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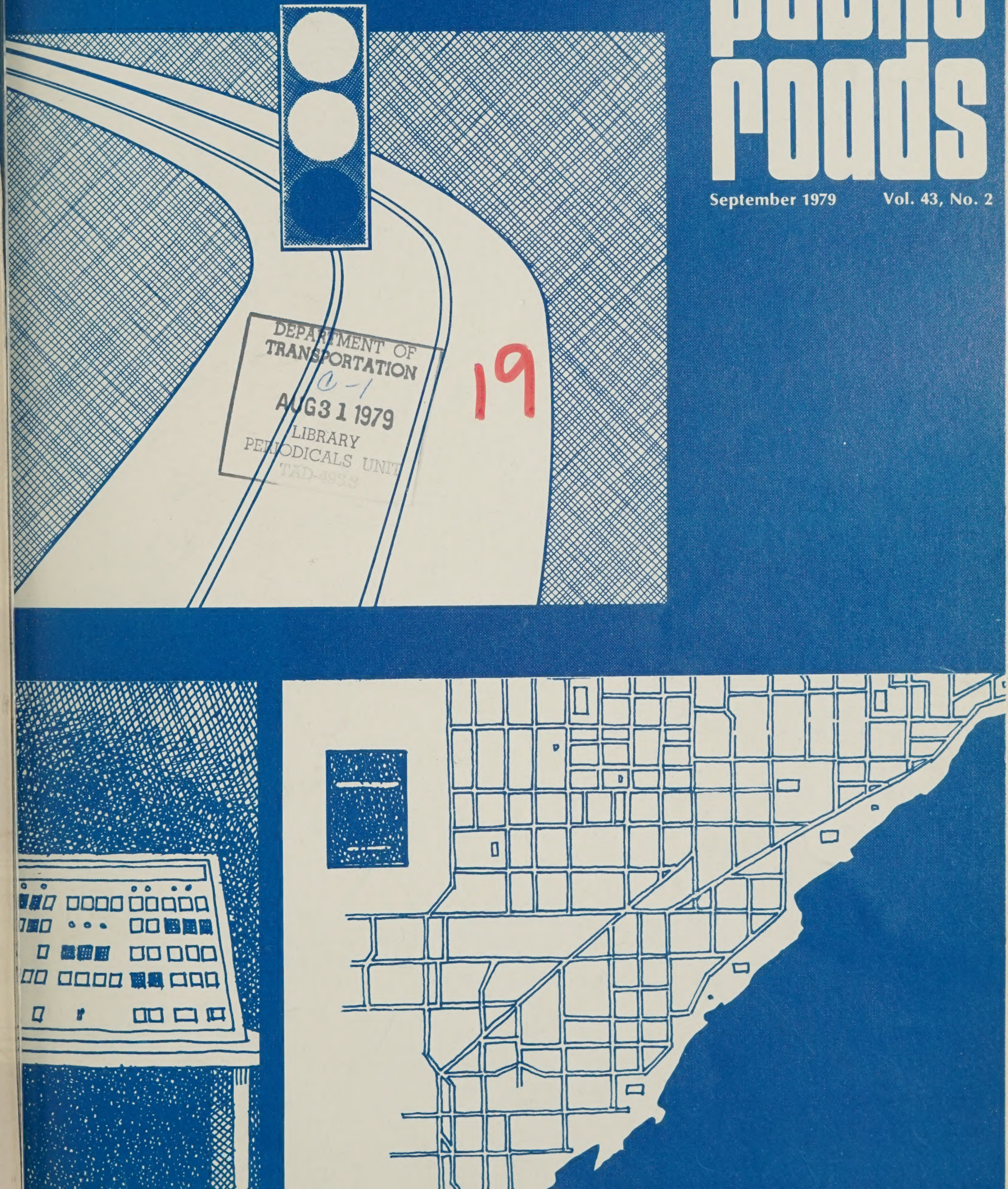

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COVER:
Artist's concept of components of a traffic control system.

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Traffic Control System X UTCS
Bus priority system
Research, Traffic   Traffic control
Traffic signals - Lights - Control

Development and Testing of Advanced Control Strategies in the Urban Traffic Control System

Authors   FORTRAN

by

John MacGowan and Iris J. Fullerton

U.S. Federal Highway Administration
(Pt. 1

This is the first in a series of articles that will trace the evolution and accomplishments of the Urban Traffic Control System (UTCS) research project. This series of articles will highlight the research activities undertaken in the past 10 years and will stress the potential application and anticipated benefits accruing from this major research project. This first article describes the development of offline signal timing programs, the provisions of the real-world fully instrumented test laboratory established in Washington, D.C., and the first generation control strategy. Future articles will present the technical accomplishments in the Bus Priority System (BPS), the second and third generation strategies, simulation model development, and vehicle detection.

Introduction

During the past two decades, the street systems in the Nation's urban areas have been increasingly plagued with traffic congestion, particularly during peak periods. The delays experienced as a result of this congestion are primarily associated with intersections where conflicting movements are controlled by traffic signals. Many of these signals are either inadequate or improperly operated for assigning green time and coordinating traffic movements through a network of intersections.

It was postulated that if the assignment of green time to conflicting flows and the coordination of signals could be optimized, an impressive improvement in the overall traffic flow could be realized. By 1967, significant advances had been made in traffic control equipment, communications and traffic surveillance techniques, and traffic control theory to warrant development of more sophisticated urban traffic control systems. Of particular note, technical advances in the state of the art included the use of digital computers to coordinate system functions, signal optimization techniques, and the application of electronic surveillance to gather information from the street system. These advances made it possible to collect, process, and analyze large quantities of traffic data, and to compute and display meaningful parameters to measure and evaluate system performance.

In response to the need to develop advanced operational control programs for computer-based control systems, which would result in a marked improvement in traffic flow, the "Development and Testing of Advanced Control Strategies in the Urban Traffic Control System" (UTCS) research project was initiated by the Traffic Systems Division, Office of Research, Federal Highway Administration (FHWA).

The project objective was later expanded to include development and testing of control strategies using simulation techniques; testing of the strategies in a real-life environment test facility in Washington, D.C.; development of traffic detection technology required by the strategies; and improvement of performance evaluation techniques for measuring the efficiency of the new strategies.

Since its inception, the UTCS project has achieved national recognition and promises to exert a far-reaching impact on the state of the art in urban traffic signal control. Most significantly, it has generated new knowledge and a reservoir of experience that can be directly applied by local traffic departments in planning digital computer-controlled traffic systems. By using the knowledge gained, municipalities and design consultants can confidently select the design and techniques most appropriate to local conditions.

Offline Signal Timing Programs

The SIGOP (SIGnal OPtimization) program was developed in 1966 for the FHWA and is considered by many to be the impetus for and forerunner of the UTCS project. SIGOP created an efficient and practical means of computing signal timing for an urban street grid network. In response to the need of city traffic engineers, it was designed to be run and implemented by staff personnel without outside consultant services. In addition, it provided a less expensive alternative to the manual development of timing for a network of traffic signals in an urban area. The evolution of the SIGOP program and the similar TRANSYT program developed in Great Britain are discussed below because of their role in the UTCS development process.

SIGOP structure

The SIGOP program consisted of six functional blocks. (*1*)[1] One of these, the OPTMIZ block, is the nucleus of SIGOP in determining optimal offset relationships. The algorithm depends on two variables: (1) Ideal offset as computed from speed, link length, and queue discharge time; and (2) link weighting, which can be either specified in the input data or calculated by SIGOP in direct proportion to the competing approach volume demands. The objective function to be minimized consists of the sum of the squares of the differences between the ideal offsets and the optimized offsets for each link in the form of a parabolic set. The optimization procedure identifies the set of actual offsets that minimizes the value of the objective function. The algorithm then uses a series of trials to select the minimum.

SIGOP pilot studies

After the basic research on the program was completed, the SIGOP implementation project was established by the FHWA Office of Traffic Operations. Six cities were selected from approximately 30 qualified candidates—San Antonion Texm; Kansas City, Mo.; Seattle, Wash.; Miami, Fla.; Indianapolis, Ind.; and Cincinnati, Ohio. These cities were asked to implement SIGOP in-house, without outside assistance, if possible. They were also asked to describe any problems encountered in implementing or utilizing the program and the corrective measures employed.

The only major complaints voiced by the pilot cities were the large volume of input data required and the man-hours required for data collection. Apparently, the major expense incurred in running the program was in data collection; the cost of data collection exceeded the cost of running and evaluating the program results.

Generally, the cities concluded that SIGOP was a useful tool and a viable means of resolving network timing problems. Even in Miami, where the signal settings generated by SIGOP did not prove to be quite as efficient as previous settings, the engineers praised SIGOP for providing signal settings in a single computer run that were comparable to those that the city had worked years to achieve.

Comparison of SIGOP and TRANSYT

While SIGOP was under development, a computer program known as TRANSYT (TRAffic Network StudY Tool) was being developed by the Road Research Laboratory of the Department of Environment, Great Britain, and a private firm. This program was designed to determine optimal traffic signal settings for fixed-time control on a network.

SIGOP and TRANSYT are probably the best known of all offline signal timing optimization programs. The similarity in objectives resulted in speculation about which program is better under what circumstances. Tests using the TRANSYT program were conducted in San Jose, Calif., and Glasgow, Scotland. A comparison of the TRANSYT and SIGOP test results indicated that in terms of traffic performance, both programs are about equal in developing signal timing plans. (*2*)

SIGOP and TRANSYT were also tested on the Washington, D.C., street network and the following conclusions were made:

- The development of signal timing plans can be effectively automated.

[1] Italic numbers in parentheses identify references on page 52.

• Because TRANSYT proved more effective than SIGOP for producing signal timing plans for Washington, D.C., TRANSYT should be used to generate UTCS first generation timing plans.

• Ongoing advanced control strategy development activities should be reevaluated to increase the effectiveness of optimization algorithms.

• A study effort should be initiated to improve SIGOP.

Development of SIGOP II

The recommendation to study and improve SIGOP led to the development of a new program, SIGOP II. (*3*) This program was intended to increase the original SIGOP signal timing effectiveness so that it generated signal timing at least as effective as that provided by TRANSYT, while requiring less computer time than TRANSYT.

A study of the interrelationships between the platoon structure of traffic flow and signal control in a fixed-time, cycle-based policy was formulated. Subsequently, a program developed under the third generation research tested the validity of this new procedure for creating optimum cycle-based signal timing plans. Encouraged by the test results, FHWA determined that a fully developed version of the new procedure could prove superior to the existing TRANSYT and SIGOP programs.

SIGOP II uses a two-part procedure to determine the optimum signal settings: Initial signal settings are determined using a somewhat coarse technique; then, an iterative search through the collection of possible signal settings is performed to locate the optimum settings.

In the iterative search, SIGOP II uses a traffic model to measure the equivalent delay in the network and then optimizes the signal timing (or minimizes the delay) by a gradient search technique. The equivalent delay consists of the actual delay experienced, the level of delay that would be equivalent to the vehicle stops encountered, and the amount of delay equivalent to the congestion arising from excessively long queues.

The three components—delay, stops, and queue length—are extracted from the flow model for each link. They are then summed into a network value called the objective function, whose form is as follows:

$$J = \sum_{i=1}^{n} W[(\text{delay on link i}) + K\,(\text{stops on link i}) + D_Q(\text{excess queue on link i})]$$

Where,

n = Number of links.
J = Equivalent delay.
W = Relative link weight, specified by the user.
K = Excess stops-to-delay equivalence factor.
D_Q = Excess queue-to-delay equivalence factor.

Upon completion of the initial testing of SIGOP II, the program was modified to improve the signal plans generated. SIGOP II is presently being analyzed and tested by the FHWA Implementation Division in the Office of Development.

Establishment of Washington, D.C., UTCS Test Facility

One of the earliest activities in the UTCS project was to develop a functional definition of a prototype traffic control system in enough detail to estimate costs, purchase and install equipment, and program the computer for basic online control. The objectives in designing such a test facility were to develop concepts for a computerized traffic control system that could readily implement experimental control strategies for approximately 100 signalized intersections in Washington, D.C.; define equipment configurations; generate installation and maintenance plans; and develop equipment specifications.

System design and specifications

The UTCS facility for Washington, D.C., was designed to interface with existing hardware, thereby minimizing changes to original equipment and providing a control technique consistent with adjacent equipment not under computer control. The original local-dial system remained intact and provided a standby mode for the computer-controlled system. In addition, the original three local-dial timing plans were duplicated within the computer-controlled system for use in checkout and data gathering.

Engineering plans and specifications for the Washington facility were completed in early 1970. Procurement and installation began in July of that year. (*4*) Four categories of equipment were specified—detectors, detector and controller communications, computer system, and displays and controls.

Detectors

The inductive loop detector, the magnetic detector, and the ultrasonic detector were initially considered for UTCS. Tests run on the loop detectors in the laboratory and on a vehicle test track provided quantitative data that showed that the accuracy limits for available detectors were compatible with UTCS requirements. (5) Loop detectors were selected on the basis of local ordinances and technical considerations.

Detector and controller communications

Several alternative communications techniques were evaluated to determine the most cost-effective configuration employing typical off-the-shelf equipment. The comparative analysis indicated that either a three-frequency division technique employing logic to double the channel capacity or an amplitude-modulated technique (tone on, tone off) would be suitable for UTCS. Consequently, the communication specification was written so that either of these two frequency division multiplexing techniques could be used.

Computer system

A wide range of system approaches and computer configurations, including both real-time and non-real-time computational capabilities, were considered in the computer system design. The primary real-time tasks encompassed traffic signal control, display control, and storage of data for experiment analysis. Non-real-time tasks included experiment analysis, simulation validation, and computer program alteration and debugging. Alternative system approaches for performing these non-real-time tasks included the following:

- Use of a remote computer, purchasing computer time as needed.
- Use of the onsite computer when the system was under standby control—at night or during periods of light traffic.
- Use of the onsite computer while performing the real-time traffic control operation.
- Use of two onsite computers, which could be used interchangeably for real-time and non-real-time tasks.

The process of selecting one approach from among the alternatives consisted of weighing the convenience of concurrent and onsite operation against higher cost. It was concluded that the computer system selected should be no more complex than necessary to perform the real-time tasks, but should be capable of performing experiment analysis tasks or program alterations during standby periods. The single computer approach was selected as the most cost-effective for the UTCS program. It could easily handle the input data processing task and was reliable enough to provide an adequate level of uptime for UTCS.

Displays and controls

Selection of system displays required extensive trade off analyses. Display requirements were developed for the monitoring and evaluation functions; table 1 shows how various types of displays would satisfy these requirements.

Elements for the displays and control console that were selected for UTCS are listed below:

- Traffic control panel.
- Alphanumeric cathode ray tube (CRT).
- Illuminated map display.
- Cross-reference directory.
- Keyboard/printer.
- Line printer.

System installation plan

Extensive surveys and studies were required to develop a street installation design that would yield maximum benefits when used in conjunction with UTCS software and central design concepts. Surveys and studies were conducted to evaluate existing controllers and controller subsystems to determine whether they should be utilized or replaced; select the intersections to be placed under computer control and which of these should be selected for critical intersection control (CIC), the type of three-dial controller, and the type of loop detector; determine the grouping (sections) of intersections and the number and locations of detectors; and improve intersection geometric design and install new signals.

In addition, geometric studies were conducted to determine parameters for generating timing plans in conjunction with offline plan generation programs. The communication cabling plan was also developed and curbside installation drawings were prepared.

Figure 1 shows the central equipmen and how it interfaces with the field equipment. Figure 2 shows the study/test area, the intersections under control, and the location of the control center. The computer, peripheral equipment, communications, and all other apparatus required to direct operations were housed in the traffi control center. Installation of equipment in the center was based i part on functional, structural, interconnection, and esthetic requirements; temperature and humidity requirements; and acoustical, lighting, and power requirements.

The 114-intersection facility began operating in November 1972. In mid-1974, the test network was expanded to 200 signalized intersections on the Washington, D.C., street network (fig. 2). Identical technology, procedures, and equipment were employed in this expansion.

System Operation and Maintenance

Because the test areas used for research and development of traffic control strategies were also operating systems for Washington, D.C., the system operations and maintenance included a number of unusual tasks. (6) For example, in addition to normal operations, the central operator had to provide software and analytical support and was involved in hardware installation, software research and development, acceptance tests, system evaluations, and failure diagnoses. During acceptance testing, the central operator activated the system for observation by FHWA and Washington, D.C., personnel. When activated, the computer took operational control of each intersection and adjusted signal and section parameters. The program was accepted when performance of all components conformed to specifications. Responsibility for central and field maintenance was jointly vested in a contractor's field crew, subcontractors, and the Washington, D.C., maintenance crew.

Operation

Although research and development work was primarily performed by systems analysts and programers, the central operator was usually present to run a strategy under development, study it for faults, and stop the system. Software personnel would then debug the program. Subsequently, the central operator would prepare update cards, run an

Table 1.—Candidate displays

Function	Display/control type						
	Alphanumeric CRT	Illuminated map display	Controller/detector status board	Keyboard/printer	Traffic control panel	Line printer	Graphic CRT
Implement strategic control							
System operation and control				X	X		
Monitor operation	X		X	X		X	X
Program query and control				X			X
Computer control and debugging				X			X
Malfunction monitoring							
Controller failure display	X		X	X		X	X
Controller operations related to geographical location		X					X
Detector failure display	X		X	X		X	X
System adaptation				X	X	X	
Permanent records				X		X	
Traffic monitoring							
System status	X		X	X		X	X
Permanent records				X		X	
Traffic variables:							
Geographical		X					X
Alphanumerical	X		X	X		X	
System adaptation				X	X		
Evaluation							
System status	X		X	X		X	X
Traffic variables:							
Geographical		X					X
Alphanumerical	X		X	X		X	
Offline functions							
Data readout				X		X	X
Computer control				X			X

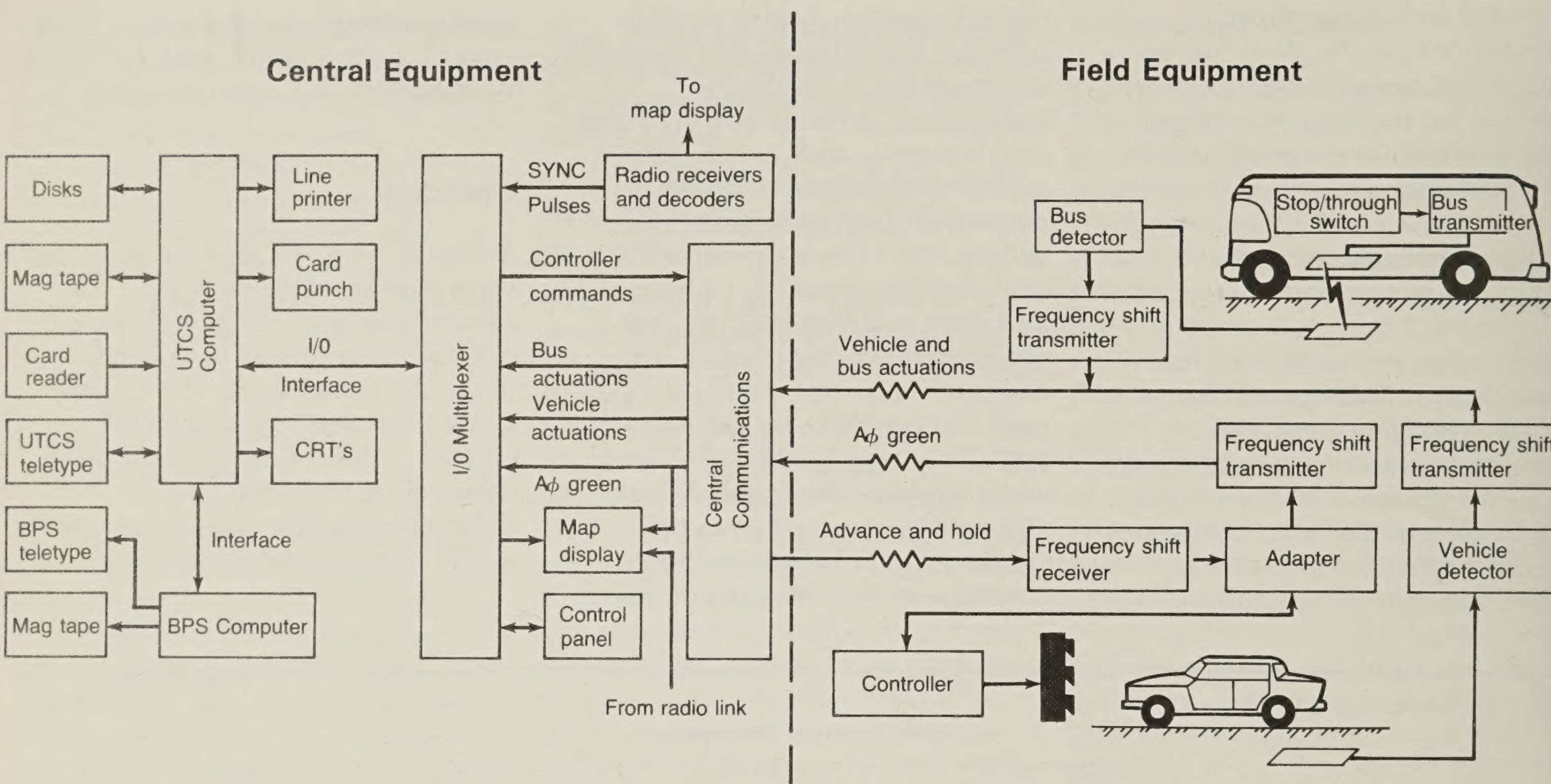

Figure 1.—Block diagram of UTCS/BPS equipment.

update on the system, and reactivate it to check whether the problems were eliminated.

During the 68 months of system operations, the central control equipment was used for each of the research and development activities as follows: Hardware installation—25 percent; software research and development—54 percent; system acceptances—1 percent; system evaluations—10 percent; and system operation—10 percent.[2]

Operations documentation

Substantial documentation of UTCS operations was maintained. This material was used by central and field personnel for daily operation and included tables, maps, charts, printouts, and logs. Essentially, the central operator maintained the following logs.

The *system operations log* contained necessary information on changes in system status and served as a daily operations diary. The date, time, weather, operator on duty, tasks being performed, and system status and performance were recorded at the beginning of each shift and when system status changed.

The *computer maintenance log* recorded computer failures, corrective action, and preventative maintenance.

The *field equipment status log* recorded damaged loop detectors and arrangements made for their repair.

The *software log* kept track of signal timing changes and updates to the data base.

[2]The considerable amount of software development work which took place during night shift and weekend hours is not represented in these figures because accurate records were not kept during these periods.

Maintenance

A planned maintenance program was established to achieve a high level of system operability and reliability and to minimize required spare parts and components. The maintenance of the UTCS facility was the responsibility of a single system management contractor. The computer manufacturer was subcontracted to provide computer and peripheral equipment maintenance. The system management contractor maintained noncomputer central equipment and field equipment with its own personnel. The major functions and maintenance responsibilities under the supervision of the system contractor are shown in figure 3. The maintenance services provided by the various subcontractors are summarized in table 2.

