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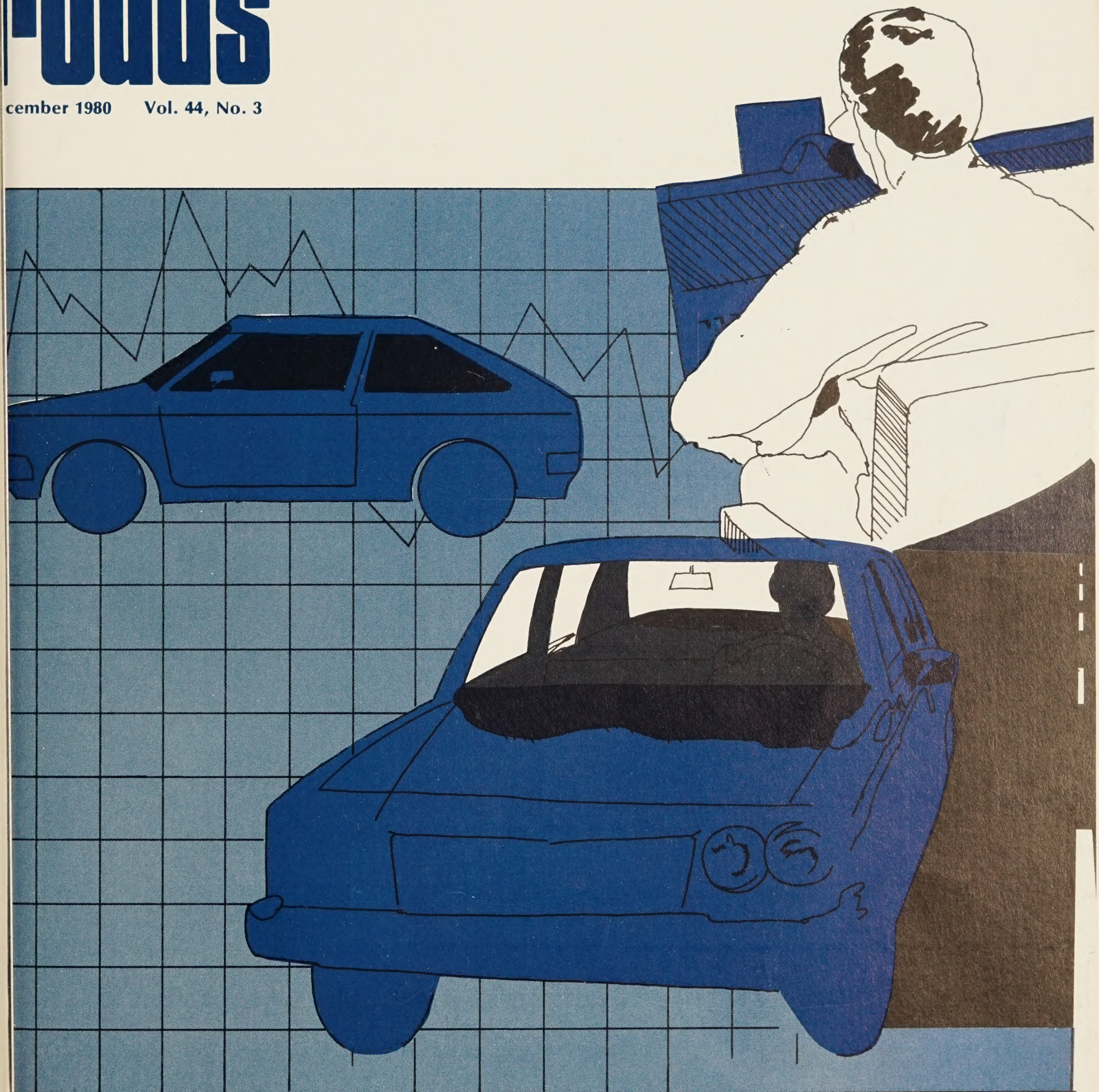

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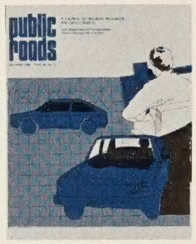

COVER:

Simulation plays a key role in driver-roadway interaction research.

