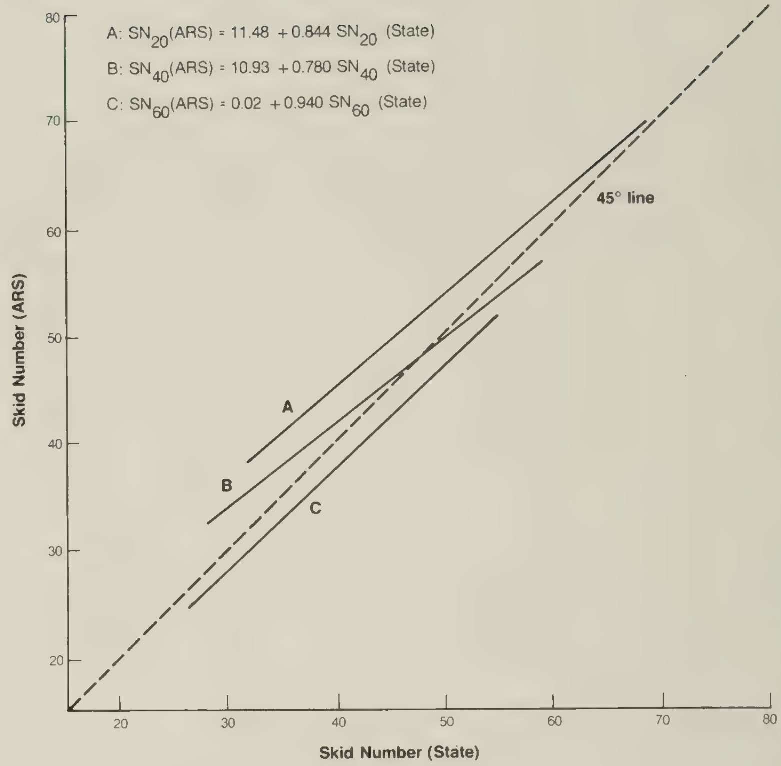

Figure 3.—First skid number correlation (State system).

Since the first correlation is performed on the skid system upon arrival at the center and the second correlation is performed just prior to departure, a comparison of the two results will reflect the effects of the test, evaluation, and calibration process which takes place in the intervening time period. The objective of these tests is to bring the subsystems into compliance as closely as possible with the standard specifications described in American Society for Testing Materials (ASTM) Specification E 274-70. (*2*)

Major subsystems that are tested include the force measuring transducer and its calibration circuitry, water distribution, velocity measuring instrumentation, tire air pressure gage, force plate (if used), and the very important procedures used by the driver and/or operator to obtain the skid measurement and to read the charts.

At the centers, an air bearing force plate which has been calibrated by the National Bureau of Standards is used to determine the vertical load and accuracy of the force calibration signal on the test wheel (fig. 6). Adjustment of the hitch load and the vertical load on the wheels is made to conform with ASTM E 274-70. The transducer output versus force is plotted to measure the linearity and hysteresis of the transducer. If necessary, the system's calibration signal is adjusted during these tests.

The water flow rate is also measured at 20, 40, and 60 mph (32, 64, and 97 km/h). Pumps, drives, and nozzles are modified or regulated for a water flow rate and distribution ahead of the test tire to comply as closely as possible with ASTM E 274-70.

Tests are conducted on the speed measuring subsystem to determine its accuracy. An adjustment and/or a calibration is made to compensate for an erroneous speed indication. Tire air pressure gages are calibrated against the center's reference. Also included in the tests is the data recording subsystem. Any electronic and mechanical malfunctions are corrected.

The driver and operator (crew members) are important to the skid measurement operation. During the first correlation runs, the crew members are instructed to take skid measurements in the same manner as they would on their home State highways. If they also read and reduce analog recordings to skid numbers, they are instructed to do so for the first correlation. However, before the second correlation

Table 2.—Skid number of Primary Reference Surface 2 at 40 mph

Point	Skid number	Deviation (d_i)	$(d_i)^2$
1	53	0.1	0.01
2	53	0.0	0.0
3	52	-0.9	0.81
4	53	0.2	0.04
5	51	-1.7	2.89
6	53	0.4	0.16
7	51	-1.5	2.25
8	50	-2.4	5.76
9	51	-1.3	1.69
10	54	1.8	3.24
11	53	0.9	0.81
12	51	-1.0	1.00
13	53	1.1	1.2
14	51	-0.8	0.64
15	56	4.3	18.49
16	51	-0.6	0.36
17	52	0.5	0.25
18	53	1.6	2.56
19	52	0.7	0.49
20	54	2.8	7.84
		4.2	50.5

$$SD^2 = \frac{\Sigma d_i^2 - \frac{(\Sigma d_i)^2}{N}}{N-1}$$

$$SD^2 = \frac{50.5 - \frac{(4.2)^2}{20}}{19}$$

$$SD^2 = 2.61$$

1 mph = 1.6 km/h

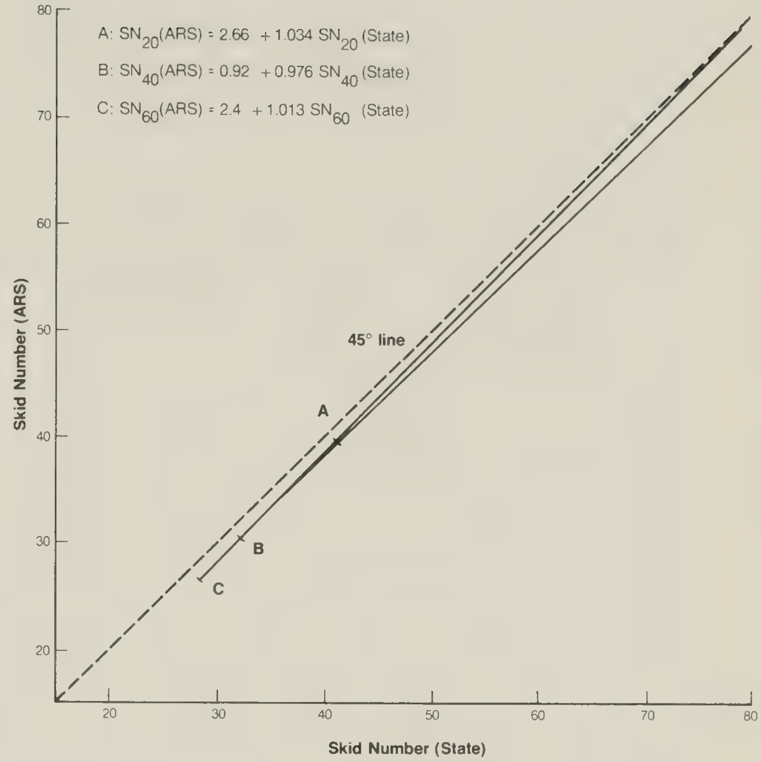

Figure 4.—Second skid number correlation (State system).

runs, the crew is checked for its ability to maintain uniform speed through the skid test, to place the test tire within specific lanes on the pavement, and to start the test at a particular point. The crew's capabilities are appraised, and instructions are given for improvement.

Results and Benefits From the Center

Results and benefits obtained through the operation of the centers are twofold. First, FHWA and the State highway agencies can compare skid measurement data taken by the various skid measuring systems operated by the States. This comparison is achieved through standardization of measurement data—a primary objective of the FHWA Skid Accident Reduction Program. Highway maintenance priority listing can be facilitated by the data standardization.

Second, and very important, are the benefits that the States receive from visiting a center. Virtually all States have attested to the knowledge gained (aside from calibration) about their skid system, although they may have had their system in operation for years. Many skid systems coming to the centers have experienced component wear or change so slowly that the operator may not have noticed the change.

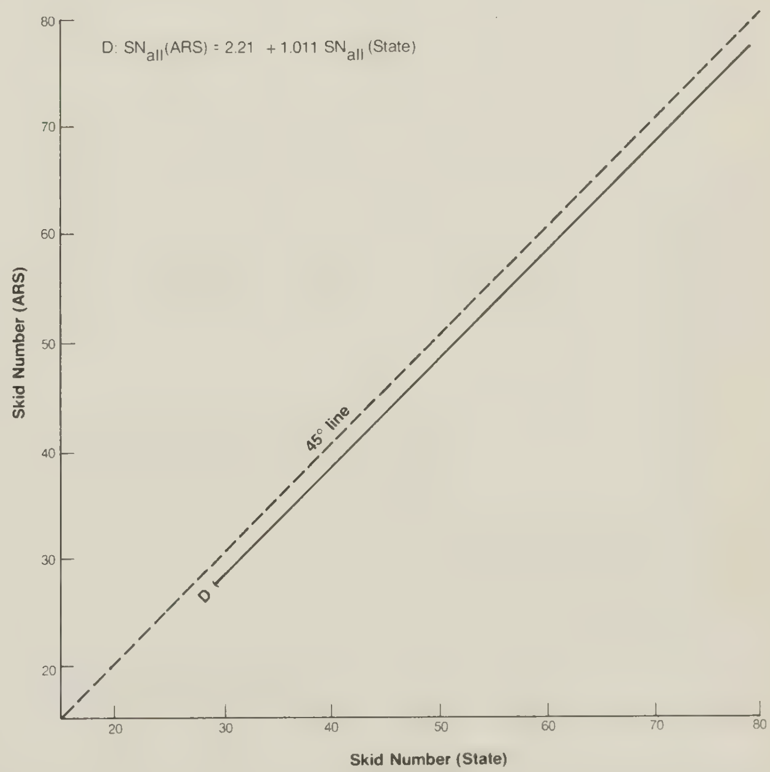

Figure 5.—Second skid number correlation—all speeds combined.

There have been instances where a newly delivered skid measurement system has had malfunctioning subsystems; and because the crew was not totally acquainted with the system, they were not aware of the problems. However, when the system is taken to a center, testing and observation by center personnel will reveal worn and/or malfunctioning components. Major repairs and replacements were made on 56 skid systems to the water system, brake system, nozzle, suspension, wheel load, or tow vehicle. Many minor repairs also were made, such as replacing a light bulb or tow vehicle ignition parts, correcting a bad wire connection, or repairing or adjusting the electronic equipment.

Another benefit of the evaluation and calibration process is that it determines the skid measurement correspondence of the State system to that of the ARS at all speeds (fig. 5). This is evident by comparing the first and second correlation equations. It is highly improbable that two systems will ever be on a one-to-one correspondence, but it is desirable to bring each equation as close and parallel as possible to the 45° line of equality between the ARS and State system. This closer correspondence of the curves will permit a single equation to be used for the standardization of the data, with the least possible variation for all speeds.

As stated before, the change in the bias and slopes between the first and second correlation equations is manifested by the evaluation and calibration process. During the process, adjustments and modifications are performed on the State skid measurement system to achieve compliance with the tolerances of ASTM E 274-70. Because the ARS has been designed to comply with the ASTM standard and is frequently monitored for compliance, the tendency of the correlation curves to approach equality is expected. This result has been observed in most of the calibrations done at the centers (figs. 3, 4, and 5).

The driver gains a benefit from the center visit because, invariably, it has been found that the driver will position the skid test tire on the same place on the pavement (precision), although not necessarily in the center of the specified lane (accuracy). Since the SN can vary widely in transversing the wheelpath on a highway, it is imperative to know where the skid test took place. To assist the driver in aiming his tow vehicle, a sighting arrangement developed at the centers is placed on the windshield and outer hood of the vehicle to keep the skid tester wheel in the proper place (fig. 7).