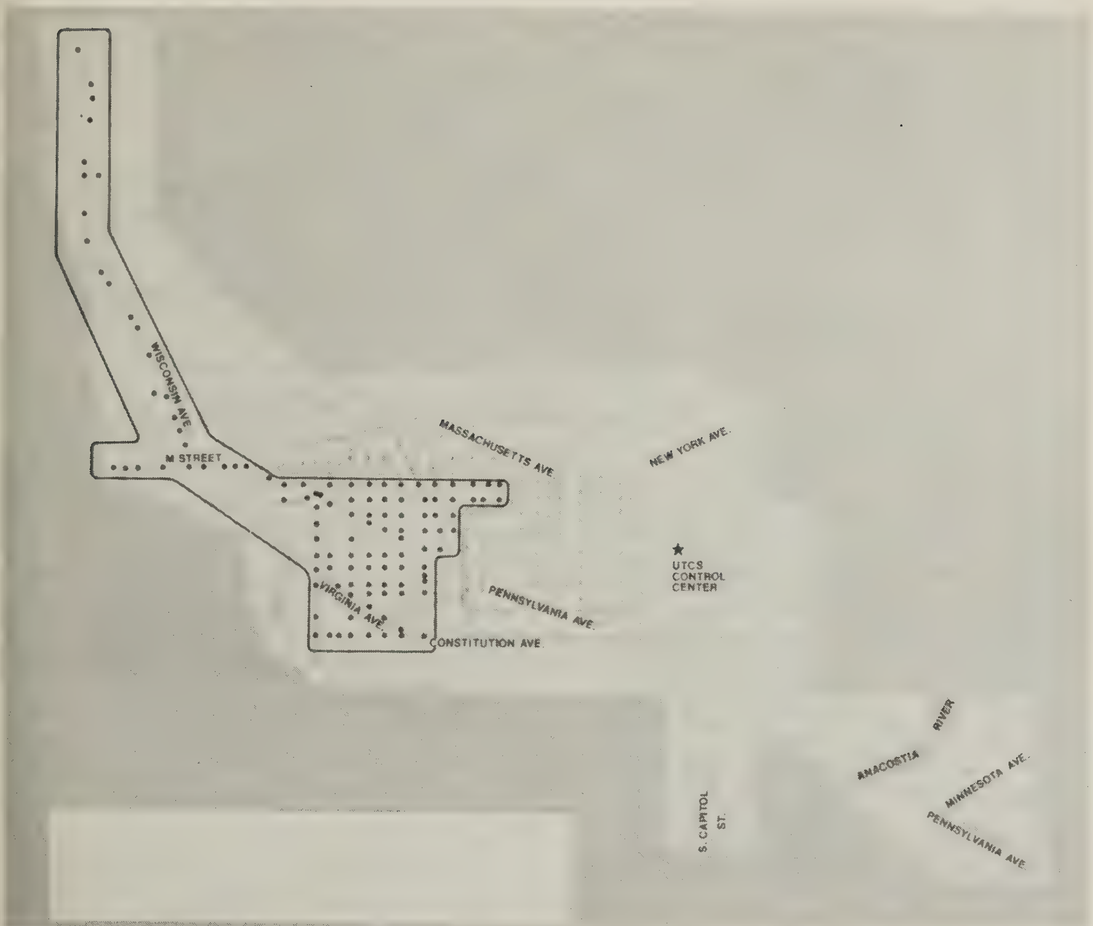

Figure 2.—UTCS network configuration.

Spare parts plan

During the initial implementation phase, the installation contractor serviced all UTCS field equipment while the Washington, D.C., shop continued to maintain controllers, detectors associated with the semiactuated controllers, radio control equipment, and existing auxiliary devices. This provided a period for establishing actual cost data and personnel requirements and allowed for gradual Washington, D.C., organizational buildup and a new equipment training period. Using trained personnel for maintaining existing equipment also reduced the initial manpower costs.

The recurring costs associated with facility maintenance are given in table 3. The spares complement purchased for the initial UTCS installation was approximately 10 percent for all items. The exceptions were that no computer or peripheral spares were purchased because the manufacturer's maintenance agreements covered those items. Vehicle detectors and communications equipment were purchased for operation on specific frequencies. A broad spectrum of spares was purchased which included all required frequencies and made up the 10 percent spares total. Additional spares for the expanded system were held to 5 percent.

During the initial maintenance period, it was determined that a lower spares inventory could be maintained by purchasing specific electronic components. This provided the capability of modifying vehicle detector and communications equipment for various operating frequencies. Both the field maintenance vehicle and repair depot storage requirements and related costs were greatly reduced by this method.

Equipment reliability

System reliability of the UTCS was examined during two overlapping periods. The first period began June 15, 1971, with the first intersection's signoff and extended to July 1, 1975—approximately 48 months. The second period began on November 1, 1972, the date of system acceptance, and continued for 32 months until July 1, 1975.

The equipment complement was divided into subsystems and analyzed to estimate each subsystem's reliability. Table 4 shows the results of this analysis, which provided conservative estimates of system component reliability in terms of mean time between failures.

First Generation Control Software

While specifications for the UTCS facility were being developed, the first generation control system (1-GC) was being designed and the programing specifications were being prepared. The actual programing was performed during facility installation. In November 1972 when the facility began operation, the 1-GC software was first used to control traffic in Washington, D.C. (*7*)

The 1-GC system consisted of three functional program modules: Service routines for data handling, "housekeeping" routines for functions not associated with data handling or traffic control, and the strategy routines for traffic control.

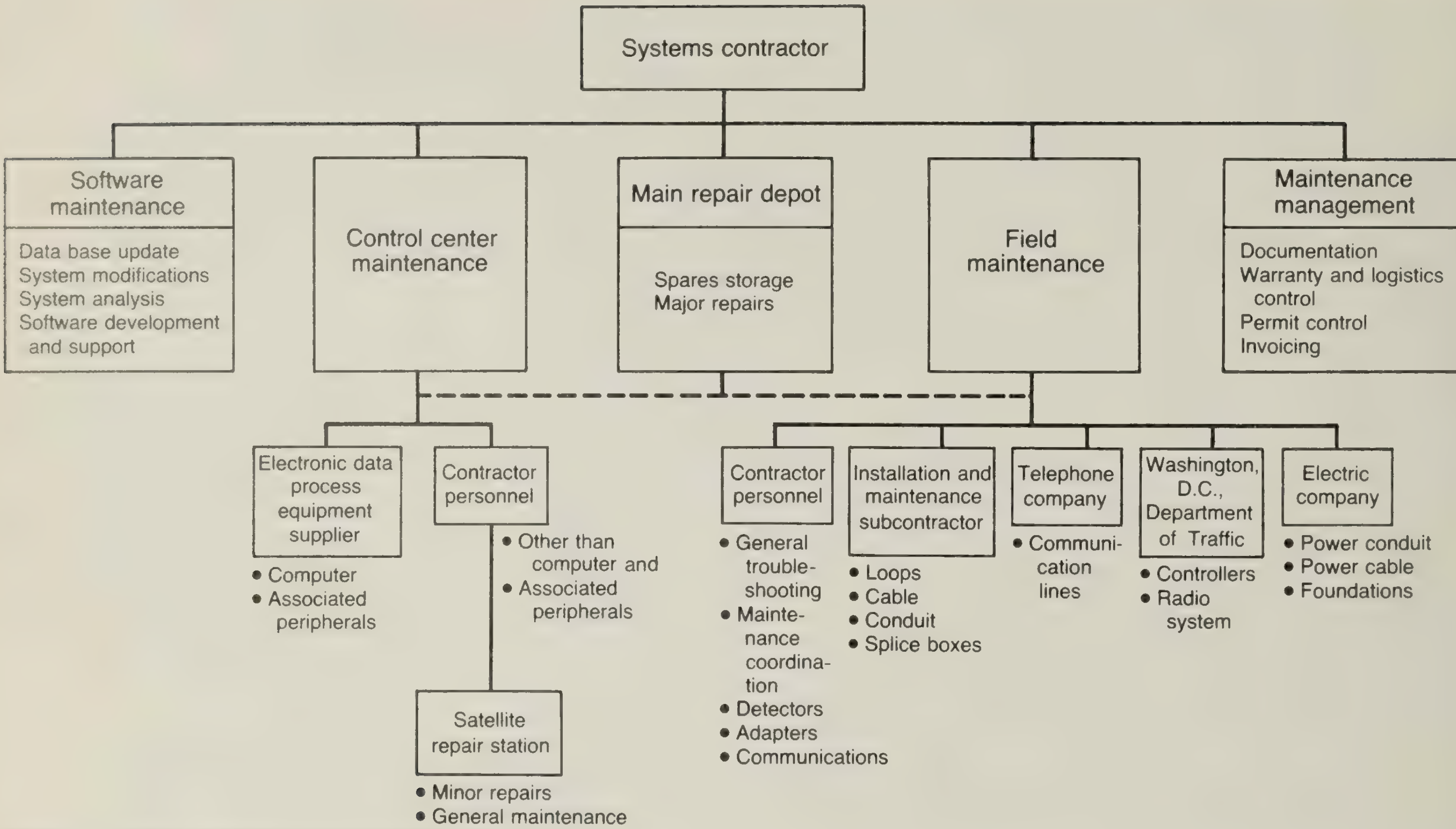

Figure 3.—System maintenance responsibilities.

Data handling functions

The detector data generated by the UTCS program were used for report generation and system analysis. The data were stored in appropriate time increments and were used for developing optimum controller timing plans, evaluating system performance in terms of measures of effectiveness (MOE's) and generating 15-minute and 24-hour reports. Seven MOE's available in UTCS are volume, occupancy, speed, queues, stops, delay, and travel time.

The detector-generated data were also used automatically online to select appropriate cycles, splits, and offsets. The data generated information signals for the map display and information for the CRT alphanumeric display.
Vehicle-by-vehicle pulses generated by the detectors were recorded to enable investigators to recreate the certain traffic flow conditions for future study.

Housekeeping functions

In addition to the data manipulation for control purposes, the 1-GC required a number of supportive routines, many of which were also required for second and third

Table 2.—Maintenance subcontractors' responsibilities

Subcontractor	Responsibility
Telephone company [1]	Line lease and maintenance
Electric company [1]	Conduit, cable, manhole maintenance and controller cabinet, or foundation replacement
Computer data services [1]	Computer and peripheral maintenance
Local electric contractors [1 2]	Vehicle and bus loop maintenance, splice box replacement, loop lead in cable replacement
CRT manufacturer	CRT maintenance
Teletype manufacturer	Teletype maintenance
Fire protection system supplier	Fire protection
Washington Metropolitan Area Transit Authority	Bus mounted, BPS component maintenance

[1] Also used for system installation, modifications, and expansion.
[2] Each used during different periods.

generation systems. These "housekeeping" functions were associated primarily with scheduling the execution of routines, generating displays, processing operator commands, and detecting malfunctions.

Control strategy functions

The first generation software uses prestored timing plans developed offline and based on previously generated traffic data. The system can store up to 40 timing plans per section. A section is a group of local controllers operating in the same mode and switching simultaneously from one timing plan to another. Timing plan selection of a desired set of timing parameters (cycle, split, offset) may be based on time of day, automatic response to traffic conditions, or operator choice. The mode of timing plan selection is determined by the operator. Four modes of area control are available in 1-GC:

- Standby (STBY)—Controllers are transferred from computer to local backup system control. The UTCS computer remains online during standby operation and limited system functions are available. Volume, speed, and occupancy parameters are displayed and recorded for all links.

- Time of Day (TOD)—Timing plans are automatically selected on a time-of-day and day-of-week basis. Change to the selected plan is accommodated with a time resolution of 15 minutes for each section in the system. This mode may be selected by the operator on a system basis (all sections), or on an individual section basis.

- Traffic Responsive (TRSP)—This mode automatically matches the timing plan best suited for existing traffic conditions. Very recent volume and occupancy data are compared with volume and occupancy characteristics of the various stored patterns. Each history for a section of controllers is uniquely related to a set of control parameters (controller timing plans). The control plan related to the best data-history match is selected from the available timing plans.

- Manual (MAN)—This mode is used primarily for handling unusual traffic conditions not provided for when operating under the TOD and TRSP modes. The MAN mode may be used during checkout of traffic-responsive control patterns or to provide rapid response to unusual traffic conditions resulting from parades, concerts, or sporting events.

Critical intersection control

Critical intersections which saturate frequently may have their split adjusted to be directly proportional to the traffic demand for green time computed for their associated links. The cycle splits for these intersections are then changed on a once-per-cycle basis as a function of the computed values of queues and volume on these links. This critical

Table 3.—Maintenance costs

Item	System: Initial 114 intersections 12/72–7/75	System: Expanded 200 intersections 2/75–7/76
Loop repairs[1]	$30,126	$6,354
Telephone line leases[2]	30,516	70,200
Computer services maintenance	91,428	38,856
Teletypewriters	750	300
CRT's	800	400
Fire protection	100	100
Cable	400	100
Splice boxes	150	75
Central supplies[3]	750	300
Consumables[4]	4,980	1,992
Components[5]	1,500	600
Vendor repair items	800	300
Total	$162,300	$119,577
Average monthly cost	5,410	7,034

[1] Initial system: 19 splice box replacements, splices, and 997 m of saw cut for vehicle and bus loop repair. Expanded system: 9 splice box replacements, splices, and 392 m of saw cut. Three subcontractors were engaged during the 42-month period at a varying cost per metre of saw cut.
[2] $672/month until 5/75, approximately $5,850/month thereafter.
[3] Such as control panel and map display lamps and line printer and teletype ribbons.
[4] Such as fuses, electrical tape, connecters, and wire.
[5] Such as transistors, transformers, resistors, and capacitors.

Table 4.—Equipment reliability

System element	Total number	Number failed	Hours of operation	MTBF[1]	MTBF for one such subsystem	Total MTBF for all such subsystems in the UTCS facility
				Hours	*Hours*	*Hours*
Vehicle detector system						
Loop	497	5	29,352	2,918,000	—	—
Detector	497	83	29,352	176,000	—	—
2 FSK[2] transmitter	497	52	29,352	280,500	—	—
Total	—	—	—	—	105,300	210
Controller subsystem						
Controller[3]	111	16	29,352	203,600	—	—
Adapter	111	29	29,352	112,300	—	—
2 FSK transmitter (for Main Street green)	111	11	29,352	296,200	—	—
3 FSK receiver (for hold/advance pulse)	111	16	29,352	203,600	—	—
Total	—	—	—	—	45,500	410
Power supply subsystem	111	3	29,352	1,086,000	1,086,000	9,800
Overall field system	—	—	—	—	24,000	135
Central communications subsystem						
Dual amplifier	42	1	8,016	8,016	—	—
2 FSK receiver	794	10	8,016	802	—	—
3 FSK transmitter	201	3	8,016	267	—	—
Power supply	13	3	8,016	267	—	—
Total	—	—	—	—	—	501
Central processor and peripheral subsystem						
Central processing unit	1	6	8,016	1,336	—	—
Rapid access disk	2	12	8,016	1,336	—	—
Computer interface unit	1	2	8,016	4,008	—	—
Magnetic tape unit	3	17	8,016	235	—	—
Line printer	1	8	8,016	1,002	—	—
Card reader	1	5	8,016	1,603	—	—
Keyboard printer	1	1	8,016	8,016	—	—
Total	—	—	—	—	—	157
Critical computer subsystem[4]	1	21	8,016	382	—	—
Display subsystem						
Control panel	1	1	8,016	8,016	—	—
CRT	2	7	8,016	2,004	—	—
Map	1	8	8,016	1,145	—	—
Total	—	—	—	—	—	501
Telephone line subsystem[5]	79	77	23,328	303	—	303
Overall system	—	—	—	—	—	49

[1] MTBF = Mean time between failures

[2] FSK = Frequency shift key

[3] This is extrapolated from an observed failure rate of four controllers in a 261-day period (9/12/72–6/7/73). Controller maintenance after installation was the responsibility of Washington, D.C., personnel. Consequently, records were kept separately from UTCS records and in a different format.

[4] Failures of the line printer, card reader, or magnetic tape unit would not generally cause a catastrophic system failure. The central processor could continue to operate as long as the failed equipment was not called upon specifically to function. However, the subsystem composed of central processor, rapid access disk memory, computer interface unit, and keyboard printer was critical. Failure of any of these elements would immediately fail the system. In the case of the teletype, if the system had been brought up, the failure would not occur until other equipment in the system failed and the system made an attempt to report it through the teletype.

[5] Frequently, the same line was visited several times within a short time period raising the possibility that the underlying cause of all the visits was identical. Therefore, the assumption is made here that only failure reports separated by more than a reasonable time will be counted for any one telephone pair.

intersection control (CIC) is used to fine tune the split allocated to each approach to the intersection, based upon fluctuations in local traffic demand. Thus, intersection control works in conjunction with and supplements area control operation.

Semiactuated control

Semiactuated intersections continue to operate in their local control mode with regard to side street demand; however, the computer synchronizes controller response to the main street with respect to the computer-timed cycle length. This maintains the proper offset relationship with other controllers in the section.

Transitions

It is critical that transitions between signal timing plans with differing cycles, offsets, and splits be performed smoothly to avoid traffic disruption during the transition period. As a result, the 1-GC software contains logic to change the magnitude of parameters over a number of signal cycles.

Priority organization of 1-GC

The 1-GC is always processing functions in one of three priority levels, otherwise the computer is idling. These priority levels correspond to functions performed every 1/32 second, every 1/2 second, and at lower frequencies or on demand.

The first priority routines involve functions directly associated with the measurement of vehicular traffic and program sequencing control. The second priority routines are primarily associated with controller commands, error checking, and data evaluation. Central to these second priority functions is a sequence of routines performed for each controller. These routines sequence the controller through its cycle, place it in online or offline, and adjust splits or offset in accordance with traffic volumes or operator commands. Other functions in the second priority routines are error checking of each controller, controller cycle timekeeping, data acquisition, and operation of semiactuated controllers. The third priority routines provide the display and operator interface functions. Data evaluation functions for which time is not critical are also performed.

Testing

After a period of debugging and integration, acceptance tests were conducted to demonstrate UTCS readiness for system operation using 1-GC software. Field and central observation tests were conducted to verify the following functions:

- Computer-controlled TOD operation for single and multicontroller operation.
- MOE calculations (volume, speed, queue, stops, delay, occupancy).
- CIC operations.
- TRSP operations.

Sensitivity tests of certain parameters were also conducted on the 1-GC system. The first such test examined the filtering time constant used in smoothing the various MOE's. Another test examined the effects of using the harmonic rather than the arithmetic mean of spot speeds. Finally, pattern matching tests were conducted in-house to fix the value of certain parameters used in the algorithm.

Conversion to FORTRAN

One activity not conducted by the Office of Research but which was closely alined with the UTCS project was the translation of the 1-GC software into FORTRAN.

The 1-GC software was initially developed for UTCS in the specific assembly language of the system's computer. In this way the computer's time and memory were utilized to provide the maximum degree of experimental flexibility. After the program proved operational, the Office of Traffic Operations authorized the conversion of 1-GC into the FORTRAN IV language to facilitate application of UTCS concepts by other cities with other computer systems. The 1-GC FORTRAN software provides the following advantages:

- Adaptability to a variety of computers, which avoids restricting or limiting the city to any particular equipment line.
- The modular design facilitates application to different systems and networks.
- Relative ease of modification and program updating.

A number of features from the original program have been modified so that the converted program will be better suited to a purely operational system configuration. In particular, CIC green demand calculation was based on volume and occupancy, and certain parameters associated with queue determination were eliminated.

Use of the FORTRAN software package significantly reduced programing difficulty. Some programing was required, however, for use with different computers and peripherals, for addition or deletion of particular functions, and for establishment of the specific data base.

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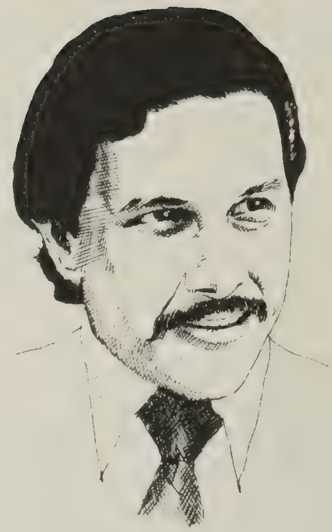
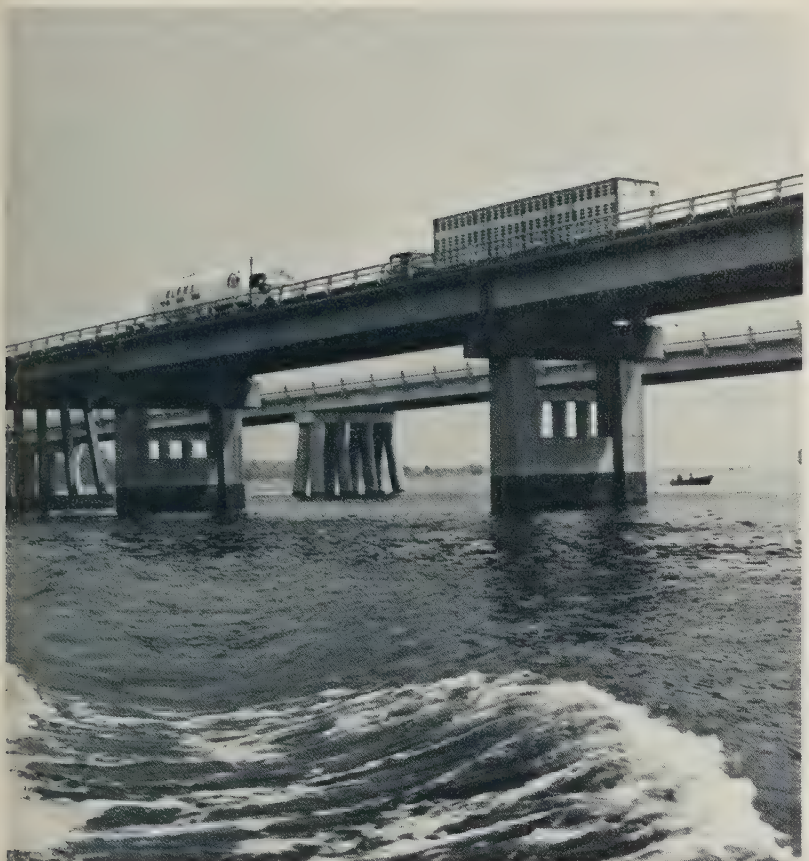
[3] Reports with PB numbers are available from the National Technical Information Service, 5285 Port Royal Rd., Springfield, Va. 22161.

John MacGowan is a highway research engineer in the Traffic Systems Division, Office of Research, Federal Highway Administration. He has been with FHWA since 1970, first as a staff member on the UTCS research team and then as the project manager of FCP Project 2B "Development and Testing of Advanced Control Strategies in the Urban Traffic Control System." Mr. MacGowan is currently the project manager of FCP Project 2M "Arterial Flow and Control." This research concerns arterial capacity, signal control for arterials, and bus signal priority systems.



Iris J. Fullerton is a Senior Public Systems Specialist for JHK & Associates. She specializes in research synthesis, systems evaluation, engineering training, and the communication and implementation of applied research. She is currently principal investigator for the FHWA project "Roadway Delineation Practices" and is participating in the National Cooperative Highway Research Program (NCHRP) Project 3-28 to develop an improved Highway Capacity Manual. Ms. Fullerton participated in the evaluation of UTCS control strategies and NCHRP Project 3-18(3) "Cost Effectiveness Methodology for Evaluation of Signalized Street Network Surveillance and Control Systems." She coauthored *Urban Street Design,* the student textbook for the FHWA's Urban Transportation Operations Training Program.