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Federal Highway Administration
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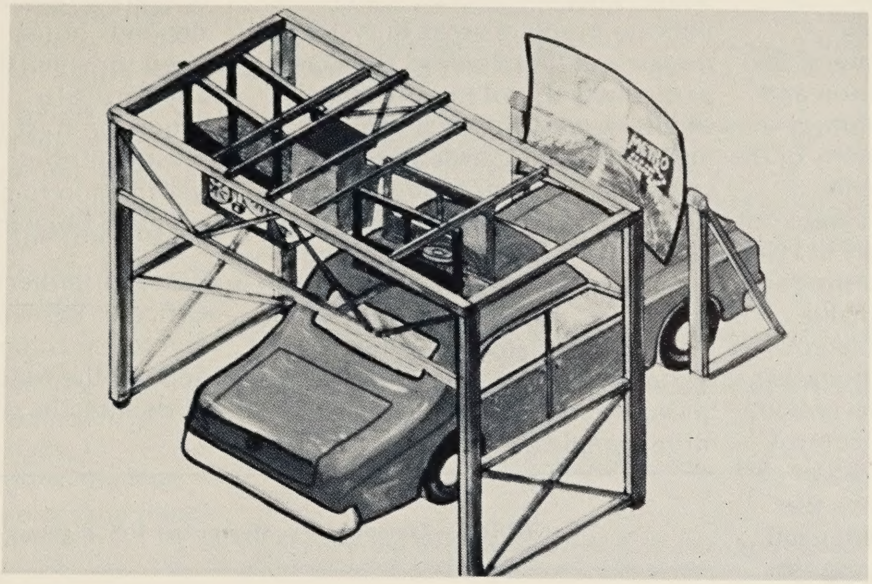
The FHWA Highway Driving Simulator



by
King M. Roberts

Introduction

The Federal Highway Administration (FHWA) Office of Research is developing a highway driving simulator (HYSIM) to study driver-roadway interactions. A modular design is being used to provide a basic, first generation system. The HYSIM will be used for a range of FHWA research projects and will be enhanced, as needed, to respond to changes in research direction and new research requirements.

This article gives an overview of the evolution of driving simulators and summarizes the HYSIM procurement and design.

Overview of Simulation

During World War II, there was an urgent need to train large numbers of military personnel in the tactical use of war machinery. Field training was undesirable from the standpoints of safety, logistics, cost, and time. Training simulators provided a practical solution. As experience with simulation was gained, it became more apparent that simulation training was safer, quicker, and less costly than "real-world" training operations.

Simulators are widely used in military training today. For example, the Naval Training Equipment Center in Orlando, Fla., spent $71 million in fiscal year 1966 for simulation training compared with $270 million in fiscal year 1977.(*1*)[1] Simulator training programs currently are being conducted by all branches of the armed services as well as other Federal and private agencies. Commercial airlines now train and certify their pilots using simulators that reproduce the characteristics of real-world aircraft counterparts. The range of operational training simulators includes fixed and rotary wing aircraft, spacecraft, tanks, ships, trains, and automobiles.

As weapon systems became more complex, the need to evaluate, optimize, and integrate system and human performance became more critical. Physical systems could be tested to destruction, whereas humans could not. Yet, many systems required operational evaluation in situations unsafe for humans. Fortunately, the development of training simulators created an inventory of simulation techniques for performing various simulation functions. These developments, during the 1950's, made it possible to construct cost-effective research simulators for human factors evaluations.

[1] Italic number in parentheses identifies reference on page 102.

Highway research simulators were developed in the late 1950's, and the first actual highway simulators were operated in the early 1960's. The FHWA constructed simplistic simulators to investigate one or two variables affecting recognition and interpretation of signed highway information, but the credibility of the simulations was questionable because the visual displays were not realistic. The state of the art in visual displays was insufficient to produce highway scenes with acceptable visual accuracy at reasonable cost. In addition, computer technology was not sufficiently advanced to provide the sophisticated levels of control required in large-scale simulator systems. Other organizations that developed simulators experienced similar difficulties, and during the mid-1960's highway simulator activity declined.