One of the major causes for skid measurement variability is the water distribution subsystem composed of the storage tank, water pump (one system uses air-pressurized water),

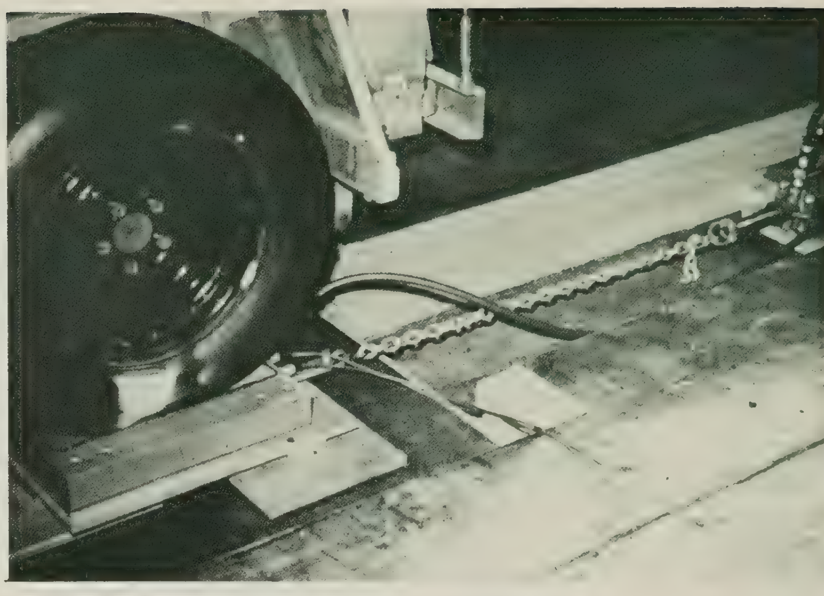

Figure 6.—Calibrated force-load plate on air bearings.

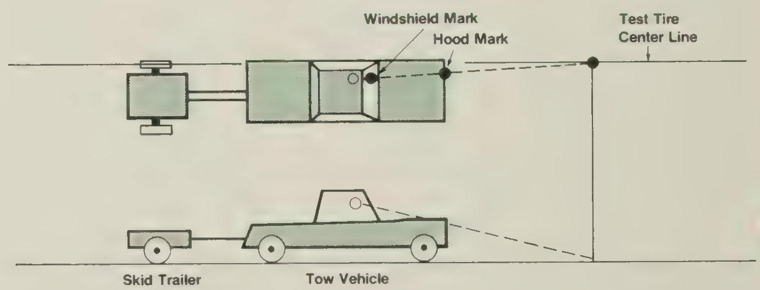

Figure 7.—Lateral positioning driver aid.

valves, and nozzle. To obtain standardization in water distribution, it is suggested that the State's water nozzle be replaced by a nozzle designed at Penn State University. (3) However, since this is a divergent flow nozzle, it is difficult to attain the water flow specified in ASTM E 274-70 for all speeds. In most cases, the flow rate is adjusted to fulfill only the specification at 40 mph (64 km/h) (fig. 8).

In the calibration report, the State receives a description of a procedure for estimating system variability at its home base. In the procedure, a suitable nearby State highway pavement section is selected based on uniformity of skid resistance. For an example, 20 duplicate tests can be conducted each time the variability of the system is to be determined. The tests are run in the same spot on the pavement with the same test tire, and the time between tests is minimized so all tests are conducted under approxi-

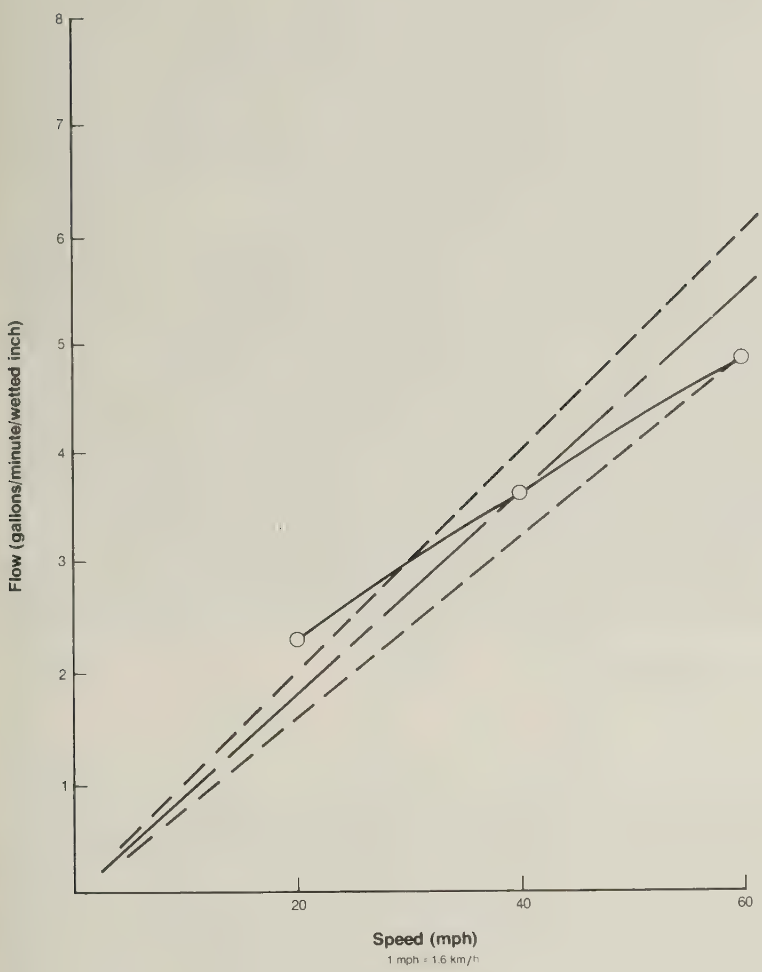

Figure 8.—Water flow rate.

mately the same environmental conditions. A standard deviation of the SN is computed for the 20 runs, and the square of this deviation is compared to the squared SN deviation at the corresponding speed range taken at the center. If the ratio of the larger variance to the smaller variance does not exceed 2.15, it can be said with 95 percent confidence that the skid system has not changed significantly since the center visit.

Changes in SN on the home test section will occur as the surfaces wear under traffic and from seasonal variations. However, these changes should occur in an orderly fashion, and any abrupt change in SN would indicate erratic performance of the skid measurement system. Test data taken on these home test sections can be forwarded to a center for evaluation.

Conclusions

Since the centers became operational, over 100 skid measurement systems have been tested, evaluated, and calibrated. Each of the 44 State highway departments owning a skid system has had at least one schedule into a center. Other governmental agencies also have taken advantage of the services provided by the centers.

Commendations received by the centers from visiting States reveal that personnel benefits are just as great as the technical calibration benefits. This is a result of the knowledge gained about the skid systems. Many of the systems have been purchased and put into operation with crews who have been given little technical knowledge about the instrumentation or how to recognize a malfunction. The crews were very appreciative of the instruction obtained from visits to the centers.

Another benefit of the operation of the centers is the correction of many skid system malfunctions and worn components. It can be assumed that most of the State skid measurement systems in operation today are in sound electrical and mechanical condition.

The objective of standardizing skid measurement data on a national basis is now a reality through combining the evaluation and calibration process with the establishment of a National Skid Reference Standard.

Many States are taking their skid systems to a center for a second evaluation and calibration, particularly when the system has had a year or more of inventory operation. FHWA will use the data from these return schedules for a study that will attempt to determine the recalibration frequency required to keep the accuracy and precision of the skid system's measurements at a specified level.

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Simulation of Traffic in Street Networks

by David Gibson and Paul Ross

Frame from UTCS-1/NETSIM model validation film taken in Washington, D.C.

Vehicular traffic simulation is becoming an important tool for traffic engineers and transportation planners, for several reasons. Actual traffic data are very expensive to obtain and are often unreliable; many hours of work are required to collect traffic counts. The periods of interest are generally short—about one-half hour—so that, because of the random nature of the phenomena, many repeated days of data are required. However, a well-calibrated simulation model yields an enormous amount of data quickly and accurately. For example, some data, such as delays, are nearly impossible to obtain in real life, whereas a simulation model can give consistent values even when they are not accurate. Installing even the simplest changes in signal systems usually requires considerable manpower, and several months of real traffic observation are necessary to evaluate the effects of these changes. But traffic simulation experiments are comparatively cheap and quick and avoid the liability associated with "playing around" with traffic signals. They can reproduce the exact conditions repeatedly so that, even if the model is not accurate in detail, the effects of small changes on the traffic system immediately become apparent.

There are three classes of traffic simulation models: single road, single intersection, and network models. Single road simulation models generally represent freeways or rural roads. Freeway models to study merging, ramp metering, the effects of traffic composition, and incident detection phenomena are becoming common. A review of these freeway models has recently appeared. (*1*)[1] Rural road models are more rare; they reproduce such phenomena as sight distance restrictions, rolling terrain, and passing opportunities. Single intersection models have generally been built for a specific purpose and are not generally available. Perhaps Webster's is the best known example of a single intersection model which he used to study the effects of isolated traffic control signals.

[1]Italic numbers in parentheses identify the references on pages 89-90.

Network models are more complex. Some represent surface streets only; others can include freeway networks. These models are very useful in testing signal control strategies, traffic diversion strategies, attempts to add or delete streets from the network, and similar tactics. These models are becoming extremely important because of the current emphasis on coordinating networks to produce systemwide optimization. Computer signal control strategies are often pretested with network simulation models.

Traffic simulation models are generally limited to evaluating the various parameters of traffic flow. Accidents and safety considerations cannot yet be examined with these models—except perhaps with a few of the specialized single intersection models.

This article discusses currently available network simulations and some earlier models that are now obsolete but are of historical interest. Also, three signal optimization programs that include network simulation models are briefly described.

History

Proposals for traffic simulation models date from at least 1951. (*2*) The earliest documented model that we have been able to identify was constructed on an analog computer in 1955. The British Road Research Laboratory model (1961) is of unusual interest. (*3*) It used a digital computer with individual memory locations representing short sections of the road; a zero meant that the corresponding section of road was empty. "Ones" were manipulated through the memory to represent vehicular progression.

The earliest traffic simulations were used only to examine the general quality of flow of traffic. For example, the Stark model produced a motion picture which displayed the traffic behavior in the simple network that is built into the model. (*4*) The TRANS model in the mid-1960's was the first network simulation model to consider details of signal settings. (*5, 6, 7, 8, 9*) Wider use of the computer has brought about proposals to control traffic with computer systems; the UTCS–1 simulation program was developed to analyze these methods of control. (*10, 11*) The current trend toward system optimization is leading to optimization programs, some of which include simulation as a component.

Current Practice

Modern traffic simulation programs use digital computers and time-oriented bookkeeping. The computer language is usually FORTRAN; the event-oriented bookkeeping associated with most simulation language seems inappropriate as vehicles respond continuously to other vehicles rather than to just the isolated events of traffic signal changes. Some models treat the traffic stream as a continuum. These models generally conceptualize the traffic stream as a fluid of some kind. Such models are called *macroscopic* in this article. In macroscopic models, individual vehicles are not identified; indeed, a link may contain fractional vehicles. Macroscopic models generally use computer memory and execute rapidly because they do not keep track of each vehicle. However, they are not able to represent the traffic stream in the detail that many traffic engineers need.

Platoon models are a half step toward detailed realism and are so called because vehicles are grouped into platoons whose location, speed, and acceleration are tracked by the program. Platoon speed is usually a function only of the general density of vehicles in the platoon, thus avoiding any complicated *car following* calculations.

Microscopic simulation models represent the ultimate in detailed treatment. Each vehicle is identified and its position, speed, and acceleration are kept in memory.

Two separate areas are treated in microscopic models: streets and intersections. On the streets, the behavior of the vehicles is usually approximated by controlling the vehicle's speed by the motion of the preceding vehicle (car following). Other details of street traffic such as lane changing may be represented. Buses and trucks may be handled separately from other types of traffic. Intersection behavior is considerably more complex and difficult to model; pedestrian interference, turning radii, and collision avoidance must be considered. However, these problems are usually treated in a superficial manner—for example, at most a random delay is assigned for pedestrian interference. Since the primary function of most network models is to evaluate signal control strategies, traffic signals are usually modeled in considerable detail.