American Association of State Highway and Transportation Officials



Highway Bridge Loadings

by
Charles F. Galambos

This article expresses the views of a researcher, not of a bridge design engineer, and compares the American Association of State Highway and Transportation Officials (AASHTO) design live loadings for bridges with various other loading situations. Only external loads from traffic are considered. Comparisons are made with some European design live loads, native and foreign legal loads, normal permit overloads, and abnormal permit loads. The results of a bridge load rating exercise are presented. Actual bridge load histograms are given, as well as a comprehensive histogram based on the national loadometer surveys for 1970. Fatigue loadings and damage are discussed in light of actual and design loadings.

Introduction

As a result of trucking industry demands for higher allowable loads and increased awareness of the deteriorated state of many bridges, the adequacy of present design loadings was a topic of considerable discussion at the annual regional meetings of the American Association of State Highway and Transportation Officials (AASHTO) Bridge Subcommittee. It is estimated that over 100,000 of the 600,000 highway bridges on all U.S. road systems are structurally inadequate or obsolete and should be replaced. Some of these are being replaced through a special bridge replacement program, and many more will be replaced with funds made available from the passage of the 1978 Surface Transportation Act. This Act makes available to the States, for the next 4 years respectively, \$0.9, \$1.1, \$1.3, and \$0.9 billion, expressly for bridge replacement. Bridge load design practices and philosophies are therefore being examined to discover how they may be improved.

Design Live Loads

The majority of U.S. highway bridges are designed according to the "Standard Specifications for Highway Bridges" adopted by AASHTO. (*1*)[1] These specifications prescribe two forms of design live loads—a uniform load per linear metre of load lane and variations of axle loads in two standard trucks. The uniform load is supplemented by one concentrated load (or two for continuous spans), which varies for moment or shear. The axle spacings of the design truck can be varied to produce maximum stress effects. The heaviest loadings

[1] Italic numbers in parentheses identify references on page 60.

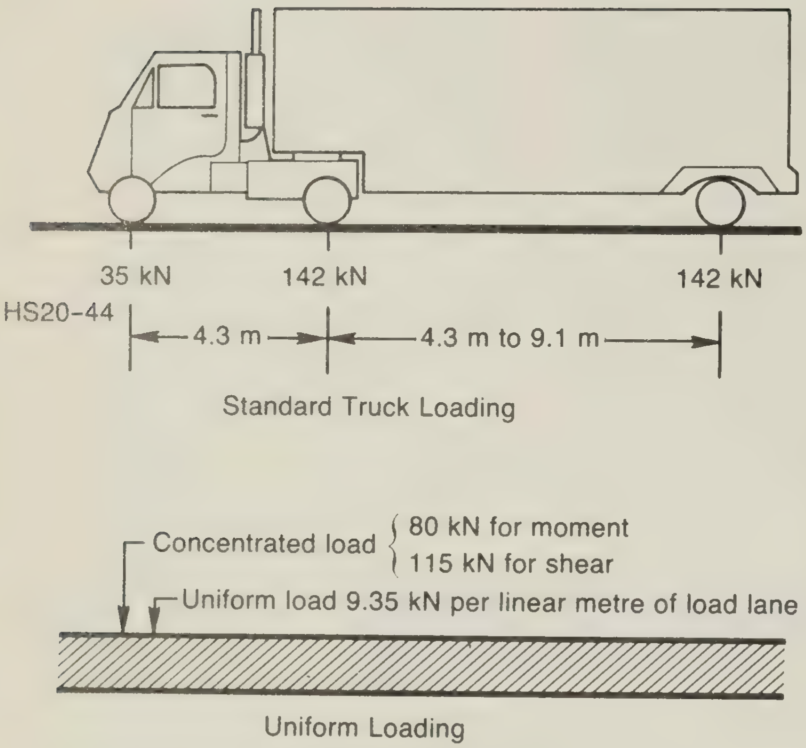


Figure 1.—AASHTO bridge design loads.

are shown in figure 1. For Interstate highway bridges, there is an alternate loading of two axles 1.2 m apart, with each axle weighing 106.8 kN. The loading which produces the maximum effect—lane load, truck load, or alternate load—should be used in any specific case. These loadings, presently designated HS20–44, have not been substantially changed since 1944.

The AASHTO design live loads are compared with those used in several other countries in figure 2. The comparison is made by bending moment calculations and accounting for impact factors and the respective allowable stress used in each country. A simply supported bridge was used, and the design vehicles were approximated by a conversion to a uniform distributed load spread out over 10 metres. The calculated values are only of theoretical interest; they cannot be considered as practical allowable values for the spans considered, but they are a good indication of the relative ultimate capacity of the structure. No lateral load reduction for more than one lane loaded was allowed for the comparison.

Figure 2 shows that there is considerable variation between countries and that the lowest design loading is the AASHTO loading. European design loads are very heavy compared to the AASHTO loads. Europeans tend to design for longer life (120 years in the United Kingdom). The transfer of heavy military loads and other occasional, extremely heavy loads, such as power plant generators, is considered in their designs.

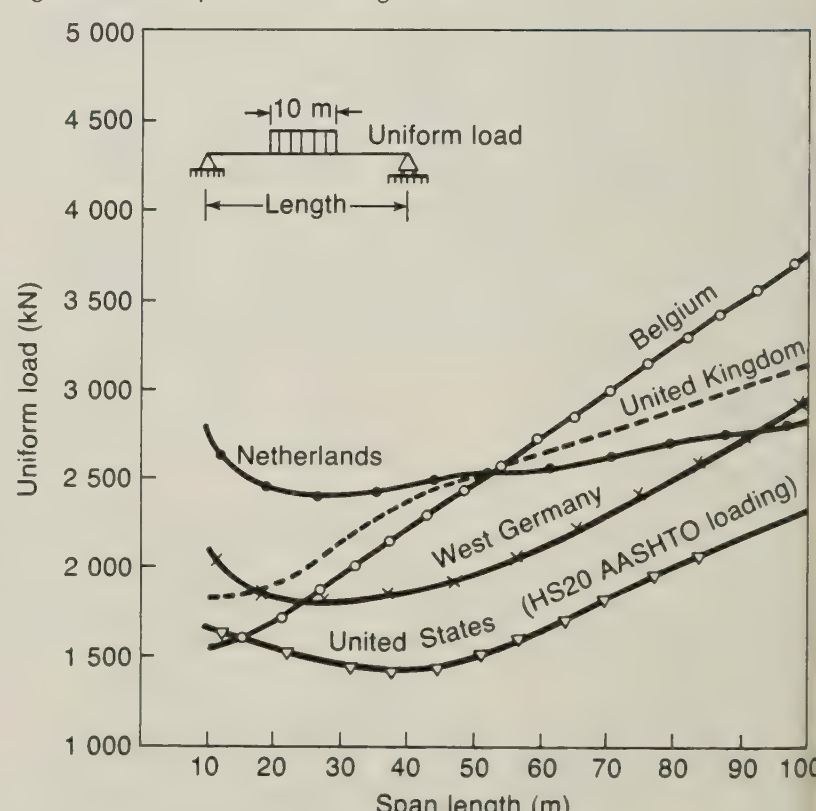
Legal Loads

Like design loads, legally allowed loads vary from State to State in the United States and also differ a great deal in the European countries. A tabulation of axle loads (single and tandem) and gross loads for a five-axle tractor-trailer combination is shown in table 1.

Except for a few countries (Belgium, France, Italy, and Spain), the allowable single axle and tandem axle loads are not too different from the AASHTO allowables: 98 kN versus 89 kN, 156 kN versus 151 kN. The five-axle vehicle allowable gross loads are somewhat greater in Europe, an average of 394 kN versus 338 kN, or about 15 percent higher than the AASHTO allowable.

A comparison of table 1 with figure 2 shows that U.S. bridges are designed to carry a slightly smaller load than what they are allowed to carry, 320 kN versus 338 kN; in

Figure 2.—Comparison of design loads.



most European countries the design loads are much higher than the allowable loads.

Allowable Overloads or Permit Loads

Occasionally, a load that is heavier than the legal loads discussed above must be moved. The question then arises as to how heavy can such a load be and still not damage a specific bridge. A distinction is made between "normal overloads" and "exceptional overloads." In the United States these stresses are recognized by two ratings—inventory and operating. Because of certain conservatism in design, construction, and material properties and the improbability that two or more heavy vehicles will precisely meet at a critical section of a specific bridge, a certain amount of overloading will not harm the bridges. (The possibility of increased fatigue damage is not included in this discussion.) In the United States, this amount is based on a stress below .55 σ yield. In several European countries, a specific weight has been assigned as the upper limit of "normal," unrestricted overloads. Total loads or gross vehicle loads are shown in figure 3. Several of the values shown are also the legal limits for a five-axle vehicle.

Most of the countries also have axle load limitations. A permit is required to operate such vehicles. Some of the permits are for single passages only, but many of them (such as permits for construction equipment) are for a season or a year at a time. Usually travel speeds are not restricted and the vehicles mix in with the traffic stream. It is implied that damage does not result from an unlimited number of such passages.

The "exceptional" overloads are seldom allowed and are carefully controlled. Vehicles carrying such loads must be slowly escorted along prescribed paths across a bridge and be the only vehicle on the span. In the United States the load allowed is derived from the allowable (operating) stress not beyond .75 σ yield. In other countries careful stress calculations for "exceptional" overloads are also made. In Belgium and France actual upper gross load limits exist (3 523 kN and 3 914 kN, respectively) along with certain axle load limitations. It is recognized that these monstrous loads need very special vehicles for safe transport across a structure.

Load Rating

A good inventory and estimate of the present live load capacity of highway bridges is desirable. Rules and regulations for compiling an inventory and making load ratings of bridges have been issued in the United States by the Federal Highway Administration for the Federal Aid Highway Systems. (2) Much of the load rating process involves engineering judgment. Substantial differences in load-carrying capacity can result for the same structure depending on who looks at it, what calculation methods are used, and how lateral load

Figure 3.—Allowable "normal" overloads.

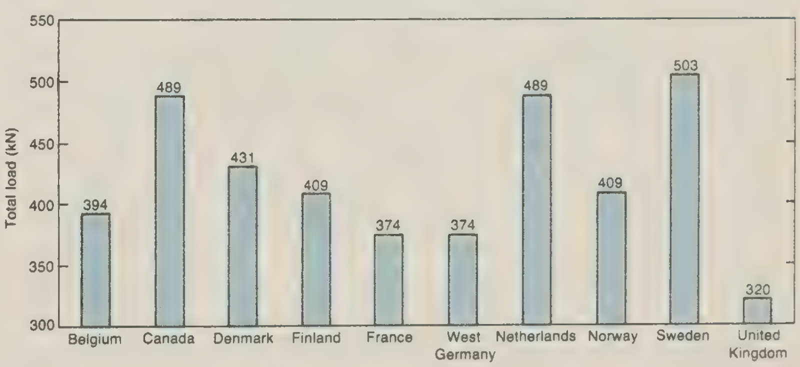


Table 1.—Maximum legal loads of freight vehicles[1]

Vehicle Type	Belgium	Canada	Denmark	Finland	France	West Germany	Italy	Netherlands	Norway	Spain	Sweden	Switzerland	United Kingdom	United States AASHTO 74	United States Highest state	United States Lowest state
	kN	kN	kN	kN	kN	kN	kN	kN	kN	kN	kN	kN	kN	kN	kN	kN
Single axle	129	98	98	98	129	98	116	98	98	129	98	98	98	89	107	80
Tandem axle	196	196	156	156	205	156	187	156	156	205	156	138	200	151	178	129
Gross five-axle	374	489	431	351	374	374	431	431	382	374	—	—	320	338	356	316

[1] Because of statutory enforcement tolerances, the values shown do not include a slight increase in loads allowed in some jurisdictions.

distribution, impact, effect of corrosion, settlement, scour, and material strengths are treated.

To illustrate possible variations in load ratings an experiment was performed by a committee of the Organization for Economic Cooperation and Development (OECD).[2] Representatives from each country involved in the experiment were given design drawings and material properties (yield strength and ultimate strength) of a specific bridge and asked to find the live load that a five-axle tractor-trailer combination (fig. 4) could carry. The Canadian Standards Association, Norway, the Province of Ontario, the United Kingdom, and the United States submitted answers they thought to be prudent and reasonable. The bridge was a 21.3 m simple span, noncomposite steel beam and concrete slab structure. (*3*) The answers, presented in table 2, vary from a low gross load of 423 kN for the vehicle to a high of 1 557 kN.

Differences result from four general areas of variations: Inventory ratings versus operating ratings and working stress versus load factor calculations. In addition, the variations are caused by philosophical differences about impact factors, load distribution, live load-dead load ratios, and what fraction of the yield or fully plastic moment seems reasonable to use.

Because there can be such a variation in even as simple and common a structure as a 21.3 m beam and slab bridge, many engineers feel that the only way to rate a bridge is to actually test it with a moving test load. Properly conducted load tests with realistic vehicles can be of great help in assessing the load-carrying capacity of a structure. However, such tests must be well planned and executed, with strain and deflection gages placed at major points. The tests determine the actual lateral load distribution; the effect of bracing members; deck behavior; joint behavior (pinned, frozen, or intermittently free); the degree of composite action; the amount of help received from curbs, sidewalks, and railings; the impact factors; vibration characteristics; and other actual bridge behavior. On older bridges where no design or construction plans exist, load tests can determine whether normal truck traffic can continue to use the bridge or whether some load restriction has to be imposed. (*4*)

Load tests, however, do not tell everything about a structure. The extent of fatigue cracking, for example,

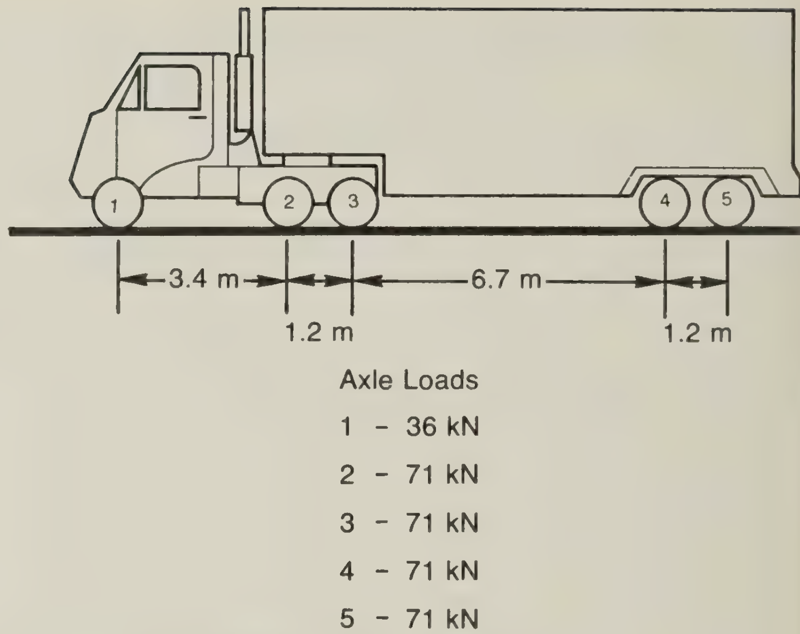
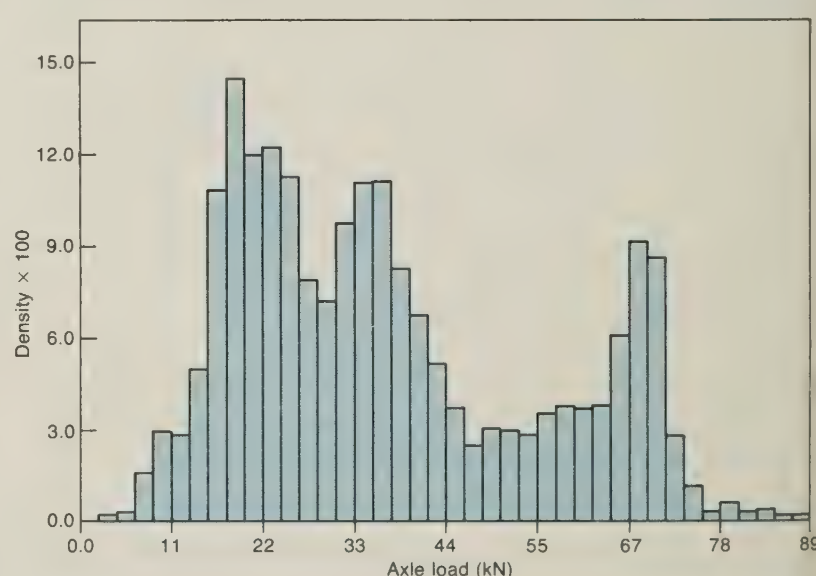


Figure 4.—Five-axle rating example vehicle.

Figure 5.—Histogram for axle load, all axles, all trucks, Shaffer Creek Bridge, 1968. (Number of trucks: 3,540.)



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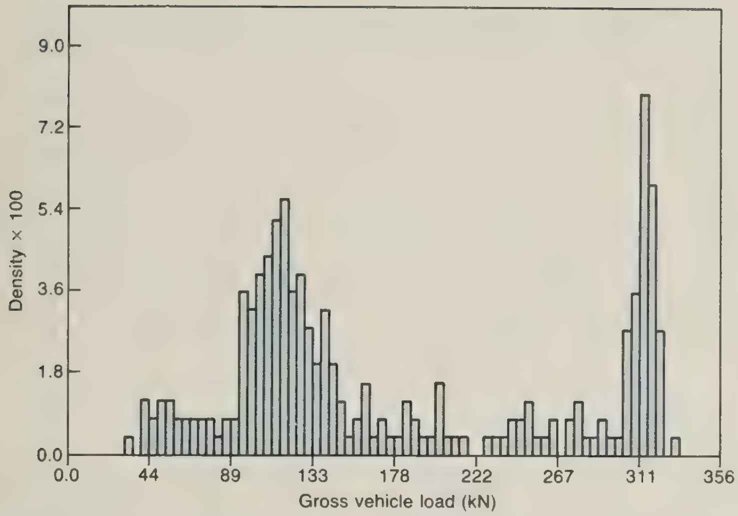


Figure 6.—Histogram for gross vehicle load, all trucks, Shaffer Creek Bridge, 1968. (Number of trucks: 249.)

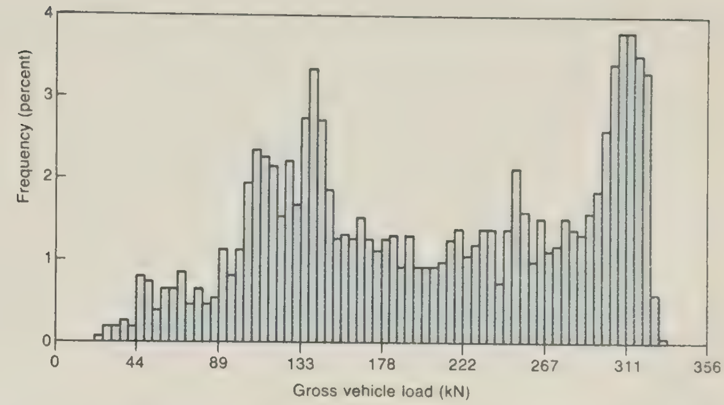


Figure 8.—Histogram for gross vehicle load, all trucks, CB&Q Bridge, 1969. (Number of trucks: 1,482.)

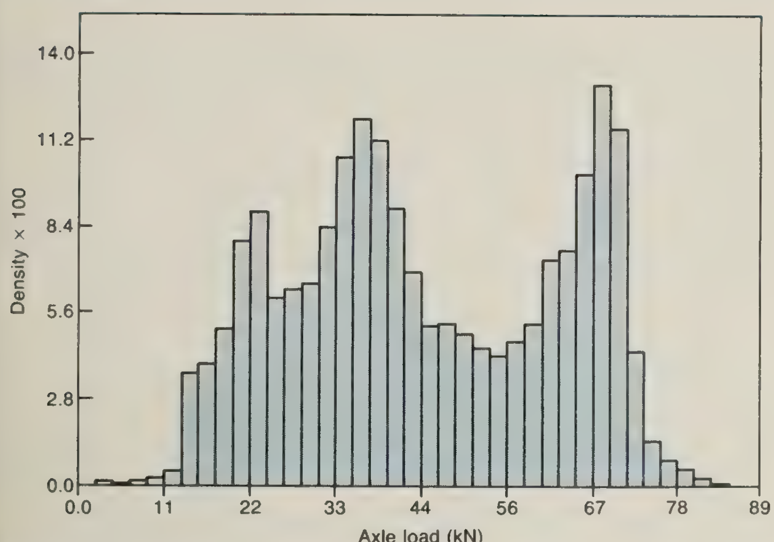


Figure 7.—Histogram for axle load, all axles, all trucks, CB&Q Bridge, 1969. (Number of trucks: 6,751.)

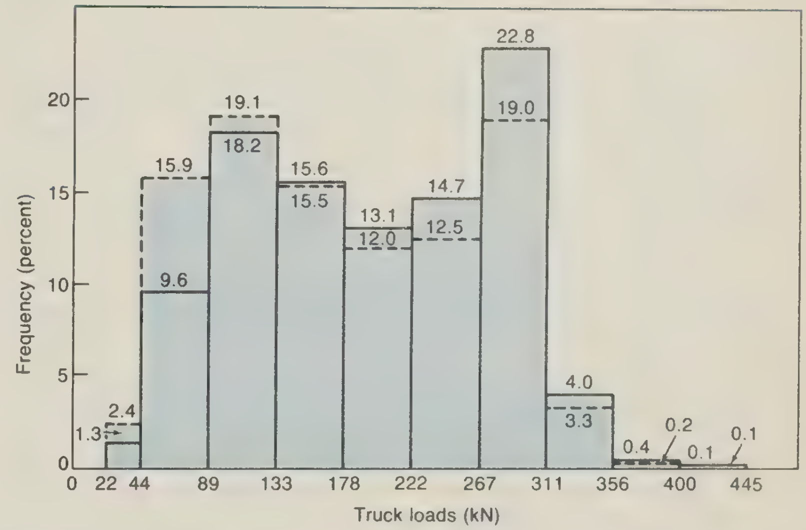


Figure 9.—Truck-load histogram constructed from eastbound weighing data from Westport (solid line) and adjusted values from Bridgeport test site (broken line).

and the material properties such as crack growth rates and notch toughness cannot be determined by load testing. The load tests are also unable to determine the ultimate carrying capacity of the structure.

Actual Loads

In recent years a great deal of actual weighing of vehicles has taken place, much of it for route planning purposes or for legal load enforcement. Yet for the most part, such vehicle weighing is not directly applicable to bridge loading problems. Special bridge loading studies are needed in which every vehicle (above a certain relatively minimum weight) crossing the structure in some representative time period is weighed. Strains (stress ranges) in selected members of the structure are collected at the same time. Several loading histograms from such studies are presented in figures 5–10.

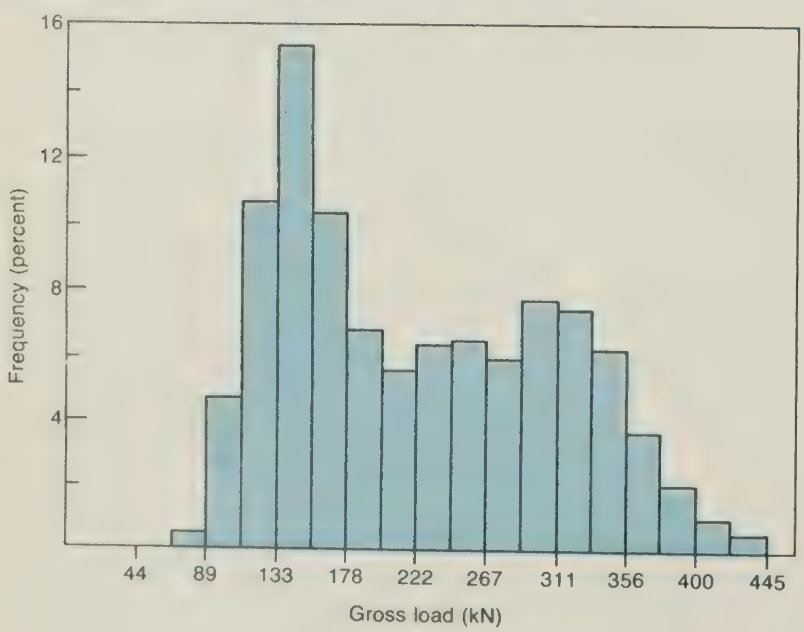
Axle load distributions, gross vehicle loads , the mean load, standard deviation, and number of vehicles weighed are shown on some of the figures. The double peak of the gross load histograms seems to be typical of many American tests. Figure 11 shows a composite loading histogram of a number of national studies in 1970.

Unfortunately, the weights obtained during bridge loading history studies are not always representative of everyday traffic. Truckers learn about the tests and take

Table 2.—Rating results for 21.3 m span

Rating method	Canadian Standards Association (S6)	Ontario	United Kingdom	Norway	United States
	Rating agency				
	kN	kN	kN	kN	kN
Unsupervised and mixing	738				
Supervised and only vehicle	1 557				
Readily available permit		881			
Controlled special permit		1 352			
Inventory working stress			423		458
Inventory load factor					494
Operating working stress			605		747
Operating load factor					818
Working stress				1 201	
Load factor				1 361	

Figure 10.—Tractor-trailer gross load distribution from Ohio. (Number of semitrailers: 2,328.)



their illegally overloaded vehicles on alternate routes to avoid the test site. To confront this problem, some States are monitoring weights all the time with built-in weigh stations. There is also considerable worldwide research and testing on equipment and schemes for weighing trucks in motion with either pavement platforms or the bridge as the weighing mechanism. (*5–8*) Many of the schemes are workable; in the next few years more attention will be paid to the monitoring and enforcement of loads.

Fatigue

Loads produce stresses and stresses, even small ones when repeated enough times, will cause a flaw to grow and possibly even cause a member to rupture. It will at least be necessary to spend money for crack repairs. There is increasingly more evidence of fatigue crack growth in American steel highway bridges. Often the problem originates in secondary members, where a relatively large initial flaw exists possibly because of poor fabrication practice and careless shop inspection. Several major load-carrying welded plate girders have completely fractured in recent years because of initial flaws in gusset plates and horizontal and vertical stiffeners. An even greater number of fatigue problems arise when flexible members frame into stiffer members. The growth of deflection-related fatigue cracks is a design problem.

The fatigue damage caused on bridges is a function of the number of live load stress ranges applied to the bridge. The stress ranges vary linearly with vehicle gross load for the main bending members of bridges. For deck elements and floor beams, the wheel and axle loads produce the stresses. Enough crack growth information is available from laboratory fatigue tests and from actual bridge damage observations on various cracked details to develop a relationship between loading and damage that can be used for design.

One such relationship is shown in figure 12, which is based on the composite truck load histogram shown in figure 11. The "damage factor" incorporates several

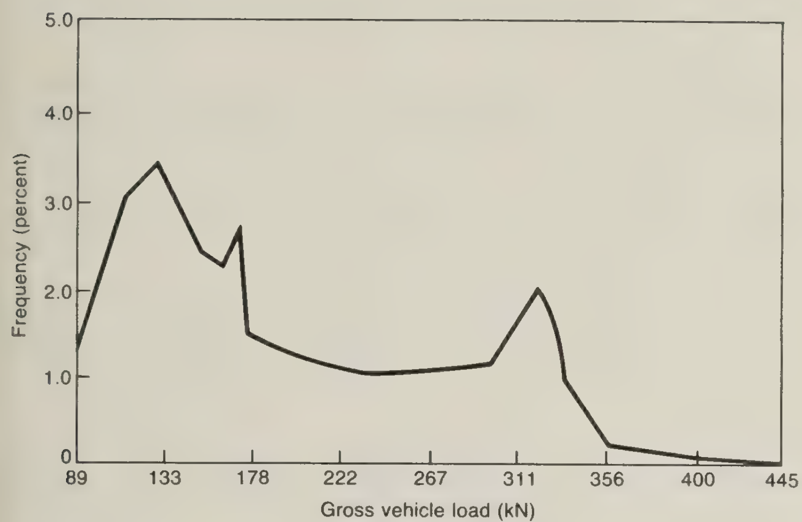
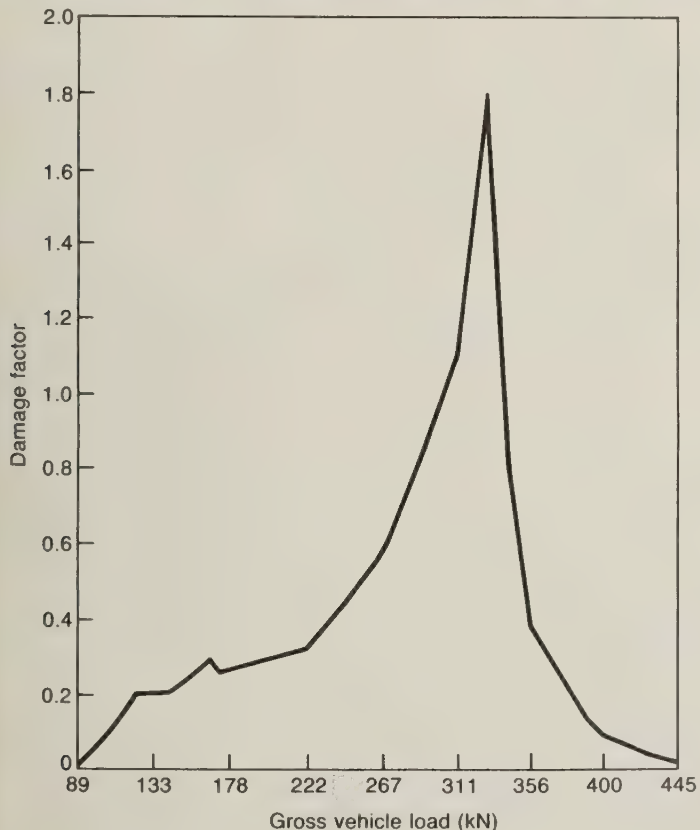


Figure 11.—Composite gross vehicle loads, 1970.

Figure 12.—Probable damage caused by various classes of truck loads. (Damage factor = damage per vehicle × number in class.)



relationships between average daily truck traffic, desired design life, ratio of actual vehicle to design vehicle weights, and Miner's hypothesis of damage. (9) By far, the greatest amount of damage results from the heavily loaded but still legal vehicles of 267 to 356 kN gross loading.

Fatigue design philosophy in the AASHTO Bridge Specifications differs between concrete and steel bridges. (*1*) In the design of reinforced concrete structures, the specifications restrict the allowable fatigue stress range (difference between maximum and minimum tensile stress) to approximately the value of the static design stress in the reinforcement. That is, a member is designed to withstand an unlimited number of stress ranges of the static design stress, which could be as high as 145 MPa.

The steel design portion of the specifications relates geometric details to allowable stresses and to the number of repetitions of these allowable stresses. Because the geometric details introduce stress concentrations, the magnitude of the unlimited number of stress repetitions must be reduced, according to the severity of the detail. For instance, a very severe detail such as a welded coverplate end would be designed to withstand a static stress of 145 MPa, but could only take a fatigue stress of 17 MPa for an unlimited number of repetitions. In steel bridges a stress of 17 MPa would quite commonly be produced by a legally loaded commercial vehicle, whereas a 145 MPa stress in a concrete bridge may never be realized in the lifetime of the bridge. Therefore, reevaluation of the philosophies behind the AASHTO fatigue design provisions may be in order.

Summary and Conclusions

- It may be time to increase the AASHTO HS design loadings. However, such an increase should not be made without a thorough economic study of the long range consequences.

- Considerable benefits in terms of longer pavement and bridge life would result from a universal and strict enforcement of legal allowable loads.

- The process used for load rating of bridges must be improved. A standard load testing vehicle and scheme can be a good tool in the rating process, especially when used in conjunction with bridge replacement priority settings.

- Because of the considerable variation in actual live loads and the lack of enforcement of legal loads in some jurisdictions, it may not be feasible to further refine the fatigue live load design provisions for steel bridges. Avoiding fatigue problems as much as possible by imitating the concrete provisions of the AASHTO specifications should improve the structures.

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Charles F. Galambos is Chief of the Structures and Applied Mechanics Division, Office of Research, Federal Highway Administration. He has been active in the field testing of bridges for a number of years and has also participated in the bridge tests of the AASHO Road Test. Mr. Galambos is the FHWA representative to the Organization for Economic Cooperation and Development committee on load rating of bridges.

Workshop on Seismic Design of Bridges

A 3-day workshop sponsored by the National Science Foundation was held in San Diego, Calif., in January to discuss earthquake problems related to bridges. International experts evaluated the state of knowledge and practice in the seismic design of bridges, discussed the objectives and scope of recent research programs, presented their findings to date, examined the needs and priorities for future research for improving current practice, and recommended research to fill gaps in knowledge.

Five working groups met to prepare recommendations in each of the following areas:

- Analytical procedures and mathematical modeling.
- Foundations, abutments, and ground motion effects.
- Experimental investigations.
- Retrofitting.
- Professional user needs.

Based on observed bridge damage, most experts agree that successful performance of bridges during seismic activity depends on the method of designing and incorporating details into the total structure. As a minimum, restraint should be provided across expansion joints, and hinges and column ties should be properly spaced to assure adequate concrete containment. Proper design of these two details will not significantly increase the cost of the structure and will improve seismic resistance.

Workshop proceedings are available. For further information, contact James D. Cooper, Bridge Structures Group, HRS-11, Structures and Applied Mechanics Division, Office of Research, Federal Highway Administration, Washington, D.C. 20590 (703-557-5272).

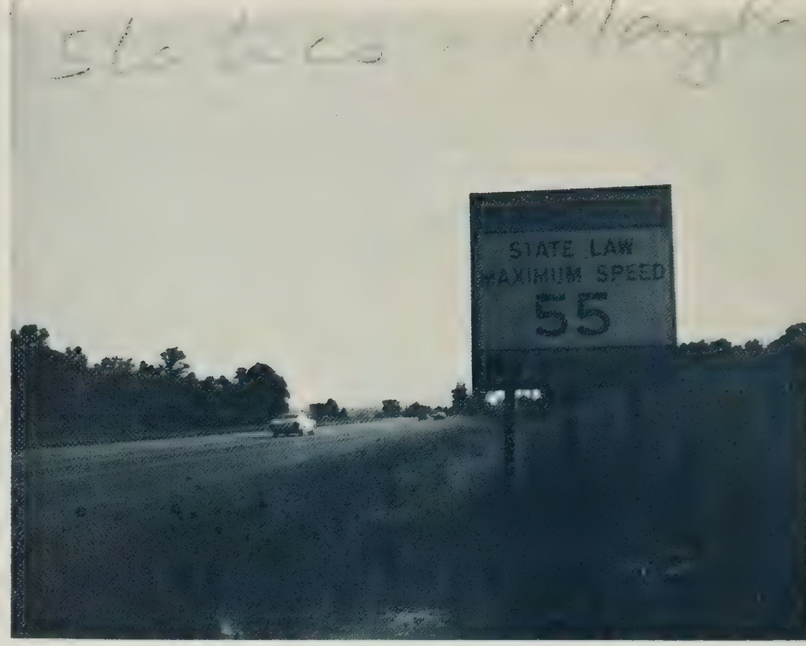
William J. Kenis Receives Award

Mr. William J. Kenis was the recipient of the 1978 award in the annual outstanding paper competition held among the employees of the Federal Highway Administration's (FHWA) Offices of Research and Development. This award covers the documentation of any technical accomplishment which may take the form of a publication, technical paper, report, or package; an innovative engineering concept; instrumentation systems; test procedures; new specifications; mathematical models; or unique computer programs. Each eligible candidate is judged on the basis of excellence, creativity, and contribution to the highway community, general public, and the FHWA.

Mr. Kenis, a highway research engineer in the Pavement Systems Group, Structures and Applied Mechanics Division, Office of Research, received the award for his research paper "A Design Method for Flexible Pavements Using the VESYS Structural Subsystem."

Mr. Kenis (left) is shown receiving a plaque for his accomplishment from Dr. Gerald D. Love (right), Associate Administrator for Research and Development. Also shown is Dr. T. F. McMahon, Chief of the Pavement Systems Group.

Analysis of Fatal Accident Trends on Maryland Highways, 1970-1976



by Harry S. Dawson, Jr.

This discussion of the analysis of accident trends in Maryland presents a somewhat different view of the influence of the 89 km/h (55 mph) speed limit on the reduction of fatalities. It, in some respects, conflicts with the conclusions drawn in past articles published in *Public Roads.* It is presented because it is believed that this point of view has some merit and deserves to be examined.

Various multivariate regression analysis techniques were used to analyze 7 years of fatal automobile accident and accident rate data for highways affected by the 89 km/h (55 mph) speed limit law in Maryland. Four of the fifteen variables studied—the normal historical decline in fatal accident rates with time, changes in the posted speed limit, changes in the level of traffic volume, and increases in police enforcement—statistically contribute to the explanation of the raw variance in the number and rate of fatal accidents. These four variables account for more than two-thirds of the total variance. Techniques for analyzing data from these highly intercorrelated variables are discussed and employed. It is concluded that the "lifesaving" ability of the 89 km/h speed limit may be substantially less than is generally thought. Additional research is required in other parts of the United States to confirm or deny the results obtained in Maryland.

Introduction

Public opinion and most technical investigations hold the implementation of the 89 km/h speed limit law to be the major factor responsible for the decline in fatal traffic accidents during and after the 1973-1974 energy crisis. However, scrutiny of the principal research efforts upon which this assertion is based reveals that none has involved an indepth statistical analysis of the data.

This article summarizes the results of a research effort, begun at the University of Maryland in late 1975, to statistically analyze the fatal accident trends between 1970 and 1976 in the State of Maryland. (*1*)[1] The study was generally limited to an analysis of fatal, nonpedestrian, automobile accidents occurring on highways that were affected by the adoption of the 89 km/h speed limit law.

Definition of Study Area

The study first identified 898 km of road segments in Maryland that were affected by adoption of the lower speed limit. These segments were not evenly distributed throughout the State; almost two-thirds of the mileage was contained in 5 of the State's 24 counties, and no highways were affected by the new speed limit law in 7 counties. Ninety-nine percent of the Interstate routes, 8 percent of the Federal Aid primary routes, and 9 percent of the State primary routes in Maryland were affected by the speed limit change. Fifty-one percent of the affected roadways experienced an 8 km/h decrease in posted speed limit, 9 percent had a 16 km/h decrease, and 40 percent had a 24 km/h decrease.

Fatal Accident Data

Fatal accident data for 1970-1976 were collected for the State as a whole, but most of the analysis concentrated on fatal automobile

[1] Italic numbers in parentheses identify references on page 68.

accidents occurring on highways affected by the lowered speed limit. Number of fatal accidents and fatal accident rates are presented in figure 1. The "total" curve on the number of fatal accidents graph shows a dramatic change in the slope of the trend line following the lowering of the speed limit. However, the "vehicle" curve on that graph and the "total" and "vehicle" curves on the fatal accident rates graph reveal that such a change may be minor or may not exist at all.

Figure 2 presents number of fatal accidents and fatal accident rates for Maryland highways that were affected by the 89 km/h speed limit law. Here, too, it is somewhat difficult to readily discern if the lowering of the speed limit affected fatal accidents and accident rates.

These trend data were further stratified and analyzed by type of route (Interstate, non-Interstate), previous posted speed limit (113, 105, and 97 km/h), amount of decrease in posted speed limit (24, 16, and 8 km/h), day of week (weekdays, weekends), and time of day (rush hour, evening, late night, other). The yearly percentage change in number and rate of fatal accidents was also computed. The results of this simple analysis were somewhat mixed. For example, although there was a decline in the number and rate of fatal accidents in the year following the lowering of the speed limit (1974),[2] the largest percentage declines frequently occurred in 1976 (3 years after the speed limits were lowered). Furthermore, routes that had only an 8 km/h permanent decrease in the posted speed limit displayed a greater decline in the number and rate of fatal accidents in 1974 than did routes that had a 16 km/h or 24 km/h decrease.

This rudimentary analysis indicated that, although the lowering of the speed limit seemed to contribute to the decline in fatal accidents, other factors also had a significant effect.

[2] In Maryland, the maximum speed limit was set at 80 km/h on Nov. 23, 1973, and then raised to 89 km/h on Jan. 10, 1974.

Traffic Volume Data

Changes in the magnitude and distribution (for example, type of route, time of day) of motor vehicle traffic have frequently been cited as contributors to the decline in fatal traffic accidents during and following the energy crisis. Figure 3 presents an aggregate summary of the annual traffic volume on Maryland highways affected by the 89 km/h speed limit law. The effect of the energy crisis on traffic volume in 1974 is obvious.

The traffic volume data were also analyzed to determine if there had been any changes in the hourly and daily distribution of traffic. It was discovered that from 1970 to 1976 there were only minor changes in the percentage distribution of traffic by time of day and day of week. There were dramatic seasonal variations, but these were repeated each year.

Thus, it would appear that the energy crisis did have an impact in Maryland on the amount of traffic using highways that were affected by the speed limit law, but had little or no impact on the relative daily or hourly distribution of that traffic.

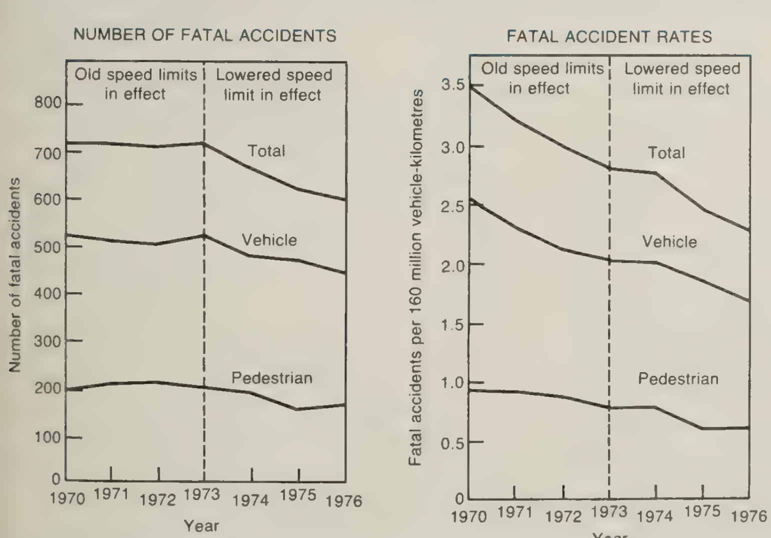


Figure 1.—Fatal accident and accident rate trends for the State of Maryland.

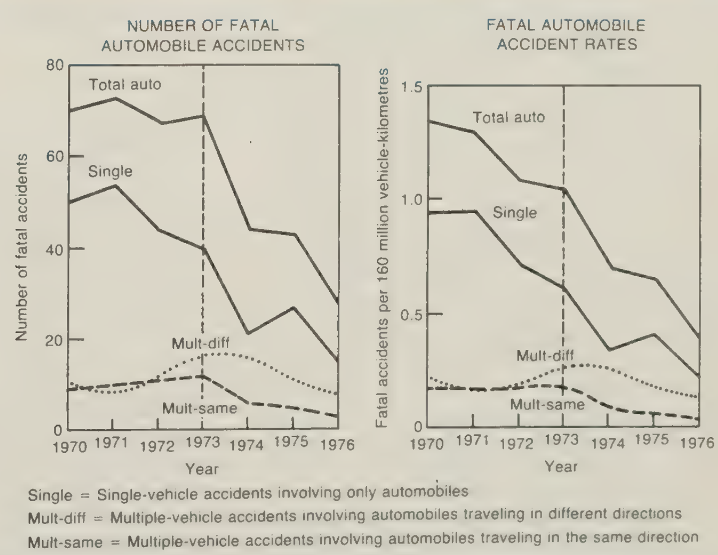


Figure 2.—Fatal automobile accident and accident rates on highways in Maryland affected by the 89 km/h speed limit law.

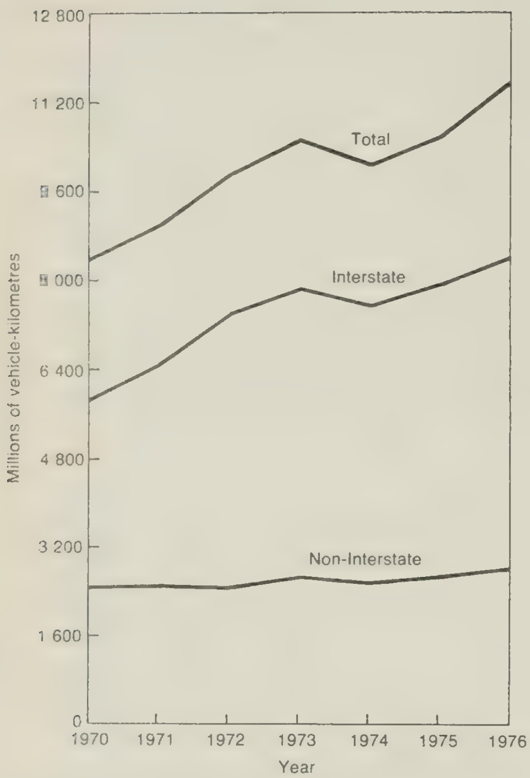


Figure 3.—Total annual traffic volume on highways in Maryland affected by the 89 km/h speed limit law, by type of route.

Spot Speed Data

There was no consistent set of spot speed data available in Maryland for the study period. Some data had been collected during July–August 1972 and during the last half of 1974, but the majority of available spot speed data was for 1975 and 1976. Most significantly, there were no spot speed data for the peak months of the energy crisis when the lower speed limits went into effect. The data that were available provided a "snapshot" of the spot speed situation before and after the speed limits were changed.