Specific Models

In the discussion of specific simulation models in this article, the following assumptions are implied unless otherwise stated:

- The models are fully *microscopic*, that is, the model has a specific location, speed, and acceleration for each individual vehicle. Moreover, the vehicles engage in car following, that is, the speed of a vehicle is determined in some reasonable, predetermined manner by the speed of any vehicle close in front of it. If there is no vehicle close in front of the subject vehicle, it travels at its *desired* speed. Vehicles obey traffic signals and signs.
- All variables in the system (vehicle locations, speeds, accelerations, and signal indications) are updated once each *time step*. The time step is a user specified constant—often 1 second. The computer program is written in FORTRAN.
- The simulation starts collecting data only after some initialization period. The initialization period begins with the system empty and ends when some test for equilibrium is satisfied; in no case is this test very rigorous.
- Vehicle routes are determined from *origin-destination* tables (O–D) or the probability of going through or turning left/right at the end of each link (turning movements). O–D input requires the program to compute the path of each vehicle. There is no clear-cut consensus as to how real drivers do this; moreover, O–D data are expensive to obtain. Turning movement percentages are readily available; they

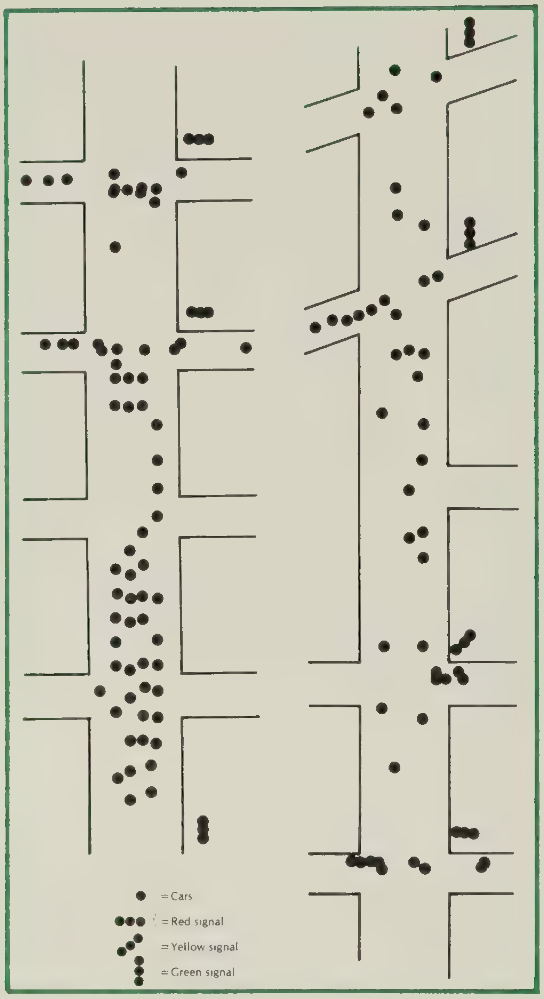
are a standard part of traffic counts and most cities have them on file. However, turning movements produce vehicles which wander aimlessly within the network.

Outmoded Models

These outmoded models are of historical interest as they show the evolution of traffic simulation models. The earlier models used very basic car following rules to trace vehicles through networks of from 1 to 10 intersections without provision for different vehicle types or control methods. Successive models improved the realism of the traffic behavior, the complexity of the network being modeled, and the types of control applied in the network.

1. The Discrete and Continuous Variable Simulation Model developed by Trautman and Davis was perhaps the first "realistic" traffic simulation model. (*12, 13*) It considered a signalized intersection with one lane in each direction and pedestrians able to delay right- and left-turning traffic. Validation was primarily by "reasonableness." This model was based on an analog rather than a digital computer and therefore each new intersection required its own model. In analog models, the model input preparation and building tasks are synonymous with circuits and gates representing distance and delays. The ratio of real time to computer time was approximately 1:1 for the single intersection and small network problems examined.

2. The Stark/NBS model was produced by Martin Stark of the National Bureau of Standards for the Bureau of Public Roads using the IBM 704 computer. (*4*)



Frame from STARK simulation of 13th Street, NW., Washington, D.C.

Realism in this model was achieved through the use of fairly detailed but deterministic traffic behavior rules including such factors as lane changing logic, gap acceptance, right-of-way, and car following.

Although the model includes traffic signals, stop signs, one- and two-way streets, and oblique intersections, they are unfortunately contained within the program structure rather than specified by input cards. As a result, this model is as rigid as the earlier Discrete and Continuous Variable Model.

Although the model was not validated, it was one of the first to yield a variety of statistics such as a frequency distribution of the running time of vehicles through the network. Perhaps the most interesting output was a CRT-based movie of the simulation which gives the model the appearance of realism.

The model was written in Assembly language for the IBM 704. Its execution speed yielded a real to simulation time ratio of 15 to 1 on a network of 9 linear blocks on 13th Street, N.W., in Washington, D.C.

3. A simulation model was developed by the Road Research Laboratory in the United Kingdom. (*3*) Realism is limited as the model does not distinguish between different types of vehicles and simulates only signalized intersections of three or four legs. No indication is given as to whether the model has been documented or validated. Outputs produced include total time simulated, volumes and delays by link, average delays and average queues by link, and maximum queue by link. The computer program was written for the Ferranti Pegasus computer. The maximum network size is 30 intersections, 80 links, and 20 peripheral arms. The program executes at approximately 18/n times real time where n is the number of intersections. The program was used in central London to examine proposed changes in traffic signal patterns.

4. The Australian or PAK-POY model (*14*) is a general purpose microscopic model which was written to analyze traffic signal controllers and intersection capacity. PAK-POY's results were only moderately realistic, because they

involved only a single intersection. Validation was limited and consisted primarily of comparison of model results with before and after field studies of intersections. Information on documentation was unavailable at the time of this writing. The primary outputs of the model are delay, queue, and degree of saturation. The model was written for the IBM 7090 and had a real time to computer time ratio of 2000:1.

5. Vehicle Traffic Simulator (VTS) was constructed by A. H. Blum with intersection modules. (*15, 16*) These are assembled to form a network, thus making the network geometry highly flexible. The realism of VTS is good. The model simulates turn percentages, traffic signals, special sources, and sinks of traffic (for example, parking lots and alleys). However, fixed time signals control intersections and the car following logic is "change lane or assume speed of vehicle in front." Vehicles are not identified by type but may be approximated through the assignment of vehicle characteristics.

No validation has been performed on this model and documentation is limited. (*16*) Many statistical outputs are available including queues, delays, lane utilization, and travel times. Some difficulty may be experienced in interpreting the large number of statistics automatically generated by the GPSS language in which the model is written. Since the model is in GPSS, it will run only on IBM computers. The model was originally written for the IBM 7090 and had a ratio of real time to computer time of 1:1 for a 10-intersection network. The maximum network size was approximately 50 intersections. This model was the predecessor of the VETRAS model, described later.

6. The TRANS Model was the first network simulation model to achieve wide usage and validation. At least four versions were produced from 1962 to 1968. (*5, 6, 7, 8, 9*) Its realism and flexibility were revolutionary for its time. Although TRANS is now obsolete, it has led to several other models.

Realism in the TRANS model was moderate. Cars were grouped into short platoons and could switch lanes instantaneously if they were queued behind left-turning vehicles. Behavior of vehicles at signals was extraordinarily good. When the signal turned green, the vehicles discharged according to a detailed rule that was variable from intersection to intersection to reflect different properties of grade and intersection geometry. Version IV of TRANS uses a user specified update period which cannot be less than 2 seconds.

The principal drawback is that all intersections must be controlled by fixed time signals, and unsignalized minor streets cannot be represented in the network.

The TRANS model was the first simulation model to be widely validated. Validation runs were made in Los Angeles, Detroit, and Washington, D.C. Data preparation, although clearly explained, is quite tedious because large numbers of detailed inputs are required. Output gave the usual traffic parameters, both link-by-link and networkwide.

TRANS was written in SAP/FAP assembly language for the IBM series 7090 computers. It achieved excellent speed of execution because of platooning on the links. For a test network of 35 links and 9 signals, a time compression of 20 to 1 was achieved.

7. The Hartley model is a very basic model using a hardware digital traffic simulator. Thus, the model is relatively easy to run but has limited realism. Only fixed time signals may be simulated. Program variables consist of network geometry, offset, degree of saturation extraction rate, and input flow rate. Validation appears to have been by sensitivity analysis and reasonableness, and documentation is limited. (*17, 18*)[2] A diffuser is used to link intersections and replicate platoon behavior, and a random pulse generator creates input traffic. Since a hardware traffic simulator was used, the speed and size of the network model is not directly comparable to other traffic simulation models and it is only moderately flexible.

8. The Birmingham model was written by C. E. Storey of the University of Birmingham, England. (*19*) The program is written in EGTRAN 3 which is a variation of FORTRAN IV. The model is fairly flexible and handles four categories of vehicles. However, car following logic is limited and vehicles are assigned to their lanes based on their turning movement at the subsequent intersection. An unusual feature of the model is the assignment of vehicle routes through the network when the vehicle is generated in order to avoid looping of vehicles. The model can simulate both fixed time and vehicle actuated signals. However, no mention is made of volume density or other controllers. Output is limited to printouts of vehicle location within the network, and both journey times and delays for each boundary category

[2] "Logical Design and Associated Computer Studies for the Hardware Simulation of Traffic Dispersion," by I. S. Saleeb, Ph. D. thesis, Manchester University, Manchester, England, 1967.

and route. The model has not been validated. However, sensitivity testing has been conducted and the model produces *reasonable* variations in output values based on input values. The model is not exceptionally fast as it has a computer time to real time ratio of 40 to 1 for a single intersection and 4 to 1 for a three-intersection network. The model has been run on the ICL computer and has a maximum network size of 99 intersections and 99 input links. The program occupies 5K of core.

Frame from Aerospace freeway simulation movie.

9. The DYNET simulation model was developed by General Applied Science Laboratories in 1969. (*20*) It was based on the TRANS model and uses many of the features of that model, but with many specific improvements. In particular, an effort was made to make the model fully microscopic and to simplify the input requirements.

The realism of the DYNET model is excellent with trucks and automobiles represented separately. The car following rule used is not clear from the only available report. A vehicle is assigned a lane when it first enters a link according to the turn the vehicle expects to make as it exits from that link. A vehicle can change lanes when it enters a queue if the queues in the adjoining lane are shorter and the turning movement will still be permissible, but only if an acceptable gap is available in the adjoining lane. Left-turning vehicles examine the oncoming traffic for an acceptable gap and remain in queue until one is found. Generally vehicles behave at stop signs and signals very much as in the TRANS model.

The DYNET model has excellent flexibility. Several of the drawbacks of the TRANS model have been eliminated. Almost any type of intersection can be represented; stop signs, yield signs, and actuated signals are allowed.

Due to the short active life of this model only one validation test has been made—in Los Angeles, Calif. Input preparation is particularly simple in this model as a result of a successful effort to reduce the input requirements from the TRANS model. About one-half as much input data are required for DYNET as are required for TRANS.

Output by the DYNET model is full and detailed and gives most characteristics that would be of interest. DYNET was programed in FORTRAN for the IBM 360 computer. Due to its detailed microscopic character and the shorter time step generally used (1 second), the DYNET model actually runs slower than the TRANS model. If time steps are the same in both models, speed of execution is comparable.

10. Sakai and Nagao developed a simple platoon macroscopic model. (*21*) In this model, roadways are partitioned into sections about 50 metres long. The computer keeps track of the number of vehicles in any 50-metre segment, and the average speed of these vehicles is determined by the number of vehicles in the segment. Buses and trucks are represented by passenger car equivalents. At the intersection, cars are delayed by a fixed amount of time to represent pedestrian interference. The time step for this model is 4 seconds.

The model was tested on a small network with six intersections in the central business district of Kyoto, Japan. This simulation was done with a minicomputer with 16K of core storage. Language, execution speed, and exact machine type are not known.

11. VETRAS (*22*) builds on Blum's earlier VTS model. Thus, VETRAS is somewhat more refined and realistic than models which were built from scratch. The logic includes provision for right turn on red. Car following logic is "switch lanes or assume speed of leader car." The model is also written in GPSS. Its modular structure and input format make it easy to set up a variety of networks within a restriction to three- or four-legged intersections. A variety of vehicle characteristics can be generated, but trucks and buses are not represented. Documentation for VETRAS is not extensive but is clear. Validation was limited to sensitivity testing.

The model's output provides statistics on virtually all queues, storages, and entities in the simulation as well as any additional user desired statistics. However, these voluminous statistics require thoughtful interpretation. The model is written in IBM's GPSS language. Use is, therefore, restricted to facilities having IBM computers with GPSS. Model size is restricted by the GPSS package at an individual facility.