Table 1 summarizes the results of the analysis of the Maryland spot speed studies conducted between 1970 and 1976. Although the actual speeds of the automobiles using the highways grouped in this table are quite different, relative to the posted speed limit there is little difference in the average, 85th percentile, and pace speeds. Also, on the higher speed roads, the percentage of vehicles in compliance with the posted speed limit *before and after* the speed limit was lowered is quite similar. On the other hand, differences in the standard deviation in spot speeds and the percentage of vehicles in the 16 km/h pace indicate that after the speed limits were lowered the traffic flow was moving at more uniform speeds. Theoretically, this should have decreased the number and severity of multiple-vehicle accidents involving vehicles moving in the same direction. (*2*) The effect on the occurrence of accidents involving single vehicles or multiple vehicles moving in different directions could have been minimal.

It is interesting to note that table 1 shows that the average speed of automobiles in Maryland has closely bracketed the posted speed limit. One would expect the 85th percentile speed to approximate the posted speed limit instead because it is a common practice to set posted speed limits at a value equal to the 85th percentile of the free flow speed. (*3*)

Police Enforcement Data

The Maryland State Police began a major speed enforcement campaign in 1974. Table 1 shows that the percentage of automobiles in compliance with the posted speed limit was about the same before and after the speed limit was lowered to 89 km/h. Therefore, the dramatic increase in the issuance of speeding citations (fig. 4) indicates an increase in the level of police enforcement, and not merely an increase in the number of motorists violating the posted limits.

In addition to the normal, short-lived "halo effect" that the presence of a police vehicle has on traffic, a major enforcement campaign, such as the one in Maryland, might have a longer term effect on motorists who drive at excessively high speeds—129 km/h and above—and would contribute to a reduction in fatal highway accidents.

Table 1.—Comparison of results of spot speed surveys in Maryland before and after imposition of the 89 km/h speed limit law

Parameter	Highways affected by the speed limit law		Highways not affected by the speed limit law
	1972	1974–1976	1974–1976
Posted speed limit (km/h)	97–113	89	40–89
Difference between average speed and posted speed limit (km/h)	0.0	1.3	−0.2
Difference between 85th percentile speed and posted speed limit (km/h)	9.3	8.0	7.1
Difference between midpoint of 16 km/h pace and posted speed limit (km/h)	3.1	2.6	1.1
Percent compliance with posted speed limit	49	52	55
Standard deviation in spot speed (km/h)	9.3	6.9	7.4
Percent of vehicles in 16 km/h pace	63	78	76

Highway and Vehicle Safety Programs—The "Time Effect"

For more than half a century there has been a general decline in motor vehicle fatality rates. This trend, for the entire United States, is displayed in figure 5. The overall decline, especially the consistent decrease since 1966, is generally attributed to the "time effect"—improved vehicle and roadway safety standards, increased use of occupant restraints, driver education programs, and improved medical technology. (*4, 5*) Thus, even without the energy crisis and lowered speed limits, there still should have been a continuation of the historical decline in fatal accident rates because the safety programs and related factors responsible for the downward trend also continued.

Modeling the Problem

Fifteen variables, divided into five groups (table 2), were analyzed for their correlation with, and possible contribution to, the decline in fatal automobile accidents and accident rates on highways affected by the 89 km/h speed limit law. These variables were analyzed through multivariate step regression analysis techniques. Stage regression was used to separate the effects of primary and secondary (or second-order) variables. (6) In this technique, the results at a desired point in the analysis are "frozen" by calculating the unexplained residuals; then, any unused variables are regressed against these residuals. This procedure was also used to account for the effects that could be anticipated by assuming that the average annual decline in fatal accident rates that occurred before the energy crisis (due to the "time effect") also continued at the *same rate* of annual improvement after the energy crisis.

The two variables included in the miscellaneous grouping in table 2 were handled as dichotomous "dummy" variables. They were given a value of 1 when they were in effect and a value of 0 when they were not.

Initially, identical 4-week periods or "months" (each year would consist of 13 "months" of 28 days) were to be used as the time increment for the analysis. However, because of the infrequency of fatal automobile accidents on the 898 km of highway under analysis, this time increment had a coefficient of variation (standard deviation divided by the mean) that was too high to yield adequate results. For this reason, a quarterly (3-month) time increment was used to analyze the data.

Scatter diagrams and simple correlation coefficients were used to determine the proper functional form for each variable in the regression analysis. With one exception, the data collected on each predictor variable were found to demonstrate a roughly linear relationship with the number and rate of fatal automobile accidents. Speeding citations were found to be log-linear.

Results of the Regression Analysis

As a result of the regression analysis, a group of four variables—the normal historical decline in fatal accident rates with time, changes in the posted speed limit, changes in the level of traffic volume, and increases in police enforcement—were found to statistically contribute to explaining the raw variance in the number of fatal accidents and fatal accident rate criteria variables. Together, these four variables explain more than two-thirds of the total variance exhibited by the criteria variables (table 3).

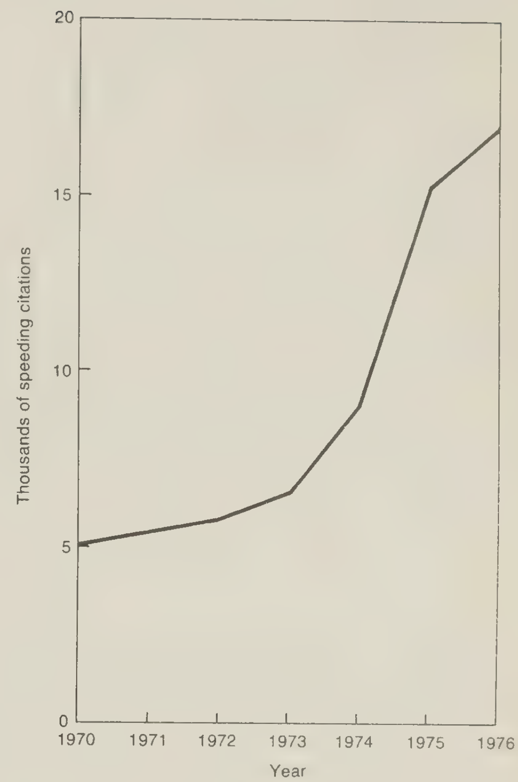


Figure 4.—Average monthly speed-related citations issued by the Maryland State Police.

Table 2.—Predictor variables used in the analysis

Grouping	Variables
Time	The "time effect"[1]
Speed	Speed limit Average speed Standard deviation in speed Percent of vehicles in the 16 km/h pace Percent of vehicles in compliance with the speed limit
Volume	Total traffic volume Percent of volume on Interstate routes Percent of volume during weekends Percent of volume during late night
Enforcement	Total number of speeding citations Percent of speeding citations issued during weekends Percent of speeding citations issued during late night
Miscellaneous	Daylight saving time or not Energy crisis or not

[1] The consistent annual decline in fatal accident rates attributable to the implementation of highway and vehicle safety programs.

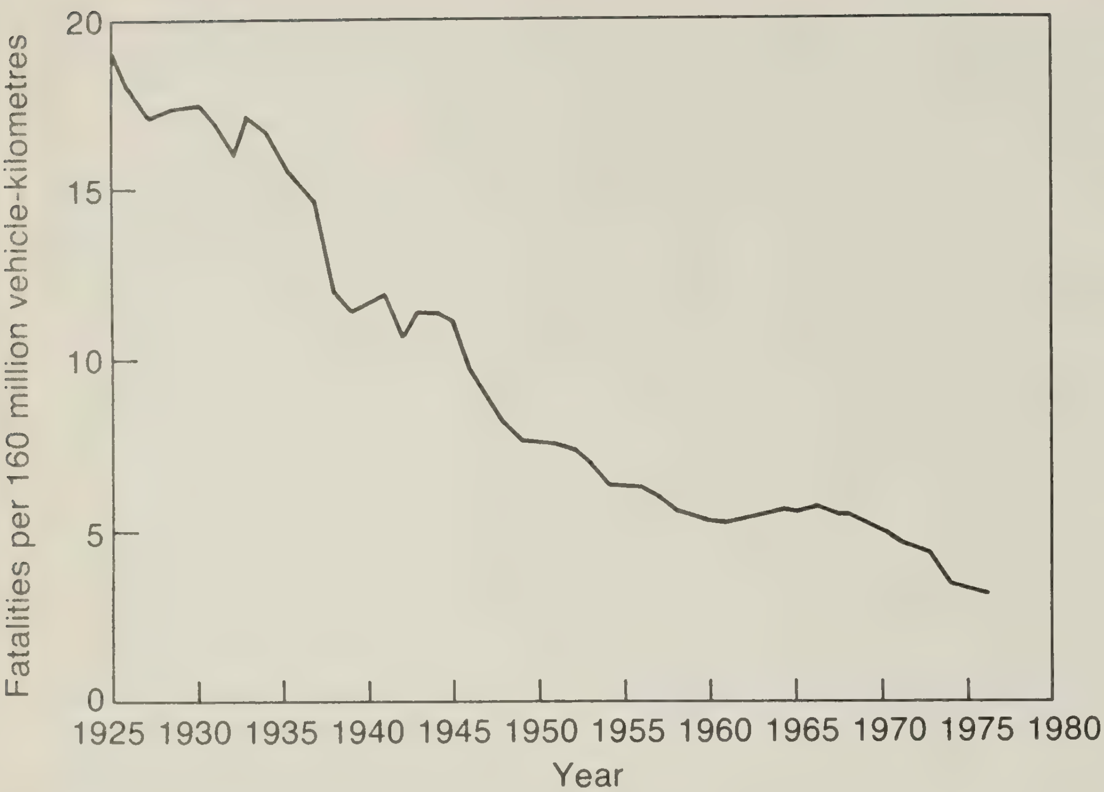


Figure 5.—U.S. motor vehicle fatality rates. (4, 5)

When one or more of the remaining 11 variables was added to the regression equation, the percentage of explained variance increased. However, because none of these variables was significant at the 95 percent level and none of the incremental regression results appreciably added to an understanding of the trends in the number or rate of fatal accidents, the variables were excluded from further study.

Statistical Tests of the Regression Results

To aid in the analysis of and to insure the validity of the regression results, a number of statistical tests were conducted. Specifically, the data were tested for normality, randomness, heteroscedasticity (lack of constant variance), auto-correlation (serial correlation between time periods), multicollinearity (a high degree of correlation between prediction variables), and spurious (false) correlation.

Difficulty with the data occurred in only one area—multicollinearity; this was found to be a serious problem. Although multicollinearity does not affect the performance or statistical measures (for example, R^2, standard error, F) for an overall regression equation, it makes it exceedingly difficult to determine how much of the total explained variance is attributable to each variable in a regression equation. (7)

The multicollinearity was caused by the simultaneous interaction of the four primary variables. At about the same time that the speed limit was lowered, traffic volume decreased and police enforcement increased. Additionally, the "time effect" applied. Given this situation, traditional regression analysis could not disentangle the results and effectively determine the relative contribution of each variable. Instead, it could only determine the aggregate effect of the four variables.

Analysis in the Presence of Multicollinearity

Two procedures were useful in unscrambling the results of these highly intercorrelated variables. In

Table 3.—Summary of regression results

Parameter	After regression on 4 primary variables[1]		After regression on 11 additional secondary variables[2]	
	Fatal accident analysis	Fatal accident rate analysis	Fatal accident analysis	Fatal accident rate analysis
Percent of variance explained	67.56	75.43	81.89	85.48
Standard deviation of the criteria variables	5.2174	0.3811	5.2174	0.3811
Standard error of the estimate	3.1520	0.2047	2.7982	0.1830
Standard error of the estimate divided by mean value of the criteria variables	0.2246	0.2212	0.1994	0.1978
F-ratio	11.973	17.652	4.1987	5.4667
Significance level (percent)	> 99	> 99	> 99	> 99

[1] The normal historical decline in fatal accident rates with time, changes in the posted speed limit, changes in the level of traffic volume, and increases in police enforcement.
[2] See table 2.

one procedure, the increase in explained variance was calculated for every possible entry position (for example, first, second, last) for each variable used in the regression equation. (*8*) Analysis of these incremental results was most useful in rank-ordering the relative importance and the degree of importance of each variable.

A second procedure was even more useful in interpreting the data. It is based on a procedure developed within the last 10 years called "commonality analysis," "element analysis," "component analysis," or "abstract partials." (*9–13*) This procedure apportions the total explained variance for a regression equation containing n predictor variables into n "unique" components and 2^n-n-1 "common" components. The "unique" components of variance are those that are contributed solely or uniquely by each predictor variable after the effects of all other predictor variables have been controlled or removed. The "common" components of variance are those that are contributed jointly by groups of two or more predictor variables after the effects of the other predictor variables have been controlled or removed.

Some simplifying assumptions make it possible to estimate the contribution to explained variance provided by *each* variable in the analysis. The results obtained in this manner were wholly consistent with the results obtained from the ranking technique described above.

Results

The results of this analysis from 1970 to 1976 on highways in Maryland that were affected by the 89 km/h speed limit law are summarized below:

- The continuation of the normal historical decline in fatal accident rates with time (due to such factors as improved vehicle safety standards, improved highway design standards, increased use of occupant restraints, driver education programs, and improved medical technology) accounts for 22 to 26 percent of the total variance or 33 to 35 percent of the explained variance in the fatal automobile accident and accident rate data.
- The lowering of the posted speed limit (and an equivalent reduction in average free flow traffic speed) accounts for 21 to 24 percent of the total variance or about 32 percent of the explained variance in the accident and accident rate data.
- The substantial increase in the level of police enforcement (as measured by the number of speed-related citations issued by the Maryland State Police) accounts for 14 to 17 percent of the total variance or 21 to 22 percent of the explained variance in the accident and accident rate data.
- Changes in the level of total traffic volume account for 8 to 10 percent of the total variance or 10 to 15 percent of the explained variance in the accident and accident rate data.

It is important to keep in mind that these results relate only to fatal automobile accidents on highways affected by the imposition of the 89 km/h speed limit law. It may be further noted that in Maryland the decline in the number of fatal accidents (of all types) on highways *not* affected by the speed limit law was *equal* to the decline on highways that were affected by the 89 km/h speed limit. Although the statistical analyses in this study did not extend to accidents on highways not affected by the 89 km/h speed limit law, it may be reasonable to assume that, of the four factors discussed above, only the first and last were of major importance on such highways. The speed factor was probably not applicable because the speed limits did not change on those roads, and the enforcement factor was probably not applicable because the enforcement campaign concentrated on high-speed highways. If this is true, then the continuation of the historical time trend due to the implementation of vehicle and highway safety programs is probably the most important factor leading to the overall decline in fatal traffic accidents in Maryland between 1970 and 1976.

Conclusions

The above results apply to a 7-year period that included about 4 years before and about 3 years after the 89 km/h maximum speed limit was adopted. However, researchers are often most interested in accident declines following the lowering of the speed limit. The results of this study can also be employed in such an analysis. The explained variance values presented above can be used to estimate the effect of a unit change in a particular variable on the occurrence of fatal accidents. In this way, it was found that for Maryland highways affected by the 89 km/h speed limit law, each 1 km/h decrease in the posted speed limit resulted in a 0.59 decrease in the number of fatal automobile accidents that occurred each year. This implies that lowering the posted speed limits on highways in Maryland from an average of 104 km/h in 1973 to an average of 88.2 km/h in 1974 resulted in about 9 fewer fatal accidents per year. If this result is further expanded to include trucks, buses, and other motor vehicles, then the implementation of the 89 km/h speed limit law accounts for a decline of about 11 to 12 fatal motor vehicle accidents each year. However, this represents only about 20 percent of the total fatal accident decline in Maryland between 1973 and 1974, or about 10 percent of the decline between 1973 and 1976.

Thus, although the saving of lives can be reliably attributed to the imposition of the 89 km/h speed limit law, the magnitude of its effect as a "lifesaver" may be much less than is generally assumed. Additional research is required in other parts of the country to confirm or deny the results obtained in Maryland.

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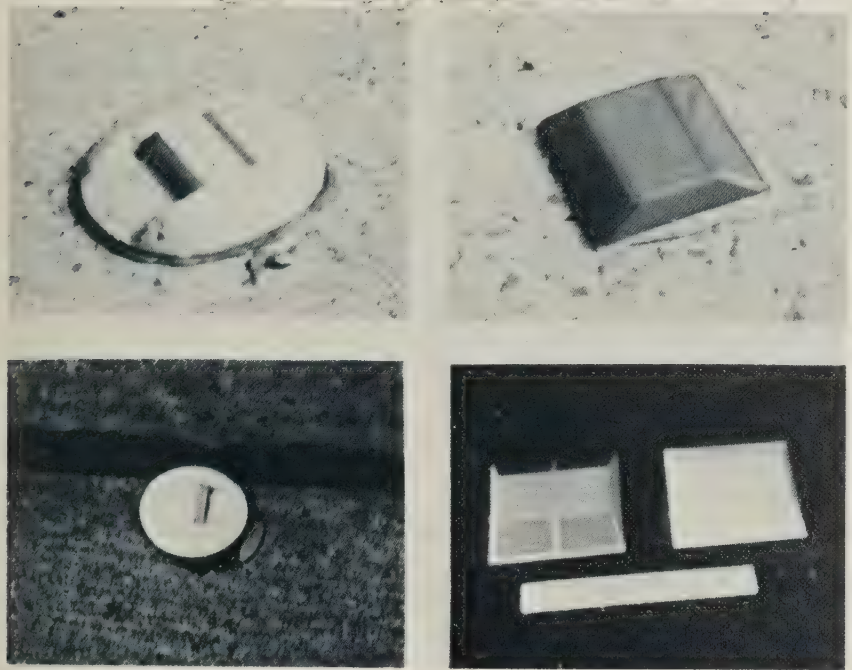
Harry S. Dawson, Jr., is involved in transportation planning and has 11 years of professional consulting experience with the firms of System Planning Corporation and Booz, Allen Applied Research. In addition to the research summarized in this article, Dr. Dawson has contributed to studies on the impact of aeronautical research and development funding on civil aviation, the practicality of utilizing helicopters for the underway replenishment of Navy ships, and the direct and indirect costs of replacing standard Army trucks in Europe with commercial vehicles. He also developed a cost-benefit procedure for determining when dual channel instrument landing system equipment should be installed at airports and coauthored the urban transportation chapter of the 1974 DOT Report to Congress.



Raised Pavement Markers for Construction Zone Delineation

by Charles W. Niessner

In 1976 the Implementation Division of the Federal Highway Administration (FHWA) initiated a series of projects to evaluate the effectiveness of raised pavement markers for guiding traffic through construction zones. During the 1976 and 1977 construction seasons, nine State highway agencies installed several types of reflectorized raised pavement markers on construction projects. These projects included bridge construction, interchange reconstruction, and roadway widening.

The cost, spacing, and ease of application and removal of the markers, and their effectiveness as traffic guides during the day and at night were evaluated. The public's acceptance of the markers was also assessed.

The general conclusion of the participating State highway agencies was that raised reflective markers provide excellent nighttime temporary delineation—particularly when the road is wet—at a low cost and with little or no maintenance. On two projects the use of the raised pavement markers resulted in a reduction of accidents. Comments from construction personnel and the public on the effectiveness of the markers were favorable.

Introduction

The problem of safely directing traffic through construction and maintenance zones is becoming more complex, especially on high-speed, high-volume roadways. Normal traffic flow must be maintained and the potential for accidents must be kept to a minimum. Several alternatives are available to maintain safe traffic flow. These alternatives depend on many factors, such as normal traffic, peak traffic, percentage of trucks, speed, geometry, seasonality, and urgency of reconstruction. These factors compound the problem and can require a variety of methods to cope with each situation. Methods presently being used include delineation posts, flares, temporary striping, reflectorized markers, signs, barricades, barriers, and flagpersons. Of these, the raised reflectorized pavement marker provides excellent wet night retroreflection and seems to be most effective. After removal, the markers are not as misleading as painted stripes. Expected high costs have discouraged widespread use of the markers, especially in situations where they are discarded as the construction progresses; however, additional safety provided by the unique reflective properties justifies their expanded use.

Discussion of Marker Effectiveness

Speed check and taillight study

On one construction project where the markers were used in conjunction with painted lines, a speed check and taillight study were conducted to determine marker effectiveness (fig. 1).

The car following method was used to determine the speed of the vehicles. It appears that speed increased because of the markers. If the markers had made no difference, the day and night differential speed should have remained constant after markers were installed.

After installation, however, the time required to travel through the area in daylight decreased about 5 seconds (the raised markers, with well-maintained paint stripes, should have little effect on daylight delineation) but the travel time at night decreased about 11 seconds. The raised markers accounted for about a 6-second decrease in travel time. (*1*)[1]

The taillight study was inconclusive because of a lack of data (not enough brake lights). However, the data collected before marker installation showed that many motorists rode the centerline or drove a flatter than actual curve by driving left of the centerline. The markers helped confine motorists to the proper lane both during the day and at night. The raised markers not only provided extra delineation but also acted as a rumble strip. (*1*)

Figure 1.—Daytime view of construction detour.

Reduction of vandalism

Use of the markers also reduced vandalism on the construction project. Traffic had been controlled by the use of "candlestick" plastic cones, "zebra board" reflective portable temporary barricades, and standard temporary signing. The barricades and cones were constantly vandalized—cones were knocked down, which allowed vehicles to stray onto paved areas not open to traffic. After markers were installed, these problems were reduced. (*2*)

[1] Italic numbers in parentheses identify references on page 75.

Accident reduction

Raised reflective pavement markers used on construction detours reduced the number of accidents. On two of the construction projects, it was possible to compare accident data before and after marker installation. The first project involved the reconstruction of 27.0 m of a four-lane roadway. Two-way traffic was being maintained on two lanes while construction was underway on the other two lanes.

During the 1976 construction season when no markers were used, there were 2 fatalities and 113 minor accidents. In May 1977, two raised markers were placed every 12.2 m on the centerline separating the two-way traffic; in 1977, there were no fatalities and only 17 minor accidents. (*3*)

On the second project, the rehabilitation of a 12.9 km section of a four-lane Interstate highway, traffic was maintained on the two-lane frontage roads. Pavement markers were installed only on the centerline. Data from a rehabilitation project conducted between July 1975 and October 1976 were used for comparison. Traffic had been maintained by use of the frontage roads but raised pavement markers had not been used on this project. Table 1 compares accident data for the two projects. Although the total number of accidents was slightly higher in the section with markers, the kinds of accidents affected most by the installation of the markers—sideswipe and nighttime accidents—decreased. (*4*)

Marker Patterns and Spacing

Four different raised reflective marker patterns were evaluated on one of the construction detours to determine optimum marker spacing.

Pattern 1 consisted of paint stripes 4.6 m long and 7.6 m apart with a raised reflective pavement marker placed midway between the paint stripes. Pattern 2 included a reflective marker at both ends of each paint stripe. For Pattern 3, a reflective marker was placed at the beginning of each paint stripe and, for Pattern 4, a marker was placed at the end of each paint stripe.

Pattern 1 provided drivers with the best visual perception on tangents. Pattern 2 was best for curves because it provided the driver with twice as many reflective markers. More markers are needed on curves because the loss of the markers on curves will be higher, leaving

Table 1.—Comparative accident data

Test section	Average daily traffic volume	Number of accidents		
		At night	Sideswipes	Total
With markers,[1] 12.9 km in small urban area	14,132	30	25	117
Without markers,[2] 13.7 km in rural area	14,945	42	35	110

[1] 15-month accident history (8/76–10/77)
[2] 16-month accident history (6/75–10/76)

voids in the pattern, and the reflective surface of many markers will be distorted by tire marks, mud, dirt, and oil. (*4*)

Markers and paint stripes

Markers used without paint stripes must be colored either white or yellow to provide adequate daytime delineation (fig. 2). The color of the raised pavement marker should be the same as the color of the pavement stripe or line which it represents. In addition, markers without paint stripes should be limited to projects that will be completed before snowplow operations are likely to begin.