Current Models

1. The Aerospace Corporation Model VPT (Vehicle Performance in Traffic) is an exceptionally detailed, totally microscopic network model. (*23, 24, 25*) It is a linking of two models known as FREEWAY and VPSST (Vehicle Performance in Surface Street Traffic).

Automobiles, trucks, and buses are generated according to a Poisson distribution. The characteristics of the drivers are generated stochastically and include desired speed, desired lane, gap acceptance characteristics, and a frustration factor which determines how long a driver will tolerate following a slower driver. These characteristics are correlated so that a reckless driver generally has the characteristics associated with that description. Cars follow each other according to a reasonable car following law based on the apparent rate of change of the visual angle subtended by the leading car. This is the only simulation program that includes accidents; when two vehicles merge into the same spot, they are considered disabled and remain parked in that spot throughout the simulation.

Flexibility of this model is somewhat restricted as surface streets are assumed to intersect at right angles only. There are also restrictions on traffic signal displays that make these representations less flexible than those found elsewhere.

Validation of this model is poor; it has given "reasonable" results in one or two small tests. Due to the proprietary nature of this model, user documentation is not generally available.

Because of the detailed microscopic nature of the model, input requirements are probably quite extensive. An interesting feature here is that freeways are described by a geographic reference system which is the same as that used by many highway departments. Twenty-five separate items of information are stored for each car. Also, each 20-ft (6.1 m) section of individual lane is represented by a word in computer storage which holds the number of the vehicles in that section of road. In this way, adjacent vehicles can easily be detected by searching adjacent words in memory.

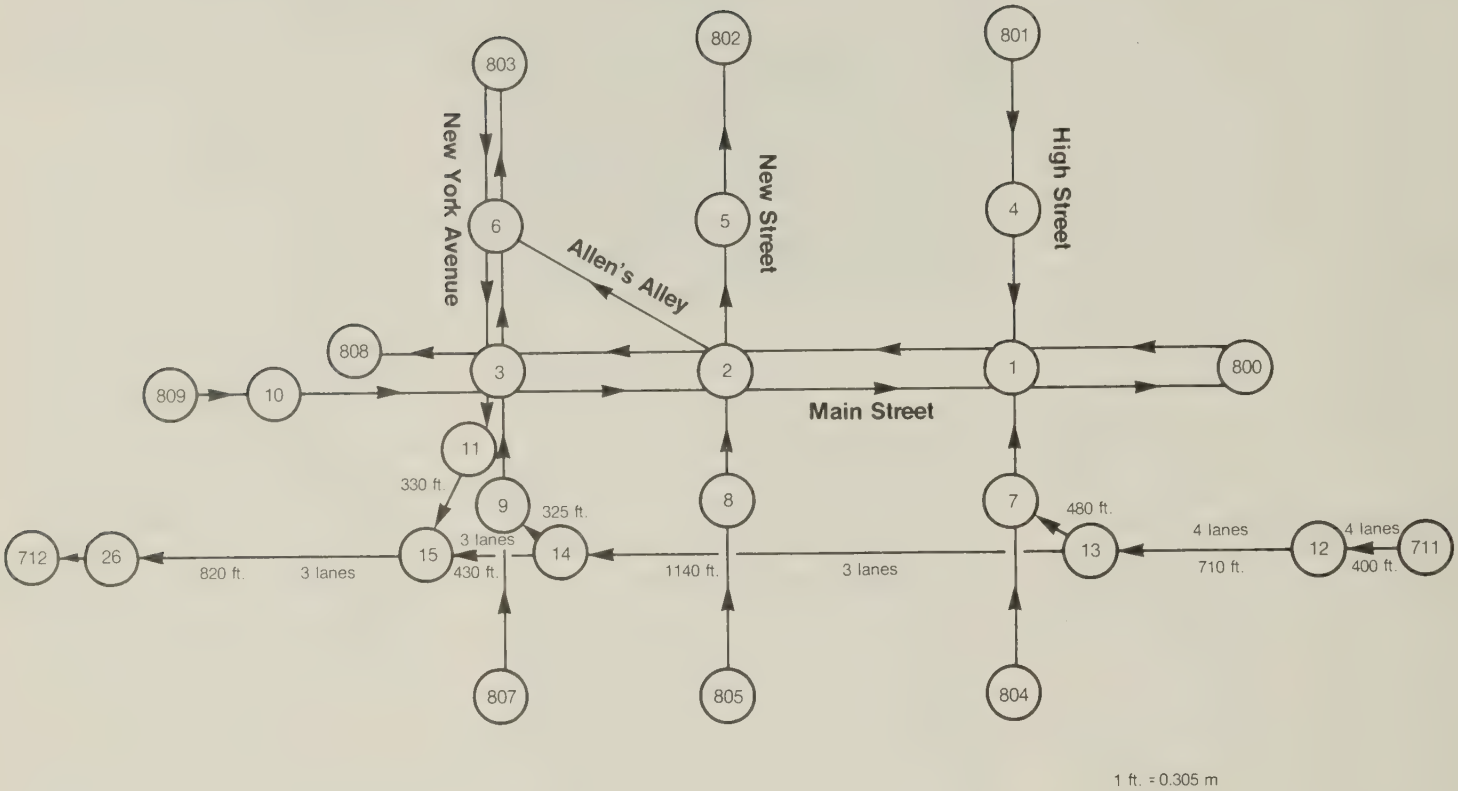
The user may choose desired output from a wide variety of traffic related measures such as average speed, average delay, fuel consumption, and vehicle emissions. The program is written in FORTRAN except for the packing and unpacking routines which are in COMPASS machine language. Speed of execution of the simulation is very slow—about one to one with real time for a moderate size network. This is due to the extreme detail of the simulation. Other interesting and useful features include movie representation and protection against loss results due to the specification of inadequate processing time. The model does not represent pedestrian interference.

2. UTCS-1 and SCOT are closely related models; they were produced for the Federal Highway Administration (FHWA) and Transportation Systems Center respectively. UTCS-1 was a result of the FHWA Urban Traffic Control System project in Washington, D.C., which evaluated traffic signal systems. (*10, 11*) It is based on the DYNET model already described. UTCS-1 is fully microscopic with vehicle data stored in packed words. Initialization runs until the number of vehicles in the system appears constant. The time step is fixed as 1 second.

SCOT (Simulation of Corridor Traffic) is UTCS-1 with the DAFT (Dynamic Analysis of Freeway Traffic) model of freeway traffic included. (*26*) On freeways, vehicles are grouped into platoons; individual statistics are not kept for each vehicle. The update period on the freeways is variable but is usually about 6 seconds.

Realism of the UTCS-1 model is excellent. Pedestrian interference is represented and the lane changing rules are reasonable. Oncoming traffic interferes with left turns as does traffic that is backed up from the preceding intersection. The freeway portion of SCOT is less realistic. The speed rule as a function of density is somewhat arbitrary and platoons have an inherent lack of realism.

UTCS-1/SCOT is fairly flexible. Almost any geometry can be represented. It is excellent for evaluating signal timing



Typical link node diagram showing the SCOT model case study network.

schemes and the effects of buses on the traffic stream. It is poor for such things as parking restrictions and changes that require route diversion by drivers.

Validation of these models is moderately good. UTCS-1 was extensively validated in Washington, D.C.; the freeway portion of SCOT was briefly evaluated in Dallas, Tex. UTCS-1 has been used extensively and is generally accepted as giving reasonable results. Documentation is hard to follow but is readily available. Input includes numerous geometric features such as the number of lanes, capacity of turning pockets, and grades. Required traffic data include the source flows and compositions (cars/trucks/buses), turning movements at each intersection, bus routes, and other items. Input preparation is difficult and time consuming because of the detailed characteristics of the data required. Some of the data from the calibration efforts can be used as default values to ameliorate the data requirements. The SCOT freeway data are particularly awkward because two required parameters must be determined by a separate computer program. These parameters are not usually directly available; indeed, even the data for the separate program are not usually available without additional data collection.

The UTCS-1/SCOT model is currently being enhanced and modularized by FHWA. The upgraded model, which will be called NETSIM (NETwork SIMulation), will be more readily adaptable to simulating unusual geometric or traffic control systems. FHWA is also preparing a state-of-the-art handbook on traffic simulation models and their applications.

Output is full and complete both link-by-link and systemwide. The new version of UTCS-1, the NETSIM model, provides emissions and fuel consumption. NETSIM, UTCS-1, and SCOT are available in versions for IBM, CDC, and UNIVAC computers. In a small network of 26 links and 12 nodes, UTCS-1 required 5 minutes and 54 seconds of IBM 360/65 CPU time to simulate 15 minutes of real time.

Among the simulation models, the UTCS-1/SCOT/NETSIM model has been the most popular. It has been used in California to analyze traffic signal timings along several arterial systems and at diamond interchanges. It has been used for the analysis of the central business district grids of Ogden and Provo, Utah. SCOT, the freeway equipped version of UTCS-1, has been used to analyze traffic-responsive versus semi-actuated strategies of signal timing along Massachusetts Avenue in Boston, Mass. These two models have also been used for research applications. The FHWA has used UTCS-1 to study the effects of right turn on red in a network and to examine advanced computer control strategies. The Transportation Research Board, Washington, D.C., performed a study on traffic control in oversaturated networks, while the Transportation Systems Center in Cambridge, Mass., studied bus priority schemes for Minneapolis, Minn.

3. Input to the SIGNET model (*27, 28*) is fairly complex. Geometric data requirements are such that a separate program was written to convert turning radii and single intersections into node-link geometry. Simulation outputs are also complex. They include values for both the system and for individual links. Statistics include total vehicle miles, total delay, average delay, delay standard deviation, and average speed. Since the program is written in FORTRAN IV (for the CDC 6500), the program may be easily modified to yield other statistics. The program will accommodate up to 85 links without modification of dimension statements and has a real time to computer time ratio of 6.5:1 for the trial network.

SIGNET borrows heavily from the TRANS model. SIGNET is fully microscopic and uses computer words to describe vehicles in a manner similar to UTCS-1/NETSIM. The model is moderately realistic but does not provide for lane changing except for turns. Stop and yield sign control is not represented. However, the model is fairly flexible in terms of intersection geometrics and traffic controllers.

Model validation is extremely limited and merely confirms that the simulation outputs are reasonable and consistent with the travel time studies.

4. Micro-Assignment (*29, 30, 31*) is a traffic assignment tool developed for transportation planning purposes; it was not designed as a traffic simulation program. (Traffic assignment is the process of loading trips onto the street network in the manner that approximates that used by actual drivers.) However, since Micro-Assignment considers origins and destinations on a block-by-block basis, it has many of the features of a simulation program. Because of its planning emphasis, it lacks many of the features normally associated with a simulation model; basically, it is designed to predict the expected traffic on any street for a given O-D pattern but, in addition, it also gives the average speed and delay on each link.

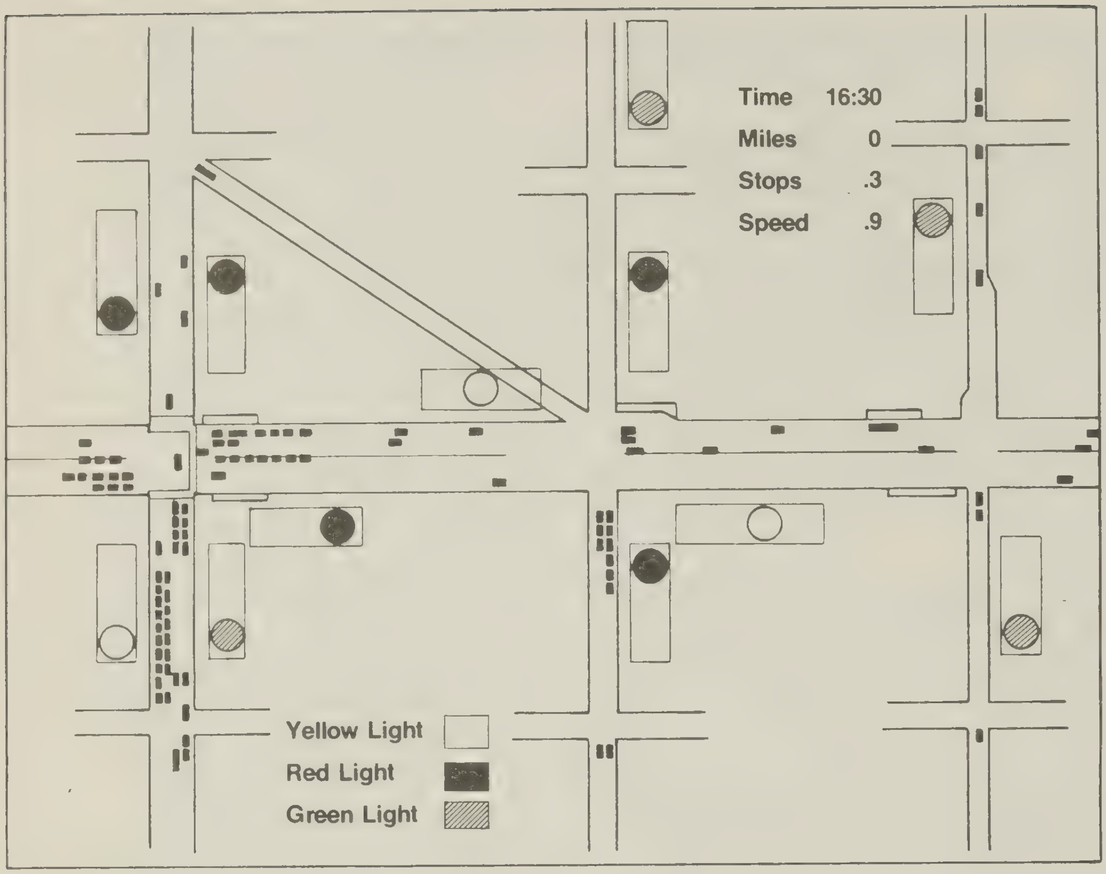
Micro-Assignment is very macroscopic (for a simulation model) and ignores many of the fine details critical to traffic engineering—in particular, the treatment of traffic signals includes only the coarsest representation of phasing and ignores the effect of offset altogether.

The key to the operation of Micro-Assignment is in the novel link-node structure. Nodes are placed in the center of each block so that a left-turning vehicle travels on a totally different link from a vehicle that is traveling straight through the intersection. In this way, each link can be assigned a fairly accurate value for the delay associated with the corresponding movement. These delays reflect the character of the control at the intersection.

For any given O-D pattern, Micro-Assignment computes the minimum path trees and then assigns all the traffic from a few origins.

Because of its macroscopic character, Micro-Assignment is able to handle large networks rapidly. Networks of about 1,000 city blocks can be accommodated. Since the assignments are essentially static, a time compression ratio is not directly pertinent. A network of 1,334 links took from 108 to 365 seconds (depending on the batch size) to do one complete assignment. If new assignments are required every 15 minutes, this corresponds to a time compression of 2.5:1 to 9:1.

5. CORQ is a new traffic simulation model written by Sam Yagar of the University of Waterloo, Waterloo, Ontario, Canada. (*32*) It has many features of the Micro-Assignment. Given a set of O-D tables for, say, successive 15-minute periods it, in effect, calculates the traffic assignments for that 15-minute period. It uses convenient link-node representation of the network and represents signals as capacity restrictions only. The unique feature of this model is that when the capacity cannot accommodate the demand, the excess vehicles are stored on the link and are added to the demand for the next period. User documentation is not yet available for CORQ.



Example frame from movie of UTCS-1 traffic network simulation.

Signal Optimization Programs

1. TRANSYT is a signal optimization program (*33, 34, 35, 36, 37*) and, therefore, outside the primary emphasis of this article. However, it contains, as an integral element, a simulation program that can be used without the optimization feature if desired. TRANSYT was developed by D. I. Robertson of the Road Research Laboratory in England.

The logic of the TRANSYT simulation program is deceptively simple. The program is totally macroscopic and completely deterministic—no random numbers are used at all. Uniform vehicular flow enters the upstream end of the farthest upstream link of the network. It arrives at the downstream end of the link where it accumulates during the red phase. When the signal turns green, vehicles discharge at the capacity rate of the signal until the queue is dissipated; thereafter the vehicles discharge at the rate at which they arrive. This emergent platoon of vehicles now has a specific shape. The platoon arrives at the next downstream stop line with a delay appropriate to the length of the link and to the speed of progression on the link. To enhance realism, the shape of the platoon is changed to reflect dispersion. The dispersion correction depends upon the length of the link and the amount of traffic. The vehicles accumulate in queue if the signal is red or pass through if it is green. Again, the discharge from the signal is at intersection capacity until the queue is discharged. In this way, traffic performance at any intersection reflects the effects of all the upstream intersections. Provision is made for turning vehicles and the arrival of vehicles at the stop line from secondary flows which have turned onto the link. Thus TRANSYT is able to represent the performance of the network in a single pass without any initialization.

Input preparation is complicated and difficult. Output is not as detailed as most models and consists primarily of delays and stops. Since the model is very simple it cannot be called realistic. However, its approximation of traffic flows is adequate to yield consistently good signal timing.

2. SIGOP II is a descendant of TRANSYT and SIGOP I (*38, 39, 40*) and, like them, is a signal optimization program. Like TRANSYT, it contains a macroscopic model of traffic flow which can be used to evaluate the stops and delays of an existing signal system. SIGOP II was developed by E. Lieberman and J. Woo for FHWA.

The logic of SIGOP II uses dynamic programing techniques to represent traffic streams as being states with specific properties which pass through stages (the intersections) where they are transformed by the decision variables (traffic control) into departing streams which exhibit different state properties. SIGOP II orders the traffic flows into nine combinations of primary and secondary platoons depending on when the platoon departs from the first intersection and when it arrives at the second intersection. Modification of platoon characteristics depends on the length of the link and the speed of progression on the link. Provision is made for turning movements and turn "pockets." One evolutionary step in SIGOP II is its ability to model the effects of multiple phase signals.

Input preparation is fairly straightforward once the notation for describing multiple phase signals is mastered. Error messages and diagnostic tests are embedded in SIGOP II to help the user. Special coding sheets are included with the documentation as is a fully documented case study. Output includes time space plots of signal control along specified arterials as well as link statistics. SIGOP II computer speed varies linearly with the size of the network so that it is slightly slower than TRANSYT for small networks and is somewhat faster for large networks. SIGOP II is still being field tested by FHWA and is not currently available.

3. CORQIC is a corridor optimization program. (*41*)[3] As such, the range of networks that can be analyzed is restricted; the program explicitly assumes a one-directional freeway and a parallel arterial with two-way connecting streets. The freeway part of the program is FREQ3C which was developed by A. D. May and his coworkers at the University of California, Berkeley, Calif. The surface street portion is the TRANSYT program discussed above. Since both models are fairly simple, macroscopic formulations input requirements are modest and execution speed should be quite short. CORQIC is too new for significant quantities of data to have been accumulated. The authors know of no case where the simulation part of this program has been used without the optimization.

General Remarks and Conclusions

At least one direct comparison of some of these models has been done. Stephen Cohen of FHWA simulated a network of 13 signalized intersections and 27 links from Washington, D.C., using TRANSYT, SIGOP II, TRANS, and UTCS-1/NETSIM. CPU times were respectively 5.5 seconds, 6 seconds, 8 minutes, and 13.35 minutes. (TRANS was run on a surviving IBM 7090, the other models on an IBM 360/65 computer.) These times reflect the fact that TRANSYT and SIGOP II are static macroscopic calculations whereas TRANS is a platoon dynamic model and UTCS-1/NETSIM is a dynamic microscopic model. In the two dynamic models, CPU time is proportional to length of period simulated—32 minutes here. In the static models, the CPU time is proportional to the number of origin-destination tables simulated—one in this test. The comparison found that TRANSYT, SIGOP II, and TRANS overestimated the average traffic speeds measured in the field by approximately 30 percent. The UTCS-1/NETSIM also overestimated the average speed, but only by 8 percent.

With the above conclusion in mind, it may be said that traffic simulation has been a success. The UTCS-1/NETSIM traffic simulation model has been used to evaluate the desirability of pedestrian all walk intervals in Ogden, Utah, and showed that too much delay would result. Second generation computer signal control strategies were evaluated with simulation by FHWA. Traffic signal warrants have been analyzed by simulation for the Transportation Research Board. The desirability of interconnecting signals along an arterial is currently being examined in California, and the use of simulation as a potential tool for studying left-turn phasing requirements is being examined in New Jersey. Users of the UTCS-1/NETSIM traffic simulation model have been generally happy with the results obtained. Traffic control strategies can be almost routinely tested once the initial data collection and coding have been completed.

Some general criticisms may be made of traffic simulation models. While the macroscopic models are fast, their realism is questionable and they are unable to portray dynamic phenomena except as represented in a new origin-destination table. They are, at best, a succession of static models. Most dynamic models—platoon and microscopic—have poor initialization criteria and generally lack such elements of modern simulation theory as independent random variable streams and variance reducing techniques. The microscopic models, while slower, tend to give the most realistic and accurate representation of traffic flow and should be used where computer resources permit.

In spite of these criticisms, these traffic simulation models have been a success. Their users are generally happy with the results obtained. Traffic control strategies can be almost routinely generated or pretested with these models.

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Our Authors

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Donald A. Gordon is a research psychologist in the Traffic Systems Division, Office of Research, Federal Highway Administration. He has been involved in research on the driving process and the development of improved signs and roadway delineations. He has published articles on topics such as perception in vehicular guidance, experimental isolation of the driver's visual input, and the contribution of psychology to the traffic flow theory. Lately, he has been concerned with the development of a highway simulator.

Joseph A. Boyle is a graduate student in the Department of Psychology, Catholic University, Washington, D.C. He is working with the Traffic Systems Division, Federal Highway Administration. He has assisted in research on the development of improved parking-standing signs. Lately, he has been concerned with the development of improved highway lane occupancy signs.

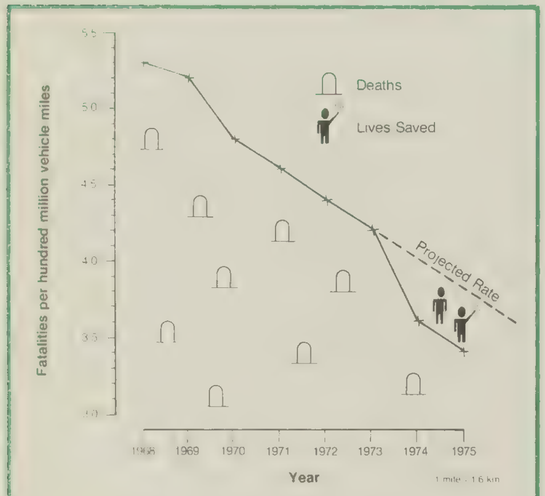
Henry C. Huckins is a highway engineer in the Implementation Division, Office of Development, Federal Highway Administration. Mr. Huckins began his career in the highway field in 1956 as Instrument Lab Chief on the AASHO Road Test located in Illinois. At the termination of this project in 1962, he came to the Fairbank Highway Research Station as a member of the research staff. Since 1973, he has been involved in the wet weather accident reduction program, particularly in the area of pavement friction measurement and the standardization of friction data on a national basis.

David Gibson is a highway engineer in the Implementation Division, Office of Development, Federal Highway Administration. Before joining FHWA in 1974, he worked for the District of Columbia Department of Transportation in traffic engineering and developed a strong interest in techniques for improving traffic flow. He is currently working on developing computer programs to control traffic signals and to compute traffic control strategies.

Paul Ross is a research physical scientist in the Traffic Systems Division, Office of Research, Federal Highway Administration. His fields of interest include traffic flow theory and simulation. He has published other research in traffic simulation and organized conferences in the field. Prior to his employment with the Federal Highway Administration, he was a college physics professor.

Recent Research Reports

The following are brief descriptions of selected reports recently published by the Office of Research, Federal Highway Administration, which includes the Structures and Applied Mechanics Division, Materials Division, Traffic Systems Division, and Environmental Design and Control Division. The reports are available from the address noted at the end of each description.