Without painted lines, colored reflective markers need to be placed 1.8 to 3.0 m apart to give the daytime appearance of a solid line. On critical alinement or abrupt changes in alinement bias, the reflective markers should be placed on 1.8 m centers. If it is necessary to slow traffic prior to an abrupt change in alinement, the marker density can be accelerated by gradually stepping down from 3.0 to 1.8 m centers.

Using single reflective markers on 12.2 m centers with a cluster of three nonreflective markers will give the daytime appearance of a lane line. Where abrupt changes in alinement are to be marked, groups of three reflective markers on 0.9 m centers and spaced 12.2 m apart should be used. (*5*)

When markers are used with painted stripes, the recommended marker spacing for tangent sections is 12.2 m to 15.2 m. For curves, the spacing between markers should be decreased to 6.1 m to 7.6 m on both centerline and edgelines.

Marker Installation, Maintenance, and Removal

The "self-adhesive" markers are well suited for construction detour use because they are easy to install and maintain. No epoxy formulation or special application equipment is necessary. These markers are virtually ready for traffic as soon as they are installed, and they prove to be surprisingly durable under normal traffic. No significant difference in the loss rate was found between markers placed with epoxy and those placed with butyl pads.

The basic installation procedure is to mark and sweep the location of the marker (fig. 3). Using a marker-size

Figure 2.—Raised pavement markers used for centerline and edgeline without paint stripes.

cardboard template, an adhesive primer is applied with a paintbrush to each premarked location. The paper backing is removed from the marker and the marker is placed on the cured primer. A car sets the marker by slowly driving over it. (*6*)

When the markers are placed on portland cement concrete (PCC) pavement, the surface should be free of dirt, oil, and general debris. Installation of the markers on new bituminous surfaces should be delayed approximately 2 weeks after the paving is completed. This delay should help minimize the loss rate and the problem of the marker impediment in the surface. Where markers are used on successive lifts of asphalt pavement, however, delaying installation is impractical.

Lower temperatures (below 10° C) seem to reduce the bonding capability of the butyl pads. In addition, the damage and loss record of ceramic markers placed with butyl pads at one location indicated that a butyl pad does not provide a ceramic marker with adequate support and adhesive properties, especially if the markers are subjected to heavy equipment traffic. (6)

Removal of markers, even when the butyl pad is used, causes minor damage (requiring no repair) to asphalt pavements (fig. 4). The damage was limited to the removal of from 6 mm to 19 mm of the asphalt pavement directly beneath the marker.

Various methods were used to remove the markers. These included heating the marker or using picks, an "L" shaped paint scraper and 0.9 kg hammer, a heavy hammer to shatter the marker, a long-handled flat shovel, and a motor grader. (7)

Eliminating the primer made removal of the markers easier and reduced the damage to the pavement. However, without the primer the markers tended to be moved or shifted by traffic and the loss rate increased. Temporary markers (used for 1 or 2 days) could be installed without the primer in areas where no turning movements were made over the markers.

Markers on the PCC surface were easily removed by a small "ditch-witch" tractor with a front-mounted blade (fig. 5). The operator drove down the middle of the roadway, using the blade to knock the markers from the road surface. PCC pavements were not damaged.

An attempt was made to save some of the butyl-backed markers for reuse. Because they had too many asphalt concrete particles on the pad, they would not stick without being cleaned. Cleaning, however, was very time consuming. Scraping the pad with a putty knife was unsatisfactory. Soaking the marker in diesel fuel removed the asphalt but destroyed the adhesive action. With the aid of the primer it was possible to reuse the marker, but the idea of saving the markers was abandoned because of the time and cost involved. (2)

Used epoxy-applied markers were soaked in various solvents for different periods of time to remove the asphalt particles, but all of the solvents eventually destroyed some part of the marker. Many markers that came loose without pulling up too much asphalt were successfully reused in areas not subject to constant tire contact. Generally speaking, however, reuse of raised pavement markers was impractical.

Figure 3.—Installation of pressure-sensitive butyl-backed marker.

Figure 4.—Pavement damage due to the removal of a raised pavement marker.

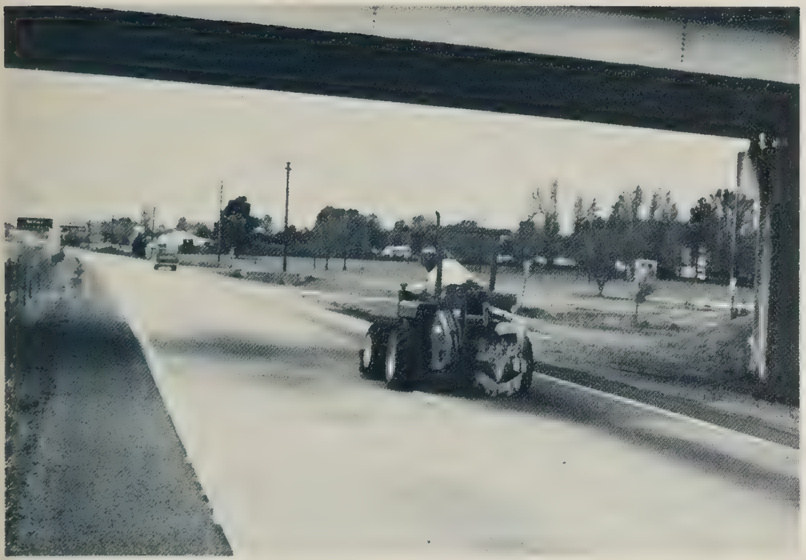


Figure 5.—"Ditch-witch" tractor removing pavement markers from PCC surface.

Markers in Gore Areas

Markers were placed at three different locations to define gore areas. The markers could be seen for quite a distance (±122 m) and gave motorists advance warning of the approaching ramp-roadway separation. The impact attenuators at these locations had sustained heavy damages over the years and placement of the markers decreased such incidents. The markers eliminated some driver indecision by giving advance warning of the approaching turn. (*8*)

Use of the butyl-backed raised pavement marker to outline the channelization markings at gore areas where impact attenuators are located should be considered even in northern States where snow removal operations are necessary. The cost of replacing the markers lost because of snow removal operations is minimal compared with the cost of repairing the attenuators. The effective pavement delineation during months when snow removal operations are unnecessary seems to be worth the cost of the markers.

Economic Considerations

Table 2 lists the installed costs for the various types of markers that were evaluated in this project. The cost ranged from $0.90 to $4.50 per marker. The cost differences resulted primarily from differences in the price of the material, because installation costs about the same for all markers. Markers with butyl pads generally cost less per unit than those installed with the epoxy.

The cost of removing the markers ranged from $0.08 to $1.00 per marker. The average removal cost is estimated at $0.25 per marker.

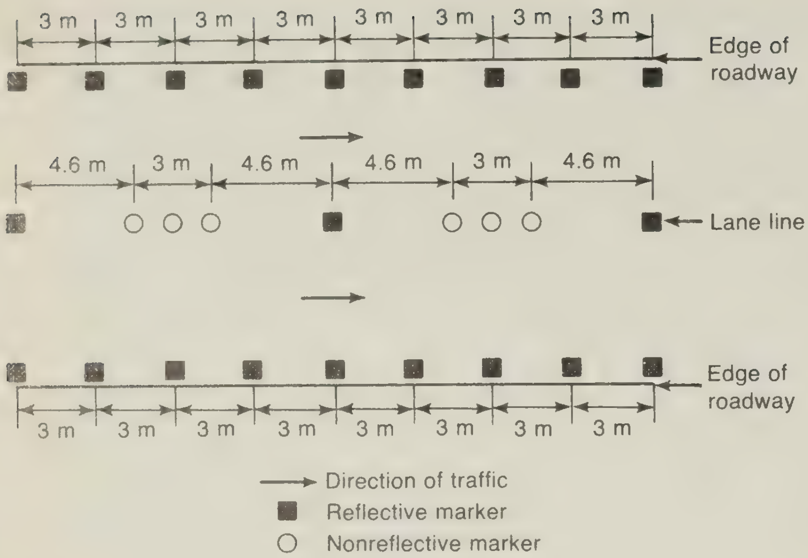
Figure 6 shows a layout for a two-lane, one-way detour without paint stripes. This layout requires 990 markers per kilometre—330 on each edgeline and on the centerline. At a marker cost of $2.50 (which includes removal), the cost per kilometre for this installation would be $2,475. The cost of painting ($0.33 per m) and then removing ($0.98 per m) two edgelines and a center skip line would be $2,965 per kilometre for the same layout—$490 more than the cost of the markers.

The cost for painting was based on the assumption that it is a bid item in the construction contract. Painting would probably cost less if done by State striping crews. However, projects with several traffic staging plans

Table 2.—Installed costs

Marker type	Number of markers purchased	Cost per marker	Type of installation crew	Adhesive
Stimsonite 88	1,402	$4.18[1]	Contractor	Butyl pad
Stimsonite 88	532	$3.00[1]	Contractor	Butyl pad and epoxy
Permark P15 and P7	2,396	$1.93	Contractor	Butyl pad and epoxy
Stimsonite 88	4,436	$4.50[1]	Contractor	Epoxy
Stimsonite 88	663	$1.70	State crew	Butyl pad
Stimsonite 946	137	$0.90	State crew	Butyl pad
Permark P17	599	$1.70	State crew	Butyl pad
Ray-O-Lite	638	$2.26	State crew	Butyl pad
Ray-O-Lite	1,943	$2.49	Contractor	Epoxy

[1]Includes cost of removal.



Cost per km using only raised pavement markers:

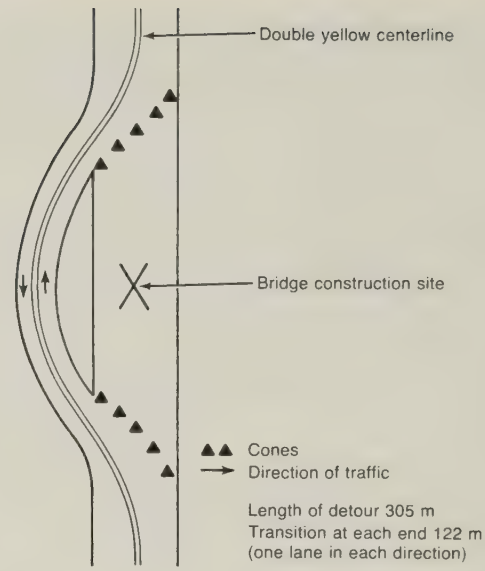
Markers on edgeline	1005.84 m ÷ 3.048 m = 330 × 2 = 660
Markers on centerline	1005.84 m ÷ 12.192 m = 82.5 × 4 = 330
Total markers per km	660 + 330 = 990

990 markers × $2.50 per marker = $2,475 per km

Cost of paint stripes:

Edgelines	1005.84 × 2 = 2011.68 lin m
Skip line (10–30 ratio)	1005.84 × ¼ = 251.46 lin m
	2011.68 +251.46 = 2263.14 lin m of stripes per km
Painting	2263.14 lin m × $0.33 per m = $747
Removal	2263.14 lin m × $0.98 per m = $2,218
Total cost	$747 + $2,218 = $2,965

Figure 6.—Layout for two-lane, one-way detour without paint stripes.



Delineation paint only

Painting
Two edgelines 548.64 m × 2 = 1097.28 lin m

Double yellow centerline 548.64 m × 2 = 1097.28 lin m

2194.56 lin m × $0.33 per m = $720

Removal, transitions only
One edgeline 122 m × 2 = 244 m
Centerline 244 m × 2 = 488 m

732 m

732 m × $0.98 per m = $720

Painting	$ 720
Removal	$ 720
Total cost	$1,440

Delineation paint and markers

Transition

Marker every 3 m on edgelines
244 m ÷ 3 m = 80 × 2 = 160 markers

Double line of markers every 3 m on centerline
244 m ÷ 3 m = 80 × 2 = 160 markers
160 + 160 = 320 markers

Detour

Paint

Two edgelines 305 m × 2 =	610 m
Centerline 305 m × 2 =	610 m
	1220 m

Markers
Every 12.2 m on edgeline and centerline

305 m ÷ 12.2 m = 25 × 4 = 100 markers

420 markers × $2.50=	$1,050
1220 lin m × $0.33=	$ 400
Total cost	$1,450

Figure 7.—Typical construction diversion road.

require a striping crew for each change in the traffic phase, which could seriously disrupt the State's normal striping operations. In outlying areas a striping crew might not be readily available. The markers, however, are available at any time.

A typical construction diversion road used at many bridge construction sites is shown in figure 7. Assuming this is a two-lane, two-way diversion road, 305 m long with 122 m transitions on each end, the cost for painting stripes would be $720. The cost for removal of the stripes only in the transitions is $720, for a total cost of $1,440.

The cost of using a combination of pavement markers and paint stripes would be $1,450. This combination consists of only reflective raised markers in the transition, spaced at 3 m centers on both edgelines and a double row on 3 m centers on the centerline. The diversion road is painted with two edgelines and a double yellow centerline, with reflective markers at 12.2 m intervals on both the edgelines and centerline.

Because there are only markers in the transition area, there will be no need for stripe removal; the diversion road with the paint stripes will be removed when the construction is complete.

As the examples in figures 6 and 7 show, the use of raised reflective markers does not substantially increase the cost of the project. The small additional cost is justified by the improved traffic performance and safety that the markers provide.

Summary

The study showed that raised pavement markers provide positive daylight and nighttime guidance. Markers used on construction detours tend to reduce the number of accidents. The cost of markers and paint is equal to or less than the cost of paint striping and removal. The additional safety, improved operations, and reduced vandalism provided by the markers, in addition to their acceptance by the public, government, and construction personnel, justify their expanded use.

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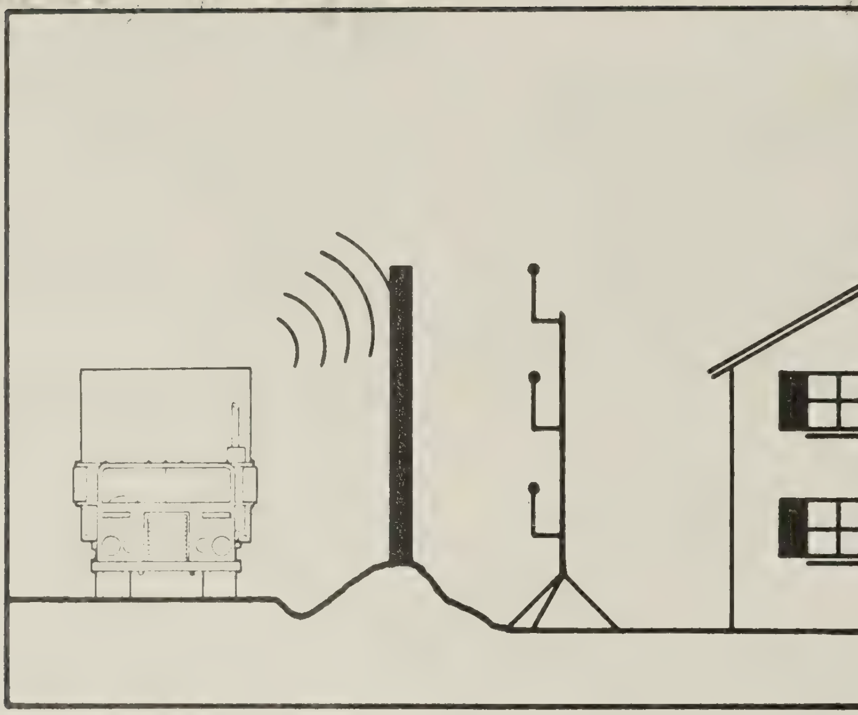
(6) "Construction Zone Delineation—Raised Pavement Markers, Final Report," *Division of Highways, Traffic Engineering Branch, North Carolina Department of Transportation,* December 1977.

(7) Donald C. Gostomski, "Construction Zone Delineation With Raised Pavement Markers at Lake Katherine Bridge," *Division of Highways, Wisconsin Department of Transportation,* May 1977.

(8) R. H. Johnson, "Raised Pavement Markers Through Construction Zones," *Department of Public Works, Commonwealth of Massachusetts,* September 1977.

Charles W. Niessner is a highway engineer in the Implementation Division, Office of Development, Federal Highway Administration. His current responsibilities include the Implementation Division's delineation and value engineering programs.



Prediction of Highway Traffic Noise

by
Timothy M. Barry

Introduction

A staff study to develop a logical, easy to use, traffic noise prediction model was initiated in 1976 between the Office of Research and Office of Environmental Policy of the Federal Highway Administration (FHWA). (*1*)[1] At that time, several models for predicting noise impacts and devising abatement strategies were available to traffic noise specialists. These models, however, were inconsistent, inadequately calibrated, poorly documented, and as a result, frequently misused. The assumptions and inherent limitations of the models were not spelled out or were simply unknown. In developing the study for the FHWA highway traffic noise prediction model, the goal was to synthesize a prediction procedure based on the best available techniques and data, clearly identifying the assumptions and the resulting limitations of the model.

The basic approach was to break down the traffic noise model into a base noise level that is corrected by a series of adjustments to account for site specific characteristics. With this approach it was possible to derive a series of adjustments that are physically significant to the traffic noise specialist. The major benefit of this approach is that users can separately examine the effects of vehicle emission levels, traffic volumes, distances, road length, excess attenuation due to absorptive ground cover, and shielding by barriers, berms, and rows of houses. Additionally, with straightforward modifications, users can tailor the basic model to meet a range of highway sites or traffic conditions not directly addressed by the basic model.

Describing Traffic Noise

Traffic noise is a very complex phenomenon of sound generation and propagation. To describe traffic noise properly, its amplitude, frequency, and phase, as each varies in time, must be specified. Because this is impractical, several single number ratings of traffic noise have evolved over the years. These single number ratings are generally either statistical descriptors or energy-based descriptors.

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

Statistical descriptors of traffic noise are highly dependent on vehicle headway spacings and vehicle noise emission levels. These descriptors are usually easy to measure but are very difficult to predict. Developing a traffic noise prediction model based on a statistical descriptor leads to a number of theoretical and technical problems that are not easily resolved.

Energy-based descriptors consider only the sound energy in traffic noise and are easy to measure and predict. Recent studies have shown that for all but the very low traffic volume situations, energy-based descriptors correlate highly with people's annoyance with traffic noise. (*2*)

The FHWA highway traffic noise model is based on an energy descriptor called the A-weighted hourly equivalent sound level, $L_{eq}(h)$, defined by the equation:

$$L_{eq}(h) = 10 \log \left[\frac{1}{T} \int_{o}^{T} 10^{L_A(t)/10} dt \right]$$

in which $L_A(t)$ is time varying A-weighted traffic noise level and T is the averaging interval (1 hour). A-weighting refers to a special frequency weighting function that is applied to the sound level to account for the relative frequency loudness sensitivity of the human ear.

The equivalent sound level converts a time varying noise to an equivalent constant amplitude sound level which contains the same amount of energy as the original time varying noise.

FHWA Highway Traffic Noise Prediction Model

Figure 1 is representative of a 120-second recording of A-weighted traffic noise measured near a road operating at level of Service C. The many sharp peaks correspond to the peak noise levels of the individual vehicles as they pass by the observer. The random fluctuations (roughness) superimposed on the time history are the results of local micrometeorologic conditions (wind, thermal gradients) and short range propagation effects. For modeling purposes, these random fluctuations are not important. Figure 2 shows the same time history with the roughness removed. The A-weighted equivalent sound level for the 2-minute noise sample is shown by the dashed line in figure 2 and contains the same sound energy as the time varying curve.

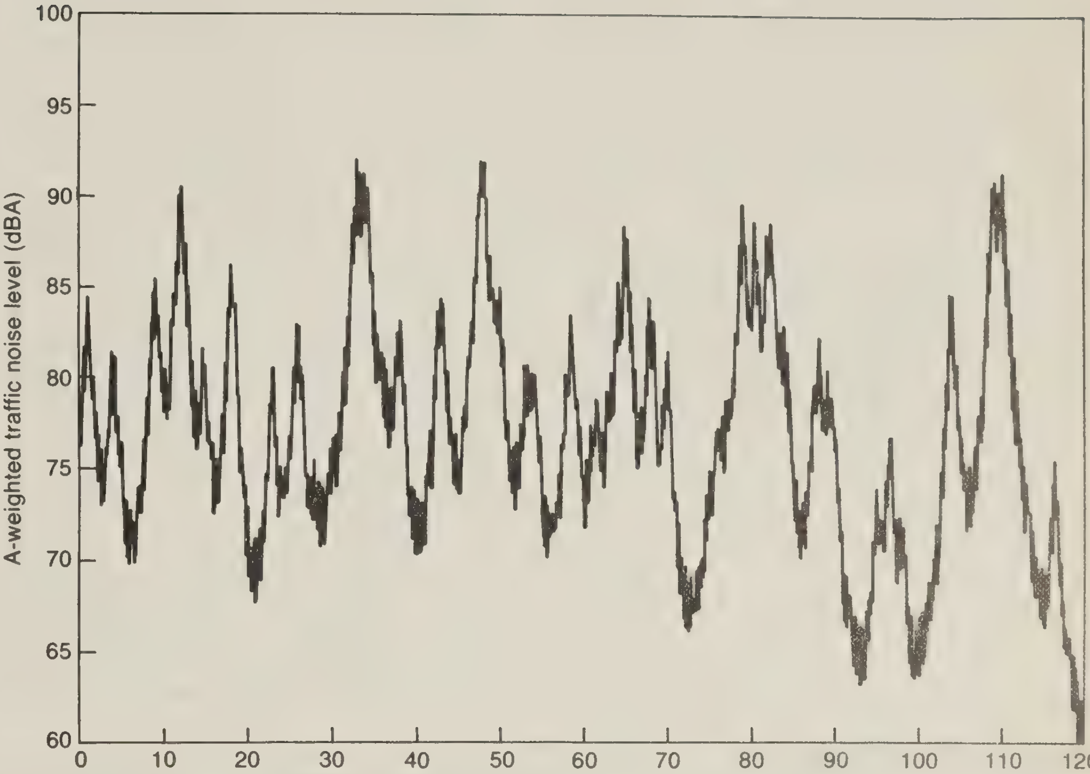


Figure 1.—A-weighted traffic noise level.

The FHWA highway traffic noise model treats each vehicle as a constant speed acoustic point source. This treatment, which has been validated by field measurements, leads to an expression that relates the noise level time history of a vehicle's passby as it passes an observer to the vehicle's speed, type of vehicle, observer distance, and ground cover conditions. Figure 3 shows a typical vehicle passby for an automobile.

Many vehicles exhibit similar emission characteristics; therefore, it is not necessary to deal with the noise of each vehicle on the highway, but rather with classes of acoustically similar vehicles. The FHWA model separates traffic into three acoustically similar classes—automobiles, medium trucks (two-axle, six-tire), and heavy trucks (three-axle or more). Noise level calculations are made for each class with the total noise level calculated by logarithmically summing the equivalent level of each vehicle class.

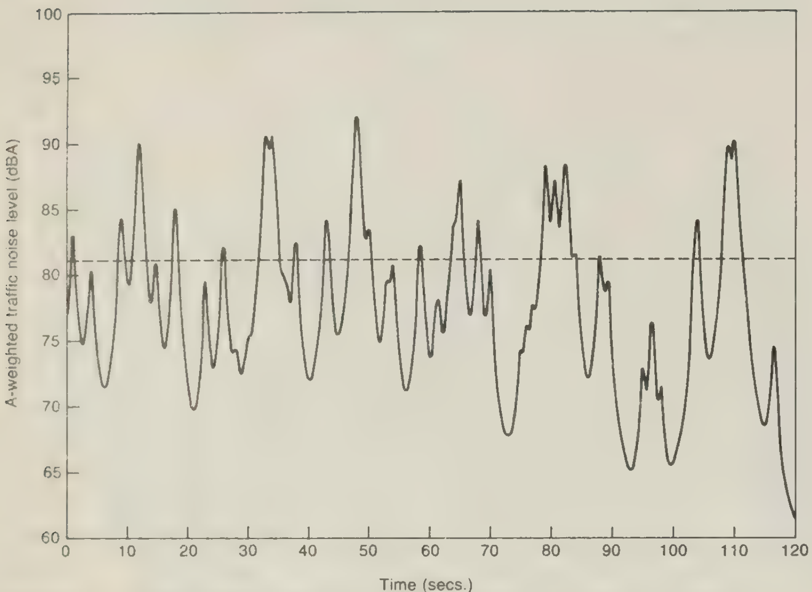


Figure 2.—A-weighted traffic noise level with random fluctuations.

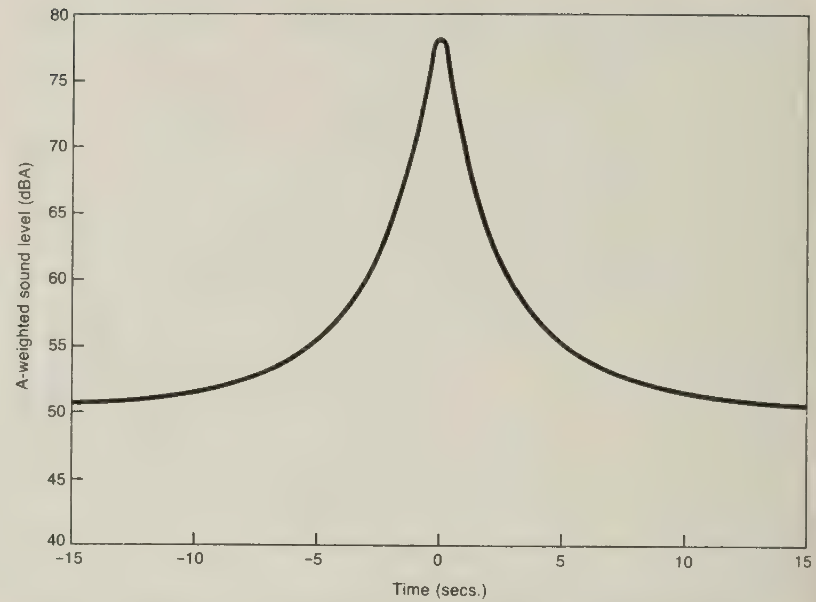


Figure 3.—Sound level time history of passing vehicle acting as acoustic point source.

Steps in Predicting Traffic Noise

The flow diagram for predicting traffic noise levels using the FHWA model is shown in figure 4. Six steps are involved in making this prediction.

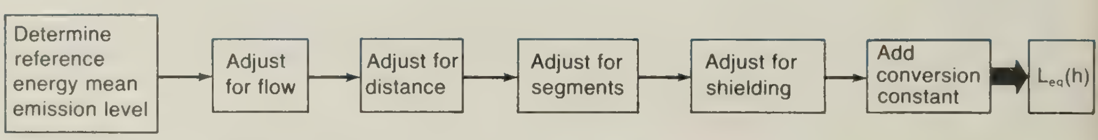


Figure 4.—Flow diagram for predicting traffic noise levels.

Step 1—Determine reference energy mean emission levels.

The reference energy mean emission level represents the base noise level as a function of speed for each of the three vehicle classes. It is the average peak passby noise level measured at 15 m from the travel centerline and has been corrected to account for the statistical variation in peak passby levels within each class. Figure 5 shows the reference energy mean emission levels used in the FHWA traffic noise model. They are based on field data taken in Florida, North Carolina, Colorado, and Washington. (*3, 4*)

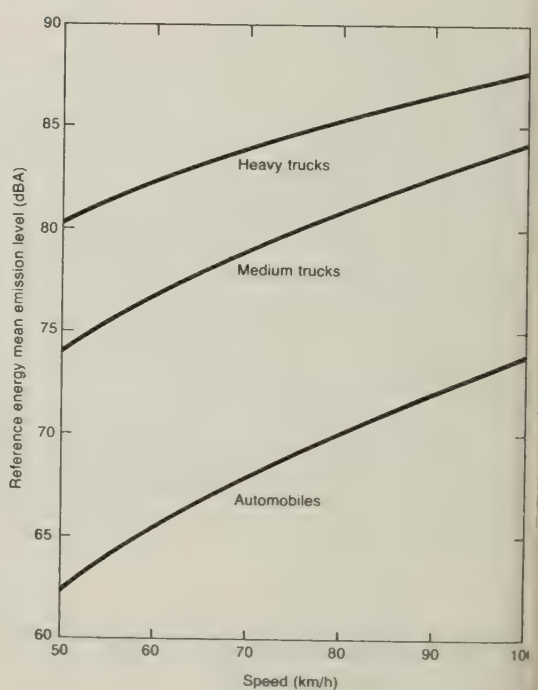
Step 2—Adjust for traffic flow.

The reference energy mean emission level represents the average peak passby noise level for a single vehicle in the specified vehicle class. To accommodate more than one vehicle, a traffic flow adjustment is used. The parameter used in the traffic flow adjustment is ND_o/S where N is the number of vehicles per hour for a given class, D_o is 15 m, and S is the class average speed in km/h. For example, for a road carrying 125 heavy trucks per hour at 95 km/h, the traffic flow parameter is 125 × 15/95 = 19.7. Referring to figure 6, the traffic flow adjustment is 13.0 dBA.

Step 3—Adjust for distance.

For observers situated more than 15 m from the road, an adjustment for distance is required. The distance adjustment requires two parameters: D_o/D and α. D_o is 15 m and D is the perpendicular distance between the centerline of the travel lane and the observer. The site parameter, α, determines the excess attenuation

Figure 5.—Reference energy mean emission levels.



characteristics of the site resulting from different ground covers. Field studies have shown that most highway sites can be characterized by $\alpha = 0$ if the intervening ground is relatively hard (reflective) or $\alpha = ½$ if the intervening ground is soft (absorptive) and contains significant vegetation. Figure 7 shows the distance adjustment for $\alpha = 0$ and $\alpha = ½$. As an example, if the setback distance D were 200 m, the distance adjustment would be −11.2 dBA if the site were hard or −16.9 dBA if the site were soft.

Step 4—Adjust for segments.

An adjustment for the length of the road must be made to calculate the noise from roadway segments that result from ramps, curved sections, sections exposed by topography, and sections in which there are significant changes in traffic volume or speed. The parameters required by the segment adjustment are α, ϕ_1, and ϕ_2, where α is the site parameter describing the excess attenuation characteristics of the segment and ϕ_1 and ϕ_2 are the angles defining the road segment relative to the receiver. Positive angles are measured to the right of the perpendicular line from the receiver to the road (or its extension), and negative angles are measured to the left of the perpendicular line. Figures 8 and 9 illustrate the segment adjustments for $\alpha = 0$ and $\alpha = ½$. As an example, for a segment defined by −40° and +60° the hard site segment adjustment is −2.6 dBA (for an included angle of 60−(−40) = 100°), while −2.9 dBA is the adjustment at a soft, absorptive site.

Step 5—Adjust for shielding.

A discussion of adjustments required by objects such as barriers, berms, and rows of houses that shield observers from the roadway noise is complex and beyond the scope of this article. Interested readers should review references 1 and 5.

Figure 6.—Adjustment for real traffic flows.

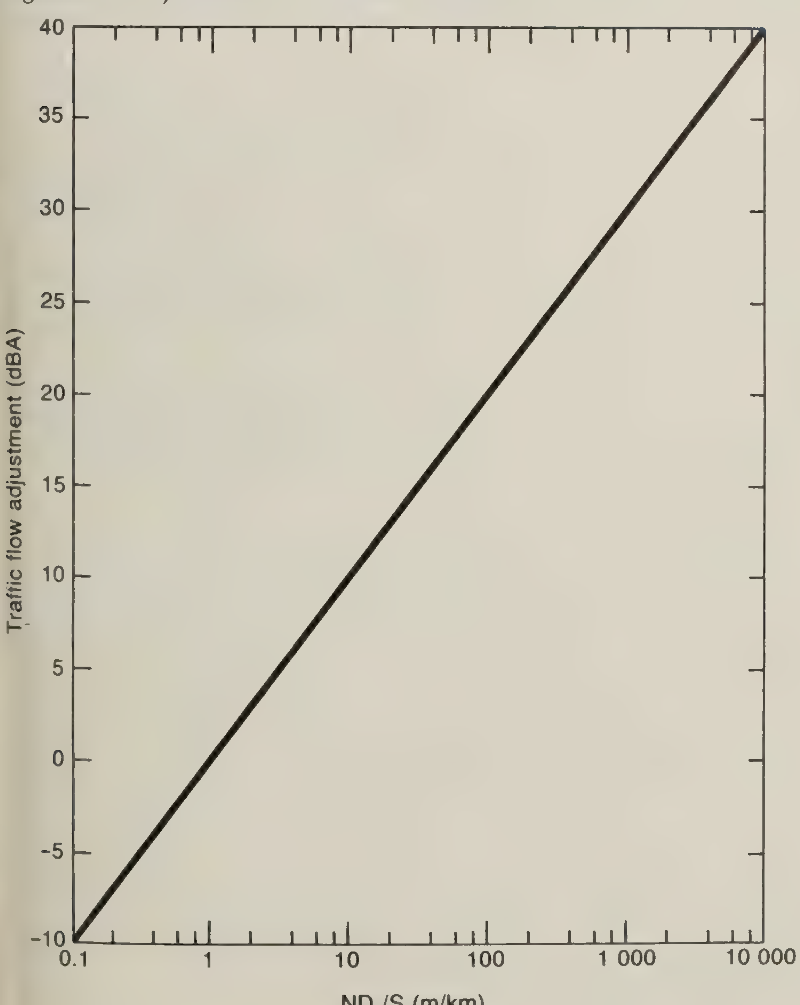
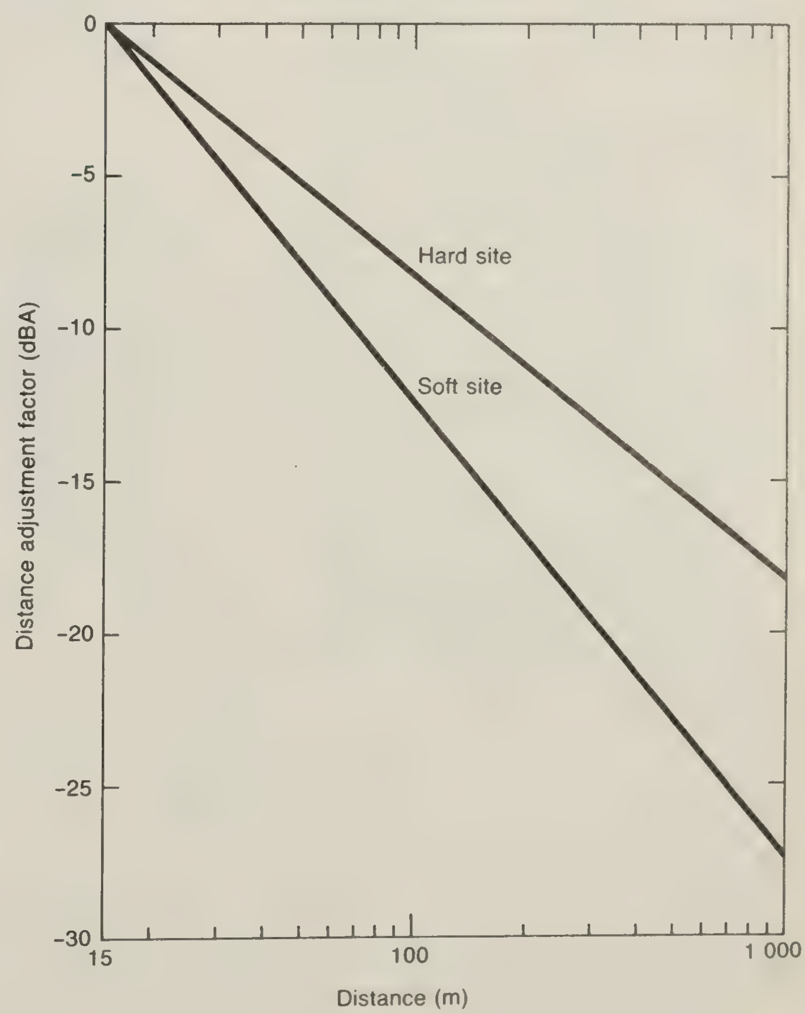


Figure 7.—Distance adjustment factors for distances greater than 15 m.



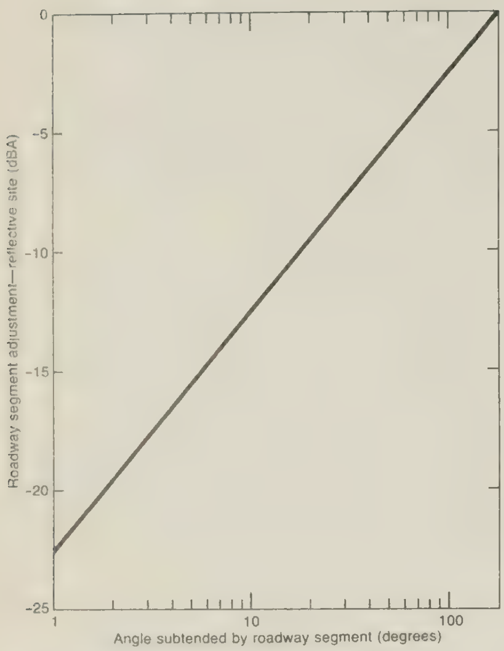
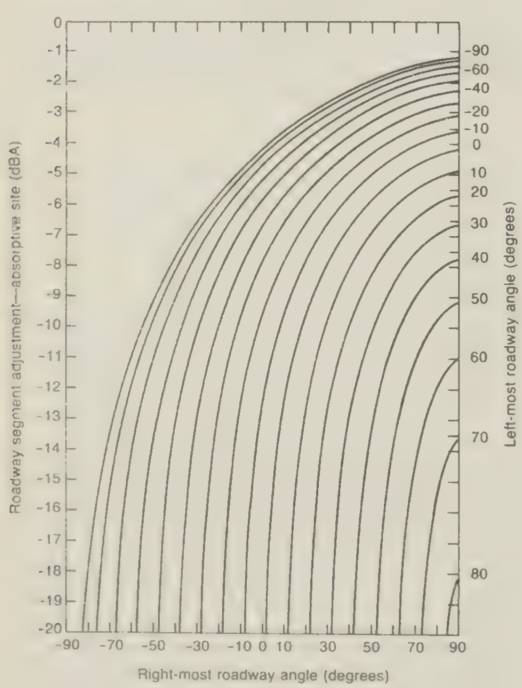


Figure 8.—Adjustment factor for finite length roadways (hard site).

Figure 9.—Adjustment factor for finite length roadways (soft site).



Step 6—Add a conversion constant.

The final adjustment to the base noise level is a constant, −25 dBA. This constant is required to compensate for the inconsistent units in the various adjustments. For example, although speeds are measured in kilometres per hour, distances are measured in metres. The conversion constant permits the use of these inconsistent units.

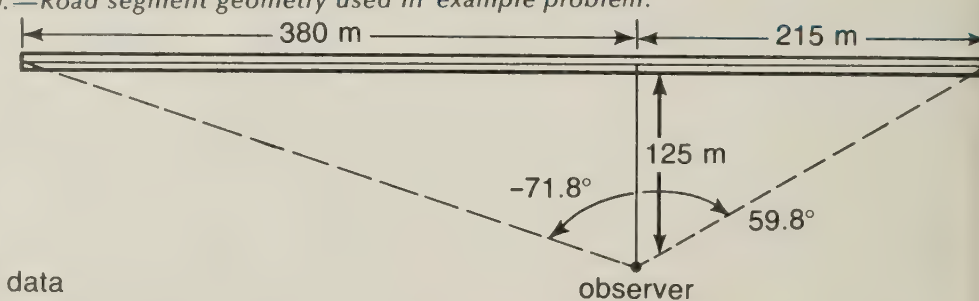
Example Problem

The FHWA highway traffic noise prediction model can be used to calculate the hypothetical problem shown in figure 10. The road segment carries the traffic volumes indicated. The chart below the diagram can be used to facilitate calculations and bookkeeping. The site is assumed to be acoustically absorptive ($\alpha = ½$). Calculation of the hourly equivalent sound level is straightforward and is seen to be 57.0 dBA.

Summary

The FHWA has developed and calibrated a model for predicting vehicular noise from highways. The model is based on a method of physically significant adjustments. Use of the model is straightforward and requires minimal training. The model is flexible in its design to allow the more advanced users to

Figure 10.—Road segment geometry used in example problem.



Traffic data

Vehicle class	*Hourly volume*	*Average speed (km/h)*
Automobiles	465	100
Medium trucks	43	90
Heavy trucks	35	95

Calculation of hourly equivalent sound level for example problem

		Automobiles		Medium trucks		Heavy trucks	
Step reference	Parameter	Parameter	Adjustment	Parameter	Adjustment	Parameter	Adjustment
1. Base level	S, speed	100	73.8	90	82.6	95	87.2
2. Flow	ND_0/S	69.75	18.4	7.17	8.6	5.53	7.4
3. Distance	D_0/D	0.12	−13.8	0.12	−13.8	0.12	−13.8
4. Segment	ϕ_1, ϕ_2	−71.8°, 59.8°	−1.9	−71.8°, 59.8°	− 1.9	−71.8°, 59.8°	− 1.9
5. Shielding	—	—	—	—	—	—	—
6. Constant			−25		−25		−25
Totals			51.5		50.5		53.9

Total A-weighted hourly equivalent sound level, L_{eq} (h)

L_{eq} (h) = 10 log $(10^{5.15} + 10^{5.05} + 10^{5.39})$ = 57.0 dBA modify it to meet specific site requirements. The development and uses of the model have been documented in detail. (*1*)

Five levels of sophistication have been developed to extend the usefulness of the FHWA traffic noise model. Users may match the degree of detail required by the site to the level of the model. Table 1 describes the five levels of the model.

Table 1.—Five levels of the FHWA highway traffic noise prediction model

Level	Descriptive title	Use
I	Nomograph	Quick estimates of noise levels at simple highway sites using nomographs. (*2*)
II	Manual method	Traffic noise estimates at simple sites using charts and tables to determine adjustments. Becomes tedious as site becomes complex. (*2*)
III	Hand-held calculator	Flow chart developed to permit users to program the model on hand-held calculators. Accurate, but tedious for complex sites. (*2*)
IV	Level I computer program, SNAP 1.0	Computer program for rapid determination of noise levels at moderately complex sites. Output is arranged in tabular form for direct inclusion in reports. (*6*)
V	Level II computer program, STAMINA 1.0	Computer program for noise calculations at simple to complex highway sites. (*7*)

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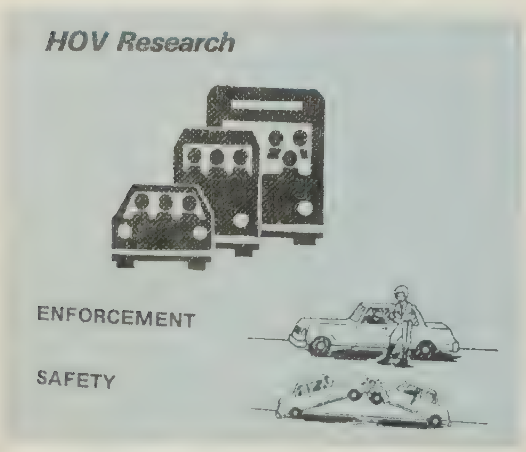
Timothy M. Barry is a research physicist in the Environmental Design and Control Division, Office of Research, Federal Highway Administration. He joined FHWA in 1975 and currently is task manager for FCP Project 3F4 "Noise and Vibration." Dr. Barry's technical background is environmental acoustics, and he is responsible for research studies concerned with the prediction and abatement of traffic-induced noise and vibration.

Recent Research Reports You Should Know About

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.

Enforcement Requirements for High-Occupancy Vehicle Facilities, Report No. FHWA-RD-79-15, and Safety Evaluation of Priority Techniques for High-Occupancy Vehicles, Report No. FHWA-RD-79-59



by FHWA Traffic Systems Division

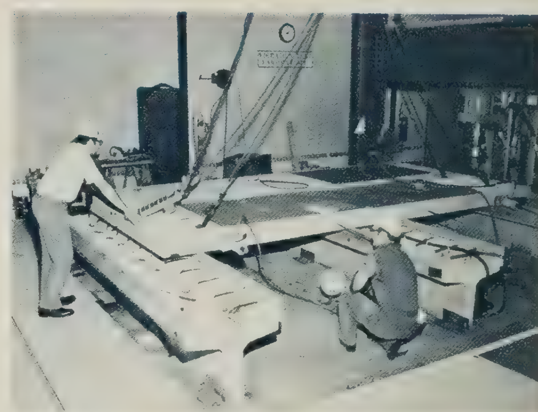
These reports review the operations of carpool/bus facilities throughout the United States. The priority techniques include restricted lanes on both freeways and arterial streets. The enforcement report presents guidelines for enforcing freeway and arterial treatments of high-occupancy vehicles. Innovative enforcement techniques are reviewed, including photographic instrumentation, mailing of citations, tandem (team) patrol, and use of paraprofessional officers. Legal issues are reviewed, and model legislation is drafted to enforce the treatment of high-occupancy vehicles.

The safety report examines accident rates, analyzes the causes of accidents, identifies potential safety problems, and recommends areas in which to improve safety. This study reviewed 22 high-occupancy vehicle projects on 16 highway facilities where data on safety operations and geometrics were collected and analyzed. Legal authority and liability issues associated with these special projects are also reviewed.

A limited number of copies of the reports are available from the Traffic Systems Division, HRS-33, Federal Highway Administration, Washington, D.C. 20590.