Nationwide Fatality Rate, 1968-1975

Safety Aspects of the National 55 MPH Speed Limit, Report No. FHWA-RD-76-191

by FHWA Traffic Systems Division

This report examines the effect of the national 55 mph (89 km/h) speed limit on highway safety in the United States. Fatality rates and injury rates since the enactment of the speed limit are compared to projected fatality and injury rates based on data collected before the speed limit went into effect. This comparison is made for nationwide data on various highway systems and for a representative sample of 17 States. Additionally, factors such as pedestrian fatalities, age of driver in fatal accidents, time of fatal accidents, and type of vehicle involved are studied.

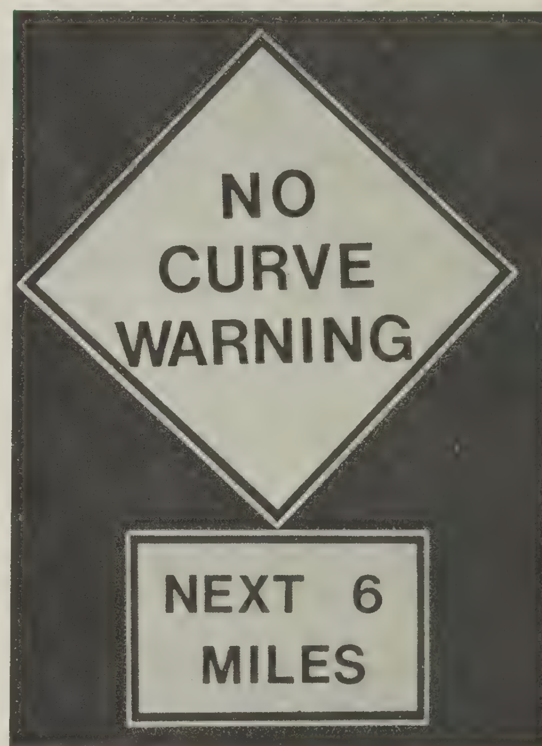
The conclusion of the report is that fatalities per hundred million vehicle miles of travel (hundred million vehicle kilometres of travel) have been reduced by the enactment of the speed limit. A more detailed summary of the research report appears on pages 58 to 67 of this issue of *Public Roads*. This summary article also contains a numerical estimate of how many fatalities and injuries have been saved as a result of speed reductions caused by the new national 55 mph (89 km/h) speed limit.

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 267989).

Signs and Markings for Low Volume Rural Roads, Report No. FHWA-RD-77-39

by FHWA Traffic Systems Division

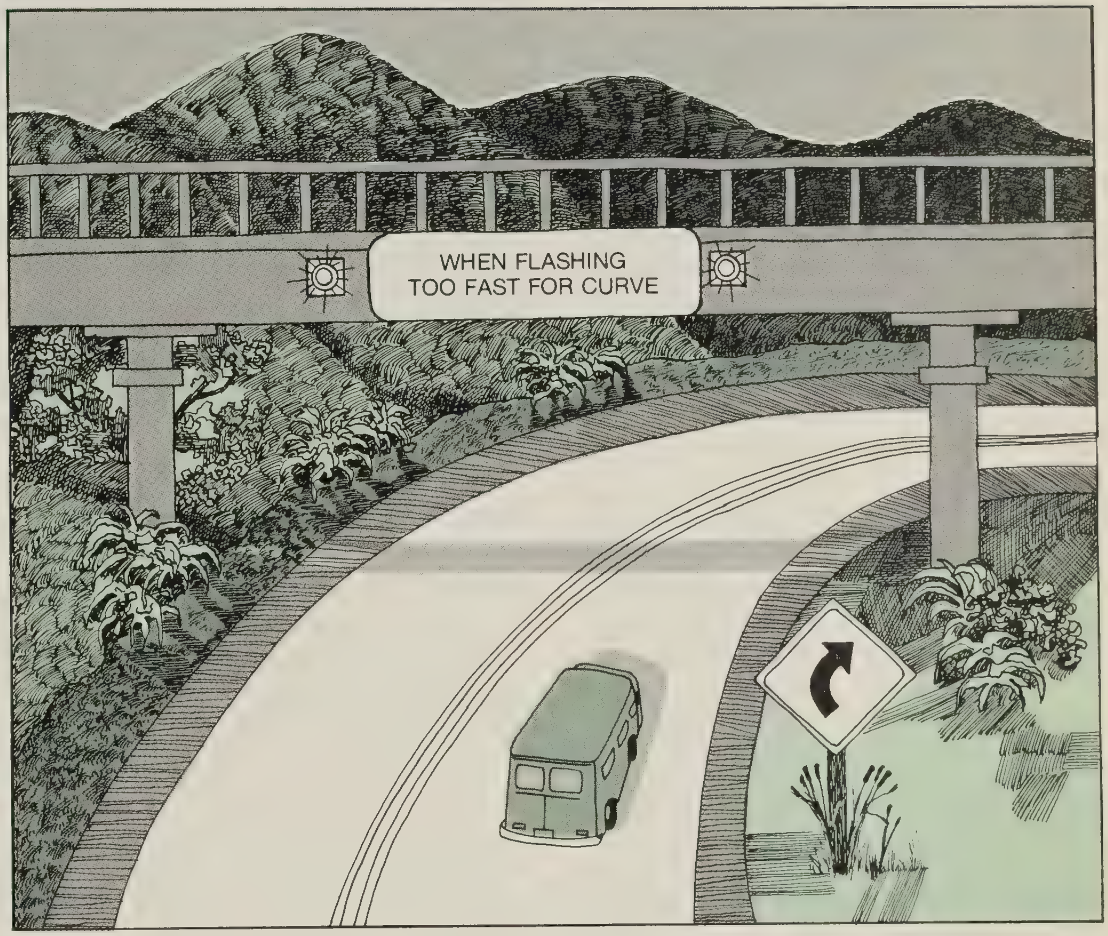
Several types of low volume roadways (average daily traffic of less than 400 vehicles) exhibiting a wide range of both design and functional characteristics are investigated in this study. Since the probability of an accident on these roadways is low, the report attempts to define what warning and regulatory signs and markings are needed to adequately balance safety with road economy.



To meet the study objective, the various types of low volume rural roads were initially classified by system (Federal, State, or county controlled) and type of surface (paved or unpaved). Using a modified form of driver task analysis, the travel characteristics were analyzed to determine the warning and regulatory signs and markings needed for satisfactory operation and safety. This analysis led to guidelines for the application of stop signs, curve warnings, and passing zone signs and markings. These guidelines were validated by testing 270 drivers to obtain their reactions to the proposed signs and markings.

The report is available from the Traffic Systems Division, HRS-30, Federal Highway Administration, Washington, D.C. 20590.

You Should Know About



Efficacy of Red and Yellow Turn Arrows in Traffic Signals, Report No. FHWA-RD-76-2

by FHWA Traffic Systems Division

A study of the effectiveness of red, yellow, and green steady (nonflashing) turn arrows in traffic signals is described in this report. Four basic study methods were employed: (1) collecting historical data on use of traffic signals and arrows; (2) determining the present practices and opinions of senior traffic engineers in both State and local jurisdictions throughout the United States; (3) obtaining the responses of motorists to a written questionnaire designed to test the understanding of various arrow displays; and (4) "before" and "after" field testing to determine motorist response and accident experience at six Washington, D.C., and two suburban Maryland sites which had experimental turn arrow displays. All of these elements of the study are described in detail in the report.

The arrows were found to be beneficial in some circumstances. Conclusions, recommendations, and guidelines for the use of three-color turn arrow displays are provided in the report. A user's manual to aid in implementing such arrow displays also is included.

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 264751).

Guidelines for Flashing Traffic Control Devices, Report No. FHWA-RD-76-190

by FHWA Traffic Systems Division

In the past, various types of intersection and nonintersection related flashing beacons have been used where a hazard existed. Now a set of guidelines is available which quantifies the justification for the devices. The guidelines are based on results of a state-of-the-art review, extensive field work, accident studies, and analytical investigations.

The following types of flashing beacon applications were examined:

- Continuous overhead beacons.
- The use of a vehicle actuated beacon with a "When Flashing—Vehicle Crossing" advisory sign.
- The use of a continuous beacon mounted on a "Stop Ahead" sign.
- The use of a "Speed Violation—When Flashing" advisory sign.
- The use of a "Too Fast for Curve—When Flashing" sign.

The guidelines developed as a result of this project will enable the engineer to select the most effective device for each situation to encourage motorist responses consistent with a higher level of safety. The devices are presented graphically with a set of procedures for their use.

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 264801).

Asphalt Concrete Overlays of Flexible Pavements, Volume 1 (Report No. FHWA-RD-75-75) and Volume 2 (Report No. FHWA-RD-75-76)

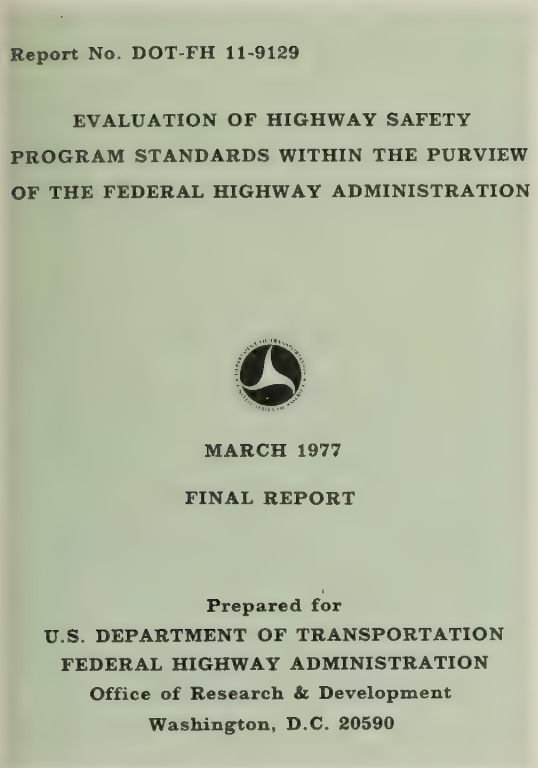
by FHWA Structures and Applied Mechanics Division

This two-volume report establishes fatigue cracking and rut depth as appropriate design criteria for asphalt concrete overlays of flexible pavements and provides design procedures based on the use of linear elastic layered theory for the analytical model. The design procedures include consideration of the structural value of the existing pavement, its remaining life, and the functioning of the various layers in an overlaid pavement. The design procedures are presented in two forms—a partial hand solution and a computer program.

Volume 1, **Development of New Design Criteria,** discusses the methodology for establishing the design criteria, summarizes the overlay design procedures, and compares these with four existing procedures.

Volume 2, **Design Procedures,** contains all the elements necessary for the user to design an asphalt concrete overlay of a flexible pavement based on inputs from deflection testing, condition surveys, traffic data, and materials characterization. A complete example problem and solution are presented.

Both volumes of the report are available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Vol. 1—Stock No. PB 263432; Vol. 2—Stock No. PB 263433).


Report No. DOT-FH 11-9129

EVALUATION OF HIGHWAY SAFETY PROGRAM STANDARDS WITHIN THE PURVIEW OF THE FEDERAL HIGHWAY ADMINISTRATION

MARCH 1977

FINAL REPORT

Prepared for
U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
Office of Research & Development
Washington, D.C. 20590

Evaluation of Highway Safety Program Standards Within the Purview of the Federal Highway Administration—Final Report, Report No. DOT-FH-11-9129

by FHWA Environmental Design and Control Division

This report evaluates the adequacy and appropriateness of the highway safety program standards in the Federal Highway Administration (FHWA). The standards are No. 9, Identification and Surveillance of Accident Locations; No. 12, Highway Design, Construction, and Maintenance; No. 13, Traffic Engineering Services; and the engineering portion of No. 14, Pedestrian Safety. Information for the evaluation was provided by FHWA, State highway agencies, Governor's Highway Safety Representatives, counties, cities, and private and public organizations.

Suggestions for appropriate alternate or revised approaches to the existing standards are reported. The following key recommendations are made as the basis for enhancing highway safety:

- Restate the existing standards to include those procedural elements related to the planning, development, and evaluation of a comprehensive highway safety program.
- Develop procedures and guidelines for selecting and implementing specific countermeasures and distribute them to Federal, State, and local agencies.
- Direct new safety standards to the areas of accident reporting and records; safety improvement planning and programing; design, construction, maintenance, and operations; manpower; and evaluations.