Polymer-Impregnated Precast Structural Concrete Bridge Deck Panels, Report No. FHWA-RD-75-121

by FHWA Structures and Applied Mechanics Division



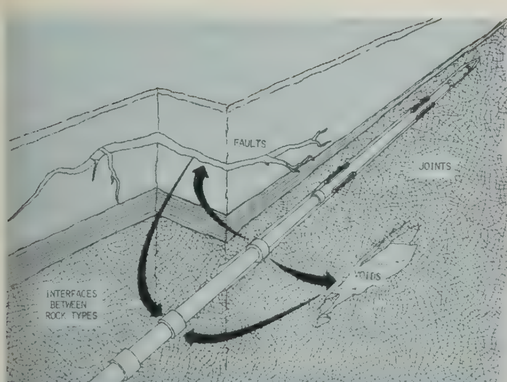
Research has shown that polymer-impregnated concrete (PIC) is a durable material under extended freeze-thaw cycling, is inert to most chemicals, has near-zero water absorption, and is essentially impermeable. Its compressive and tensile strengths are much greater than those of normal concrete. This report describes the design, impregnation, polymerization, physical testing, and evaluation of precast, prestressed PIC bridge deck panels.

Four basic steps were involved in the impregnation/polymerization process—drying the concrete panels, evacuating air from the dried panels, impregnating the panels by immersing them in monomer under pressure, and polymerizing the monomer in the panels by applying heat. The panels were not merely surface treated but were fully impregnated and would be highly resistant to the adverse effects of deicing salts. Two separate and independent panel designs were made for a hypothetical 27 m span, HS20–44 design bridge. Pretensioned panels, 1.2 m by 4.9 m by 150 mm thick, were placed transversely across supporting girders on 2.4 m centers.

The research program included static loading to failure, fatigue tests, horizontal shear tests of the connection between panel and girder, physical property tests, and prestress loss determination.

Actual applications of PIC precast bridge deck panels should result in innovations for fabricating, processing, and erecting these panels. Although PIC deck panels may have a significantly higher initial cost than conventional concrete decks, the PIC deck panels should prove cost effective because they should require almost no maintenance.

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 279885).



A New Sensing System for Pre-excavation Subsurface Investigation for Tunnels in Rock Masses, Volume I—Feasibility Study and System Design (Report No. FHWA-RD-77-10) and Volume II—Appendixes: Detailed Theoretical, Experimental, and Economic Foundation (Report No. FHWA-RD-77-11)

by FHWA Structures and Applied Mechanics Division

These reports describe a proposed downhole field instrumentation system for improved pre-excavation subsurface investigations for tunneling in rock. Costs of tunneling in rock have often risen far above initial estimates because rock conditions could not be accurately predicted from surface geology, vertical borings, or pilot tunnels.

The design specification for the prototype system covers the above-ground control and support functions, the downhole sensing package, and the physical and electrical links to the surface. The sensing package includes three subsystems—a ground-probing radar, acoustic transmitting and receiving sensors, and an array of electrodes for resistivity measurements. A controllable nose cone guides the sensing package into the predrilled borehole; data are taken as the sensors are being retracted.

The combination of three sensing modes provides detailed information on rock characteristics in the immediate vicinity of the borehole and general information at distances up to 30 m. The radar and acoustic return signals permit definition of joint planes, angular orientations, voids, water-bearing cavities and faults, changes in rock types and strengths, and many other underground features. The resistivity mode senses the bulk characteristics of the surrounding rock medium and can penetrate to distances equivalent to the electrode spacing which may be adjusted to meet the needs of the user. The quality of the data improves as the diameter of the horizontal predrilled borehole increases, up to a diameter where the necessary contact of the sensor with the borehole wall cannot be maintained.

Initially, a prototype of this system will be used to collect signatures of known geologic phenomena which will be stored in computer data banks. Similar signatures from other sources will be correlated with the information in the data bank to identify the geologic features.

The reports are available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock Nos. PB 276720 and PB 276721).

Design and Construction of Compacted Shale Embankments, Volume 4 (Report No. FHWA-RD-78-140) and Volume 5 (Report No. FHWA-RD-78-141)

by FHWA Materials Division

Shale, used for embankments in many locations, can cause excessive settlements and slope failures. These reports and the three preceding volumes identify the shales that cause these problems and develop methodologies to use these shales successfully in future construction.

Volume 4, **Field and Laboratory Investigations, Phase III,** presents the results of investigations of six highway embankment sites selected to cover a range of ages, various degrees of distress, and different types of construction. Undisturbed samples from the embankments provided information on the character of embankment materials, degree of shale deterioration, in situ density, water content, compressibility, and shear strength. Unweathered parent shale samples were used to determine durability indexes for comparison with in situ conditions and service performance. The type of accelerated weathering index test that can predict long term performance of compacted shales was also investigated.

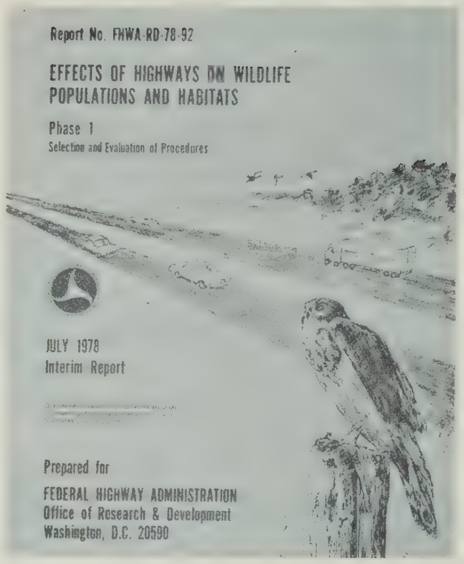
Volume 5, **Technical Guidelines,** outlines geological investigations, durability classifications, design features, and construction procedures for compacted shale embankments for highways.

Techniques are given for evaluating existing shale embankments and remedial treatment methods are provided for distressed shale embankments. Index tests and classification criteria for determining shale durability and techniques for evaluating excavation characteristics, alternative excavation procedures, and placement and compaction of shales to achieve adequate stability and minimum settlement are described. The use of drainage measures, selective grading, and nondurable shales in thin lifts with procedural compaction provisions based on field test pads is emphasized.

The reports are available from the Materials Division, HRS-21, Federal Highway Administration, Washington, D.C. 20590.

Effects of Highways on Wildlife Populations and Habitats: Phase I, Selection and Evaluation of Procedures, Report No. FHWA-RD-78-92

by FHWA Environmental Design and Control Division



State highway agencies are required to assess the impact of new highway construction, operation, and maintenance on wildlife. Because of this, a study was initiated to determine the effects of highways on wildlife populations and habitats, develop a system to help predict and evaluate the possible impacts, and recommend ways to minimize the adverse effects and maximize the beneficial ones.

The study was divided into two phases. This report discusses Phase I research—selecting, testing, and evaluating procedures for rapid, efficient assessment of wildlife populations and habitats in relation to highways. The results of studies conducted in different parts of the United States and designed to recommend wildlife assessment procedures are given. The procedures and techniques presented in this report were developed specifically for use in Phase II of this study, but they also should be applicable for immediate use by highway personnel in assessing the effects of their actions on the environment.

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 293796).

Comparison of California Accident Rates for Single and Double Tractor-Trailer Combination Trucks, Report No. FHWA-RD-78-94

by FHWA Environmental Design and Control Division

Truck accident rates were analyzed to determine the relative safety of single and double truck combination vehicles. The single combination type is a tractor unit attached to a semitrailer, and the double combination type is a tractor, semitrailer, and full trailer, in that order.

Estimates of truck exposure data were combined with 1974 California accident data to give accident and injury rates based on vehicle-kilometres of travel and average cargo weights. These measures permitted a safety evaluation of the two combination vehicle types. Significant differences in accident and injury rates were obtained.

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, Office of Development, Federal Highway Administration (FHWA). Some items by others are included when they have a special interest to highway agencies.

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

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

Emergency Escape Ramps for Runaway Heavy Vehicles, Report No. FHWA-TS-79-201

by FHWA Implementation Division

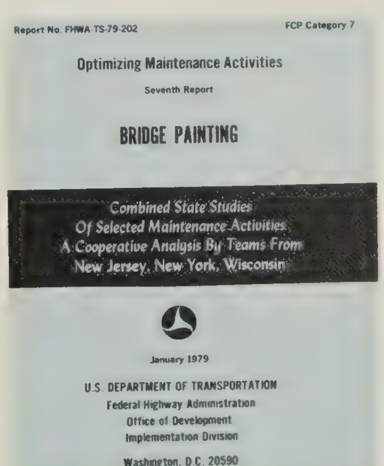
Runaway heavy vehicles often cause fatalities and property damage. Emergency escape ramps can safely stop these vehicles; however, there are no nationally accepted design standards for these safety devices.

This report and slide-tape presentation give a state-of-the-practice synopsis of escape ramp technology, findings of a current survey, and an overview of the design, construction, and operation of existing ramp facilities.

The escape ramps are classified into three general categories: Gravity type, arrester beds, and a combination of these. It was found that no one type of escape ramp is feasible in all situations. Factors to be considered in escape ramp design include terrain, ramp dimensions, delineation, advance signing, and service roads. Also vital to the successful design of escape ramps is a public awareness program to familiarize motorists with the purpose and operations of emergency escape ramps.

The report is available from the Implementation Division. The slide-tape presentation is available from FHWA regional offices (see inside back cover), FHWA division offices, and FHWA National Highway Institute.

Optimizing Maintenance Activities, Seventh Report, Bridge Painting, Report No. FHWA-TS-79-202


Report No. FHWA-TS-79-202 FCP Category 7

Optimizing Maintenance Activities

Seventh Report

BRIDGE PAINTING

Combined State Studies
Of Selected Maintenance Activities
A Cooperative Analysis By Teams From
New Jersey, New York, Wisconsin

January 1979

U.S. DEPARTMENT OF TRANSPORTATION
Federal Highway Administration
Office of Development
Implementation Division
Washington, D.C. 20590

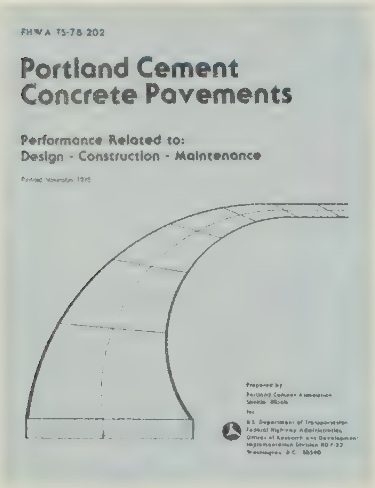
by FHWA Implementation Division

This report, the seventh in a special series on highway maintenance, summarizes the results of a bridge painting value engineering study in New Jersey, New York, and Wisconsin. All aspects of bridge painting activities—material, equipment, and labor—are analyzed. The report includes recommendations for painting cycles, implementing an inspection program, using contract painting, and training bridge painting inspectors. These recommendations pertain to either maintaining present maintenance policies or implementing revised or new policies to improve the product and reduce costs. Successful implementation of the recommendations in the three study States would save an estimated $350,000 annually.

This report is intended for practitioners familiar with the subject matter. It has been assumed that the reader has a basic understanding of the problems and procedures in managing a bridge painting program.

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

Portland Cement Concrete Pavements, Performance Related to: Design-Construction-Maintenance, Report No. FHWA-TS-78-202



by Portland Cement Association and FHWA Implementation Division

There are over 185 000 km of portland cement concrete (PCC) pavements in the United States. Many older pavements are still structurally sound while some newer pavements have shown distress after only a short period of service. Design, construction, and maintenance techniques were studied to find reasons why similarly designed pavements perform differently.

This report and a slide-tape presentation give results of an evaluation of PCC pavement distress and recommend solutions to the problem. Detailed surveys were made on 85 pavement projects in 7 States. These pavements, generally less than 15 years old, included plain, plain doweled, conventionally reinforced, and continuously reinforced pavements.

The surveys covered various aspects of PCC pavements including categories, characteristics, condition, and performance of the pavements; design and construction practices of the States; construction and maintenance costs; and design and construction recommendations.

Recommendations contained in the report include increased use of dowels for load transfer, short joint spacing, depths of D/4 for transverse and D/3 for longitudinal joints, tied shoulders, and good drainage.

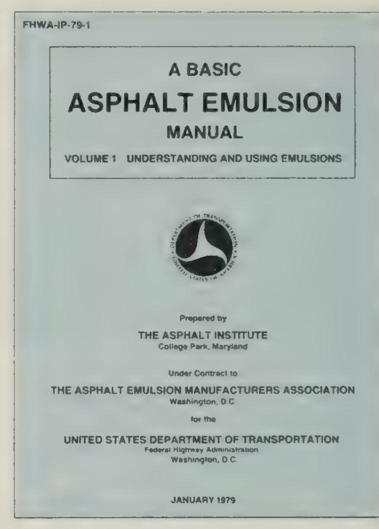
Limited copies of the report are available from the Implementation Division. The slide presentation with script, which generally follows FHWA policy and guidance, is available from FHWA regional offices (see inside back cover).

A Basic Asphalt Emulsion Manual, Volumes 1 and 2, Implementation Package 79-1

by FHWA Implementation Division

Asphalt emulsions, used in lieu of asphalt cutbacks for road construction and maintenance activities, will reduce energy consumption and air pollution. These two manuals were prepared to present a clear understanding of asphalt emulsions.

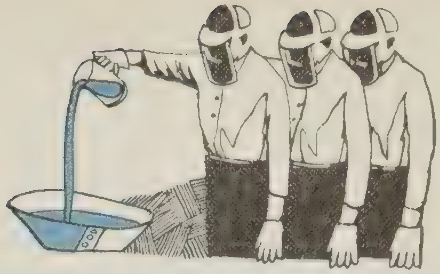
Volume 1, **Understanding and Using Emulsions,** presents general characteristics and uses of asphalt emulsions. This information will be



helpful in choosing the best emulsion for a particular project and in solving problems that may arise on projects where emulsions are used.

Volume 2, **Mix Design Methods,** outlines 11 design methods varying in degree of usefulness. Each of these methods uses some parts or modifications of the standard Marshall or Hveem test methods. Although a universally accepted emulsion-aggregate mix design method has not been developed, The Asphalt Institute and the Illinois methods (described in this volume) are probably the best for designing emulsified asphalt-aggregate mixtures using the standard test methods.

Limited copies of the manuals are available from the Implementation Division.



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 1I: Traffic Lane Delineation Systems for Adequate Visibility and Durability

Title: Investigate Alternatives for Solvent Based Traffic Paints. (FCP No. 41I2253)
Objective: Evaluate pavement marking systems in terms of their suitability for highway delineation and their ability to comply with air pollution regulations.
Performing Organization: California Department of Transportation, Sacramento, Calif. 95805
Expected Completion Date: September 1981
Estimated Cost: $140,000 (HP&R)

FCP Project 1K: Accident Research and Factors for Economic Analysis

Title: Analysis of Urban Arterial Road and Street Accident Experience. (FCP No. 31K1068)
Objective: Identify and determine the general dimensions of, specific characteristics of, and appropriate countermeasures for the urban arterial (nonfreeway) motor vehicle accident program.
Performing Organization: Goodell-Grivas Inc., Southfield, Mich. 48075
Expected Completion Date: March 1981
Estimated Cost: $413,000 (FHWA Administrative Contract)

FCP Project 1M: Operational Safety Improvements for Two-Lane Rural Highways

Title: Warning Signs and Advisory Speed Signs—Reevaluation of Practice. (FCP No. 41M2152)
Objective: Investigate drivers' visual fixation patterns in view of warning and advisory speed signs.
Performing Organization: Ohio University, Athens, Ohio 43215
Funding Agency: Ohio Department of Transportation
Expected Completion Date: September 1981
Estimated Cost: $131,000 (HP&R)

FCP Project 1X: Highway Safety Program Effectiveness Evaluation

Title: Determination of the Operational Performance Requirements for a Roadside Accident Countermeasure System. (FCP No. 31X3082)
Objective: Use accident data and operational data on roadside encroachments to identify hazardous locations, evaluate the degree of protection required, and select appropriate countermeasures.
Performing Organization: Southwest Research Institute, San Antonio, Tex. 78284
Expected Completion Date: September 1980
Estimated Cost: $548,000 (FHWA Administrative Contract)

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

FCP Project 3F: Pollution Reduction and Environmental Enhancement

Title: Vehicle Emissions at Intersections. (FCP No. 43F3532)
Objective: Simulate intersection traffic and predict emissions by use of the Texas Traffic Simulation Model and U.S. Environmental Protection Agency emission factors. Collect and analyze pollutant concentrations and dispersion at intersections.
Performing Organization: Texas State Department of Highways and Public Transportation, Austin, Tex. 78701
Expected Completion Date: August 1980
Estimated Cost: $193,000 (HP&R)

FCP Category 4—Improved Materials Utilization and Durability

FCP Project 4C: Use of Waste as Material for Highways

Title: Softening or Rejuvenating Agents for Recycled Bituminous Binders. (FCP No. 34C4043)
Objective: Develop tests and techniques to evaluate the compositions of salvaged bituminous binders. Determine the necessary properties and composition of rejuvenators to restore the salvaged binder to a condition suitable for paving purposes with a reasonably defined performance.

Performing Organization: Texas Transportation Institute, College Station, Tex. 77243
Expected Completion Date: September 1980
Estimated Cost: $132,000 (FHWA Administrative Contract)

Title: Test for Efficiency of Mixing Recycled Asphalt Pavements. (FCP No. 34C4053)
Objective: Develop a test method and necessary test equipment to evaluate the mixing efficiency for recycled asphalt pavement materials. Determine if recycled paving mixes are homogenous.
Performing Organization: University of Washington, Seattle, Wash. 98195
Expected Completion Date: September 1980
Estimated Cost: $142,000 (FHWA Administrative Contract)

FCP Project 4D: Remedial Treatment of Soil Materials for Earth Structures and Foundations

Title: Field Calibration of Electric Cone Penetrometers in Soft Soils. (FCP No. 44D5154)
Objective: Develop reliable estimates of undrained shear strength by electric quasi-static cone penetration tests in cohesive soils.
Performing Organization: Louisiana State University, Baton Rouge, La. 70803
Funding Agency: Louisiana Department of Transportation and Development
Expected Completion Date: September 1980
Estimated Cost: $189,000 (HP&R)

FCP Project 4G: Substitute and Improved Materials to Reduce Effects of Energy Problems on Highways

Title: Determination of Oxygen and Moisture Levels in Structural Concrete. (FCP No. 34G3113)
Objective: Develop field methods for making long term local measurements of the oxygen concentration and the free moisture content in portland cement concrete structures at the level of the reinforcing steel.
Performing Organization: Southwest Research Institute, San Antonio, Tex. 78284
Expected Completion Date: September 1980
Estimated Cost: $218,000 (FHWA Administrative Contract)

FCP Project 4H: Improved Foundations for Highway Structures

Title: Field Study of Pile Group Action. (FCP No. 34H2012)
Objective: Load test an experimental group of 9 piles to failure 7, 28, and 60 days after driving. Repeat the load tests on subgroups of five and four piles, respectively. Instrument all piles for load transfer analysis.
Performing Organization: University of Houston, Houston, Tex. 77027
Expected Completion Date: September 1980
Estimated Cost: $600,000 (FHWA Administrative Contract)

Title: An Investigation of Scale Effects Between Model and Full-Scale Pile Groups. (FCP No. 24H2032)
Objective: Conduct axial load tests on instrumented model single piles and pile groups. Compare the load deformation and load transfer behavior with that of corresponding full-scale pile groups and single piles.
Performing Organization: Federal Highway Administration, Washington, D.C. 20590
Expected Completion Date: September 1980
Estimated Cost: $156,000 (FHWA Staff Research)

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

FCP Project 5A: Improved Protection Against Natural Hazards of Earthquake and Wind

Title: Pasco-Kennewick Bridge Wind and Structure Motion Study. (FCP No. 35A1092)
Objective: Determine the characteristics and magnitudes of natural wind and bridge motion at the site of an existing inland, long-span, cable-stayed bridge.
Performing Organization: Battelle Pacific Northwest Laboratory, Richland, Wash. 99352
Expected Completion Date: March 1981
Estimated Cost: $117,000 (FHWA Administrative Contract)

Title: Earthquake Resistant Bridge Bearings. (FCP No. 35A2092)
Objective: Develop new designs for fixed and expansion bridge bearing devices to provide adequate superstructure restraint against earthquake imposed loading forces.
Performing Organization: Engineering Computer Corporation, Sacramento, Calif. 95825
Expected Completion Date: July 1980
Estimated Cost: $121,000 (FHWA Administrative Contract)

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

Title: Wave Forces on Causeway-Type Coastal Bridges: Effects of Angle of Wave Incidence and Cross Section Shape. (FCP No. 45H4852)
Objective: Conduct a hydraulic laboratory study of dynamic wave forces acting on two types of bridge (longitudinal beam and slab and a box girder) when the waves have various angles of incidence. Conduc the study in a three-dimensional wave basin 11.6 m by 12.2 m in plan
Performing Organization: Mississippi State University, Jackso Miss. 39206

☆U.S. Government Printing Office: 1979—620-103/

Funding Agency: Mississippi State Highway Department
Expected Completion Date: October 1980
Estimated Cost: $69,000 (HP&R)

FCP Project 5K: New Bridge Design Concepts

Title: Segmental Polymer-Impregnated Prestressed Concrete Hollow-Core Bridge Decks. (FCP No. 35K1012)
Objective: Evaluate the feasibility of using polymer-impregnated prestressed concrete hollow-core members for a segmental bridge deck. Develop a design, perform structural testing, and perform analysis and make recommendations for proposed usage.
Performing Organization: U.S. Army Concrete Engineering Research Laboratory, Champaign, Ill. 61820
Expected Completion Date: September 1980
Estimated Cost: $165,000 (FHWA Administrative Contract)

Title: Applications of High Strength Concrete for Highway Bridges. (FCP No. 35K1022)
Objective: Develop applications of high strength concrete for highway bridges through analytical design studies, structural laboratory testing, and development of recommended revisions to the controlling design codes.
Performing Organization: Concrete Technology Corporation, Tacoma, Wash. 98421
Expected Completion Date: September 1980
Estimated Cost: $221,000 (FHWA Administrative Contract)

FCP Project 5L: Safe Life Design for Bridges

Title: Weighing Trucks-In-Motion With Instrumented Highway Bridges. (FCP No. 45L3062)
Objective: Design and test a stand-alone prototype instrumentation and processing system for weighing trucks-in-motion. Deliver to State personnel for operation. Test the system for planning surveys in conjunction with currently available portable scales.
Performing Organization: Case Western University, Cleveland, Ohio 44106
Funding Agency: Ohio Department of Transportation
Expected Completion Date: December 1980
Estimated Cost: $179,000 (HP&R)

FCP Category 0—Other New Studies

Title: Significant Factors in Truck Ride Quality. (FCP No. 30B5114)
Objective: Identify factors which contribute significantly to differences in ride quality between various truck models and configurations over a range of realistic operating conditions. Restrict measurements to the cab environment. Record and analyze vibrations, noise, temperature, and air quality.
Performing Organization: Systems Technology Inc., Hawthorne, Calif. 90250
Expected Completion Date: September 1980
Estimated Cost: $249,000 (FHWA Administrative Contract)

Title: Mixing Water and Maturity of Hardened Concrete Using SEM Techniques. (FCP No. 40M3533)
Objective: Develop visual comparison standards of SEM photographs to permit determination of original water-cement ratio and degree of hydration of hardened portland cement concrete.
Performing Organization: North Carolina State University, Raleigh, N.C. 27650
Funding Agency: North Carolina Department of Transportation
Expected Completion Date: June 1980
Estimated Cost: $56,000 (HP&R)

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