The report is available from the Environmental Design and Control Division, HRS-41, Federal Highway Administration, Washington, D.C. 20590.

Rational Determination of Priority Targets for Research and Development: Summary of Guidelines for Establishing R&D Priorities

by FHWA Environmental Design and Control Division


Draft Report August 1976

RATIONAL DETERMINATION OF PRIORITY TARGETS FOR RESEARCH AND DEVELOPMENT

Summary of Guidelines for Establishing R&D Priorities at FHWA

By GORDON I. THOMPSON
PURNENDU CHATTERJEE

Prepared for

FEDERAL HIGHWAY ADMINISTRATION
TRANS POINT BUILDING
WASHINGTON, D.C. 20590

STANFORD RESEARCH INSTITUTE
Menlo Park, California 94025 · U.S.A.

The rational determination of priority targets for research and development is a major problem confronting many Government agencies. This report presents a set of guidelines for conducting decision analyses developed to assist the Federal Highway Administration (FHWA) in establishing research and development (R&D) priorities. A general discussion of the basic steps that would typically be followed in evaluating R&D alternatives by the decision analysis approach is provided.

The methodology described has been applied to several R&D projects and is a logical and consistent approach to evaluating projects and establishing priorities among those projects. Uncertainties on research outcomes and social and economic impacts can be handled effectively by this decision analysis methodology. In addition, the methodology is widely applicable and can be used to make decisions at several levels, from individual project planning to establishing priorities among research areas for FHWA as a whole.

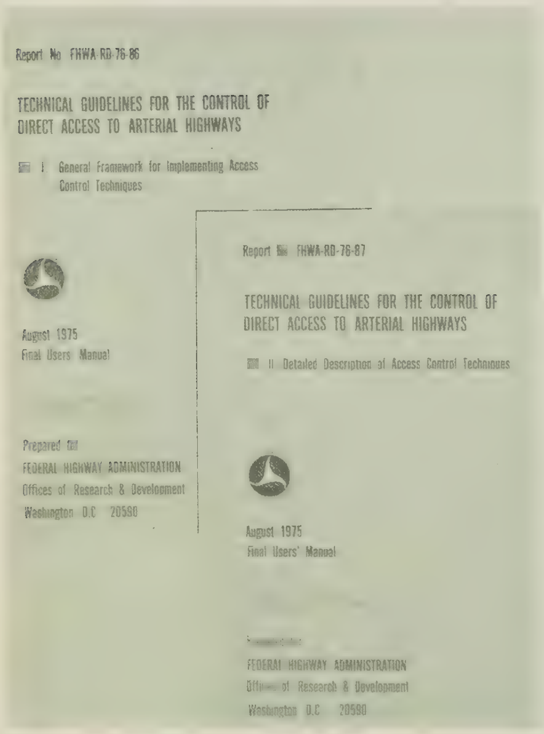
The report also presents new material on portfolio selection methods and a program for assessing values of R&D outcomes. A Highway Information Library for use in the R&D planning process is described.

The report is available from the Environmental Design and Control Division, HRS-41, Federal Highway Administration, Washington, D.C. 20590.

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

The following are brief descriptions of selected items which have been recently completed by State and Federal highway units in cooperation with the Implementation Division, Offices of Research and Development, Federal Highway Administration (FHWA). Some items by others are included when they have a special interest to highway agencies. These items will be available from the Implementation Division unless otherwise indicated.

U.S. Department of Transportation
Federal Highway Administration
Offices of Development
Implementation Division, HDV-20
Washington, D.C. 20590


Report No. FHWA-RD-76-86

TECHNICAL GUIDELINES FOR THE CONTROL OF DIRECT ACCESS TO ARTERIAL HIGHWAYS

Vol. I General Framework for Implementing Access Control Techniques

August 1975
Final Users' Manual

Prepared for
FEDERAL HIGHWAY ADMINISTRATION
Offices of Research & Development
Washington, D.C. 20590

Report No. FHWA-RD-76-87

TECHNICAL GUIDELINES FOR THE CONTROL OF DIRECT ACCESS TO ARTERIAL HIGHWAYS

Vol. II Detailed Description of Access Control Techniques

August 1975
Final Users' Manual

Prepared for
FEDERAL HIGHWAY ADMINISTRATION
Offices of Research & Development
Washington, D.C. 20590

Technical Guidelines for the Control of Direct Access to Arterial Highways, Volume I (Report No. FHWA-RD-76-86) and Volume II (Report No. FHWA-RD-76-87)

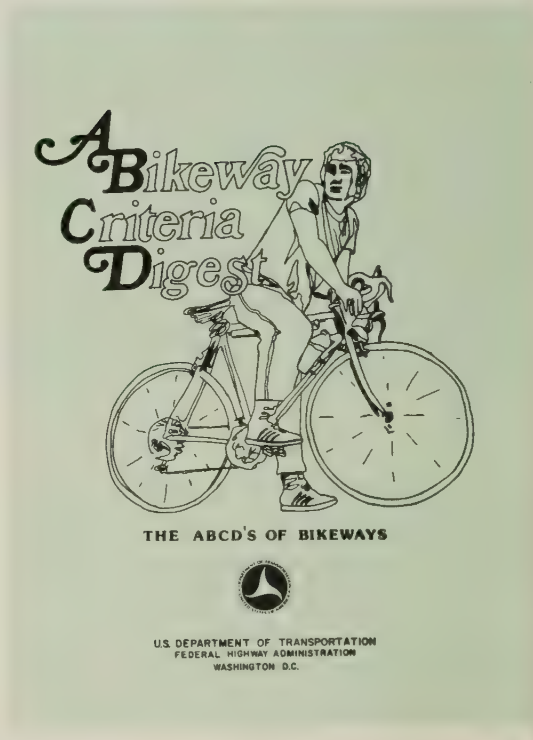
by FHWA Implementation Division

These reports give highway agencies general guidelines toward a more comprehensive application of direct access controls to commercial properties on arterial highways. The two volumes provide highway agencies with a basic orientation tool which can be applied to counteract operational deficiencies on existing highways or improve proposed designs for new or reconstructed highways. They can also be used in connection with the development of a thorough access control policy as a means of protecting the functional integrity of the highway.

Volume I, **General Framework for Implementing Access Control Techniques,** discusses the basic problem dimension, summarizes the evaluation of the 70 identified techniques, and gives a general decision framework for implementation of access controls.

Volume II, **Detailed Description of Access Control Techniques,** is a reference document containing detailed descriptions of the design, application, cost, operational effectiveness, and cost effectiveness of each access control technique.

The reports are available from the Implementation Division.



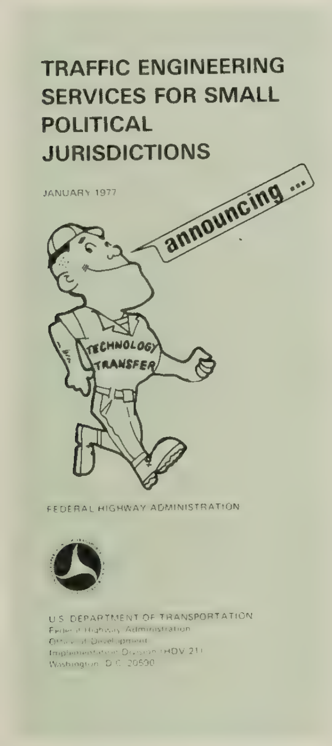
A Bikeway Criteria Digest, the ABCD's of Bikeways

by FHWA Implementation Division

This digest provides users with access to current practices in the expanding art of bikeway facility planning and design. Planned and constructed bikeways are not always the solution to bicyclists' needs or desires; sometimes, a bikeway has an adverse impact on bicycle usage and the solution to problems or needs could have been found in legislation, law enforcement, or perhaps no action at all. This criteria digest first indicates whether or not a facility is needed; then, if a bikeway is the answer, it tells how to implement one.

To be useful to small government units with limited staff as well as State and Federal agencies with complex specialized staffs, this digest is organized into four major, topical steps: Planning, Location, Design, and Operations. These steps provide enough information for a planner or designer to be able to construct a bikeway wherever one is desired.

The digest is available from the Implementation Division.



Traffic Engineering Services for Small Political Jurisdictions, Implementation Package 77-6

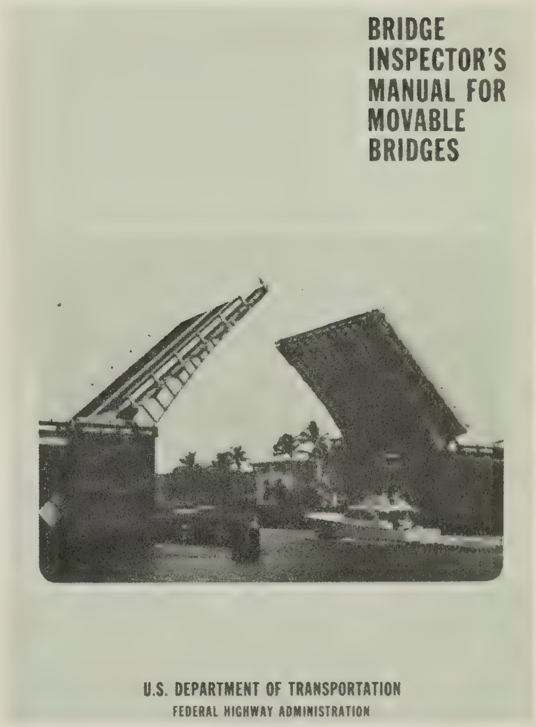
by FHWA Implementation Division

It is important that the need for traffic engineering services be recognized in all jurisdictions, regardless of size. Short term, stop-gap methods are no longer valid in solving problems that are continually growing more complex, particularly in urban areas. Smaller communities are becoming more involved with and responsible for systematic, comprehensive traffic engineering services and programs.

This report reviews the status of traffic engineering services as applied in cities and counties varying in population from 2,500 to 40,000. The data resulted from questionnaires sent to the 50 States and 1,350 selected cities and counties. The study found that traffic engineering services and studies for local off-system streets in smaller jurisdictions were greatly dependent on the availability of Federal-aid funds.

The document contains a matrix to enable smaller jurisdictions to evaluate their current programs and a guideline for assistance in obtaining services.

The report is available from the Implementation Division.



Bridge Inspector's Manual for Movable Bridges

by FHWA Bridge Division, Office of Engineering, and FHWA Implementation Division

Movable bridges, which are opened for marine traffic, are built to meet rigid specifications. They are designed to provide many years of service because they operate in a harsh environment. Periodic maintenance and inspection are vital for movable bridges to remain in service. This new manual is intendec to be an instructional tool for training bridge inspectors. Also, it serves as a reference and guide for experienced inspectors. A thorough understanding of the movable bridge's mechanical and electrical components which are covered in this manual should enable an inspector, after some experience, to make a competent inspection of this unique type of bridge.

The following topics are discussed in the manual: the three basic types of movable bridges, their method of operation, and the components associated with each type; the components found on all movable bridges; the machinery unique to swing-spans, vertical lifts, and bascules; actual onsite inspection; operation and inspection of electrical equipment; motor and bridge control circuitry; testing and inspection procedure for electrical equipment and circuitry; and safety procedures to be followed for an electrical inspection. An appendix and a glossary of mechanical and electrical terms also are included.

The manual is available from the Implementation Division.

A Summary of Highway Facilities Where Hazardous Materials Are Restricted

by FHWA Implementation Division

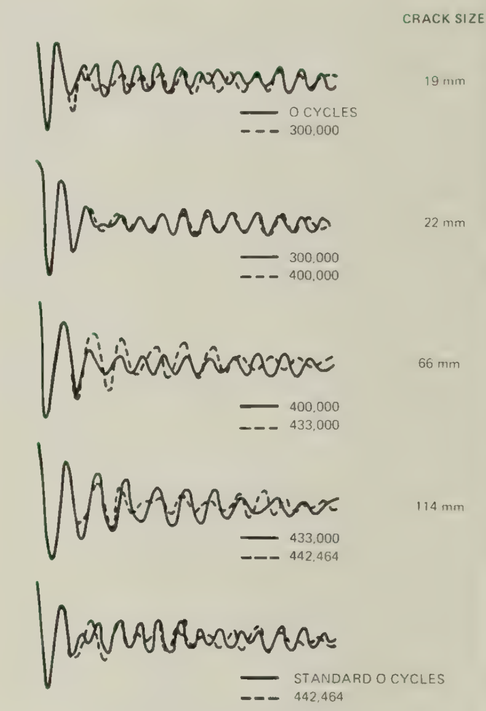
The transportation of hazardous materials is restricted or prohibited on many highway facilities. This report identifies all those restricted facilities which were reported by the Division Offices of the Federal Highway Administration and State and local agencies during a recent survey. This identification should help the carriers of hazardous materials to more easily determine the fastest and safest routes that they can use.

The report is divided into two parts. Part I contains an alphabetical listing of those States where no restrictions were reported. Part II contains listings, by State, of those highway facilities which have restrictions and summarizes these restrictions.

The report is available from the Bureau of Motor Carrier Safety, HMC-11, Federal Highway Administration, Washington, D.C. 20590.

ERRATA

In the article "The Effect of Yielding on the Fatigue Strength of Steel Beams," in the June 1977 issue of *Public Roads*, Vol. 41, No. 1, figure 15, page 17, was not shown. This figure appears below.



"RANDOMDEC" VIBRATION SIGNATURES FROM LABORATORY BEAM.

New Research in Progress

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

FCP Category 1—Improved Highway Design and Operation for Safety

FCP Project 1A: Traffic Engineering Improvements for Safety

Title: Estimating the Safety Benefits for Alternative Highway Geometric and/or Operational Improvements. (FCP No. 31A1694)
Objective: Evaluate highway safety design improvements through an analysis of existing accident records. Develop an estimation procedure for the implementation of new construction and spot roadway improvements based on predicted accident and/or severity rates for the applicable site.
Performing Organization: Alabama State Highway Department, Montgomery, Ala. 36104
Expected Completion Date: September 1978
Estimated Cost: $150,000 (FHWA Administrative Contract)

FCP Project 1C: Analysis and Remedies of Freeway Traffic Disturbances

Title: Guidelines for Design and Operation of Ramp Control Systems. (FCP No. 51C3503)

Objective: Develop design guidelines and standards for ramp control projects which will address selection of electronic hardware, control logic, and ramp displays and markings for real-time and isolated types of ramp control installations. Key guidelines to several levels of system cost.
Performing Organization: Texas Transportation Institute, College Station, Tex. 77843
Expected Completion Date: May 1979
Estimated Cost: $250,000 (NCHRP)

FCP Project 1E: Safety of Pedestrians and Abutting Property Occupants

Title: Effective Treatments of Over and Undercrossings for Use by Bicyclists, Pedestrians, and the Handicapped. (FCP No. 31E3032)
Objective: Determine the feasibility of and develop warrants and design strategies for over and undercrossings for bicyclists, pedestrians, and the handicapped.
Performing Organization: DeLeuw Cather and Company, San Francisco, Calif. 94120
Expected Completion Date: March 1979
Estimated Cost: $190,000 (FHWA Administrative Contract)

FCP Project 1G: Safer Traffic Guardrails and Bridge Railings

Title: Improvement of Safety Facility Accessories. (FCP No. 41G1244)
Objective: Determine performance of curved guardrail installations. Develop transition between concrete median barrier and box-beam median barrier. Develop a new post-to-rail connection for the box-beam median barrier.
Performing Organization: New York Department of Transportation, Albany, N.Y. 12226
Expected Completion Date: March 1980
Estimated Cost: $139,000 (HP&R)

FCP Project 1J: Improved Geometric Design

Title: Highway Geometric Design Consistency Related to Driver Expectancy. (FCP No. 31J3302)
Objective: Develop and evaluate procedures and criteria to identify and correct inconsistent highway geometric design elements on rural nonfreeway facilities by providing for a geometrically consistent path guidance system for the driver. A literature review, a consistency evaluation of design elements, and a traffic performance evaluation with videotapes will be included.
Performing Organization: Texas A&M Research Foundation, College Station, Tex. 77843
Expected Completion Date: June 1978
Estimated Cost: $143,000 (FHWA Administrative Contract)

FCP Project 1L: Improving Traffic Operations During Adverse Environmental Conditions

Title: Augmentation of Earth Heating for Purposes of Roadway. (FCP No. 31L7042)
Objective: Develop preliminary design and cost information of systems for augmenting earth heat either with

another heat source or by conservation. Do some experimental or developmental work.
Performing Organization: Dynatherm Corporation, Cockeysville, Md. 21030
Expected Completion Date: September 1978
Estimated Cost: $87,000 (FHWA Administrative Contract)

FCP Category 2—Reduction of Traffic Congestion, and Improved Operational Efficiency

FCP Project 2D: Traffic Control for Coordination of Car Pools and Buses on Urban Freeway Priority Lanes

Title: Enforcement Requirements for High Occupancy Vehicle Facilities (FCP No. [illegible]2D4554)
Objective: Determine enforcement guidelines for freeway and surface street operations which provide preferential treatment for high occupancy vehicles. Provide model legislation to permit application of effective enforcement techniques where current law limits enforcement procedures.
Performing Organization: Beiswenger, Hoch, and Associates, North Miami Beach, Fla. 33160
Expected Completion Date: April 1978
Estimated Cost: $83,000 (FHWA Administrative Contract)

FCP Project 2K: Metropolitan Intermodal Traffic Management

Title: Simulation of Traffic Management Strategies. (FCP No. 22K2014)

Objective: Apply traffic simulation models to the evaluation of traffic management strategies making the necessary changes in the models.
Performing Organization: Federal Highway Administration, Washington, D.C. 20590
Expected Completion Date: October 1979
Estimated Cost: $60,000 (FHWA Staff Research)

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

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

Title: Sewage Treatment at Freeway Rest Areas. (FCP No. 43E1152)
Objective: Assess the potential and field test the effectiveness of land treatment systems for treatment and polishing of sanitary effluents from rest areas. Develop and field test a spray system for applying rest area wastewater to highway medians. Determine cost effectiveness of these systems.
Performing Organization: Michigan State University, East Lansing, Mich. 48824
Funding Agency: Michigan Department of State Highways and Transportation
Expected Completion Date: May 1978
Estimated Cost: $125,000 (HP&R)

Title: Runoff Water Quality. (FCP No. 43E3072)
Objective: Improve the capability to determine the significance of highway development and maintenance to highway runoff problems in Washington.
Performing Organization: University of Washington, Seattle, Wash. 98195
Funding Agency: Washington Department of Highways
Expected Completion Date: April 1982
Estimated Cost: $498,000 (HP&R)

FCP Project 3F: Pollution Reduction and Visual Enhancement

Title: Environmental Engineering Investigation Along the Yukon River—Prudhoe Bay Haul Road. (FCP No. 33F1022)
Objective: Determine the rate and magnitude of thaw penetration and subsidence. Document existing vegetation types and rate of change in plant communities. Determine methods to enhance restoration by native plant species. Characterize the annual air and ground temperatures and precipitation regimes along the Haul Road.
Performing Organization: U.S. Army Cold Regions Research and Engineering Laboratory, Hanover, N.H. 03755
Expected Completion Date: March 1979
Estimated Cost: $280,000 (FHWA Administrative Contract)

FCP Category 5—Improved Design to Reduce Costs, Extend Life Expectancy, and Insure Structural Safety

FCP Project 5B: Tunneling Technology for Future Highways

Title: Representative Ground Parameters for Structural Analysis of Tunnels. (FCP No. 35B3032)
Objective: Evaluate the accuracy, sensitivity, and reliability of site exploration information which is required for the design and construction of tunnels.
Performing Organization: Delon Hampton and Associates, Washington, D.C. 20036
Expected Completion Date: March 1979
Estimated Cost: $426,000 (FHWA Administrative Contract)

FCP Project 5D: Structural Rehabilitation of Pavement Systems

U.S. Government Printing Office: 1977—720-212/4

Title: Stresses and Strains in ACHM Overlays on PCC Pavements. (FCP No. 45D2474)
Objective: Purchase or construct instrumentation for the monitoring of vertical and horizontal displacements, vertical stresses, temperature, and moisture profiles to determine the actual behavior of AC overlay structure in the vicinity of joints in the overlaid PCC pavement. Develop design criteria for overlays to prevent reflection cracking.
Performing Organization: Arkansas State Highway and Transportation Department, Little Rock, Ark. 72203
Expected Completion Date: December 1979
Estimated Cost: $140,000 (HP&R)

Title: Development of a Pavement System for Oregon Highways. (FCP No. 35D3272)
Objective: Determine the rate of deterioration of pavements, the type of treatments and when to apply them, and the benefits in terms of extended service life. Develop a pattern management system incorporating these to optimize maintenance strategies.
Performing Organization: Oregon Department of Transportation, Salem, Oreg. 97310
Expected Completion Date: November 1979
Estimated Cost: $279,000 (FHWA Administrative Contract)

Title: Effects of Vehicle Size, Weight, and Configuration on Pavement Performance and Maintenance. (FCP No. 35D3272)
Objective: Determine the effects of changes in truck size, weight, and configuration on pavement performance. Relate these effects to pavement design, maintenance, and rehabilitation processes and their related costs.
Performing Organization: Austin Research, Austin, Tex. 78746
Expected Completion Date: September 1978
Estimated Cost: $234,000 (FHWA Administrative Contract)

FCP Project 5F: Structural Integrity and Life Expectancy of Bridges

Title: The Overloading Behavior of Composite Beam-Slab Type Highway Bridges. (FCP No. 45F3742)
Objective: Develop a reliable method of analysis to estimate the load carrying capacity of beam-slab type bridge superstructures with reinforced deck and steel girders.
Performing Organization: Lehigh University, Bethlehem, Pa. 18015
Funding Agency: Pennsylvania Department of Transportation
Expected Completion Date: September 1980
Estimated Cost: $316,000 (HP&R)

Title: Implementation of Program BOVA. (FCP No. 45F3753)
Objective: Refine and simplify computer based determination of the overloading of beam-slab type highway bridges.
Performing Organization: Lehigh University, Bethlehem, Pa. 18015
Funding Agency: Pennsylvania Department of Transportation
Expected Completion Date: June 1978
Estimated Cost: $59,000 (HP&R)

FCP Project 5H: Protection of the Highway System from Hazards Attributed to Flooding

Title: Laboratory Study of Scour Depths at Culvert Outlets. (FCP No. 35H1172)
Objective: Conduct laboratory experiments to determine scour depths at culvert outlets for several sizes of cohesionless soils and two classes of cohesive soils. Soils include fine sand, gravel, sand and gravel, and sandy clay and gravel. Culvert sizes include 10 in, 15 in, and 18 in (254 mm, 381 mm, and 457 mm) diameters.
Performing Organization: Colorado State University, Fort Collins, Colo. 80523
Expected Completion Date: June 1979
Estimated Cost: $179,000 (FHWA Administrative Contract)

Title: Urban Highway Storm Drainage Design Model. (FCP No. 35H2042)
Objective: Develop a comprehensive urban highway storm drainage system computer design model from available knowledge and experience for use by highway engineers in the design of such facilities.
Performing Organization: Water Resources Engineers, Inc., Springfield, Va. 22151
Expected Completion Date: October 1978
Estimated Cost: $166,000 (FHWA Administrative Contract)

FCP Category 7—Improved Technology for Highway Maintenance

FCP Project 7C: Management

Title: Technology Transfer to Reduce Highway Costs. (FCP No. 47C2036)
Objective: An all-inclusive review of the State's design and construction policies, practices, standards, and specifications aimed at identifying areas, activities, and items in which cost reductions can be realized through better utilization of existing technology.
Performing Organization: Illinois Department of Transportation, Springfield, Ill. 62764
Expected Completion Date: December 1978
Estimated Cost: $56,000 (HP&R)

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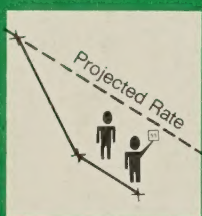

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