The state of the art in visual displays and computer technology advanced rapidly in the late 1960's. Much of the technology was developed by the National Aeronautics and Space Administration (NASA) to support its space program. These improvements renewed interest in highway simulation techniques; by 1975, several "driving" simulators were operating throughout the United States. A representative sample of contemporary U.S. driving simulators—classified by facility, kind, and use—is shown in table 1.

Technical problems in simulator design are being resolved. These resolutions have allowed the development of more complex research simulators. These simulators are more general in purpose and flexible in nature than training simulators. Simulators are no longer restricted to addressing only one or two specific aspects of driving. The major advantage of these simulators is their versatility. They can be easily and economically configured to simulate a variety of human factors research problems. They allow evaluation and optimization of human performance within system constraints and indicate problem areas in system design and functioning. They are particularly useful in selecting a viable system approach from numerous alternatives and evaluating system performance before field testing. The major disadvantage of research simulators is their high initial acquisition cost. In addition, operation and maintenance costs for research simulators are slightly higher than for training simulators because research simulators are more complex.

Why Procure a Simulator?

Whether or not to procure and operate a research simulator depends primarily on the level of need for simulation capabilities in any particular research program. A simulator must be cost effective and must contribute timely, meaningful information to aid in meeting program objectives.

Research projects within the FHWA's Office of Research were analyzed to determine common activities that defined the functional requirements of a simulator responsive to program

Table 1.—Examples of contemporary U.S. highway simulators

Facility	Kind	Used to study
Systems Technology	Computer-generated CRT	Driver performance, physiology, vehicle dynamics
University of Missouri, Rolla	Closed-circuit TV	Driver performance, improvement, physiology, driver evaluation, improving vehicular design
University of South Dakota	Point light source	Driver performance, improvement, physiology
Ohio State University	Moving-base, car following	Driver performance, improvement, physiology
University of California	Cinematic and closed-circuit TV	Improvement of driver, road, signing, signaling, delineation, vehicular design
Central Missouri State University	Cinematic	Driver training
Wayne State University	Cinematic	Improvement of road, signing, signaling, delineation, vehicular design
University of Michigan	Moving belt	Improvement of vehicular design, driver evaluation, driver performance, physiology
Virginia Polytechnic Institute	Computer-generated CRT	Improvement of driver, road, signing, signaling, delineation, vehicular design
Ohio State University	Cinematic	Improvement of driver, road, signing, signaling, delineation, basic driver performance, physiology
Liberty Mutual	Point source	Driver improvement
University of California, Los Angeles	Cinematic	Basic research, physiology, driver evaluation, improvement of road, signing, signaling, delineation

needs. These common activities include the following:

• *Identification of driver's needs:* Studies are needed to identify the kind, quantity, and quality of information that drivers require to operate safely and efficiently in various roadway situations and conditions. Once identified, the informational elements for any specific situation must be prioritized relative to their contribution to satisfying driver information requirements.

• *Preliminary evaluation:* The effectiveness of large numbers of competing candidate signs, markings, or other highway devices must be evaluated to select the most suitable devices.

• *Optimization of subsystems:* During the development of complex highway information systems, studies are required to select and optimize subsystems to accommodate driver requirements.

• *Evaluation of system interactions:* Before field testing, studies are required to evaluate the entire system to optimize driver performance. These studies "fine tune" the system by indicating functioning weaknesses and identifying areas where redesign or modifications are needed.

These common activities usually must be performed to successfully complete complex FHWA Office of Research projects that implement roadway information systems. The kinds of activities probably will not change substantially regardless of future research direction. Most of the listed activities, almost exclusively within the domain of human factors research, can be performed efficiently using HYSIM.

The HYSIM can be used effectively for preliminary screening or in some cases as an exclusive alternative to field testing in the research projects. It also can reduce the costs and enhance the safety of field investigations.

Table 2.—Typical problems for investigation using HYSIM

Test and evaluation of experimental signs, signals, markings, roadway lighting, and geometric designs.
Tests of in-vehicle audio systems intended to warn the driver of sign messages or dangerous conditions ahead ("Talking Signs").
Pretests of warning devices, signs, signals, and markings for narrow bridges, restricted roadways, and dangerous rural intersections.
Tests of ability of the driver to see signs and road markings where there are cars, trucks, trailers, and recreational vehicles ahead.
Evaluation of road markings codes (color and shape) for route guidance.
Tests and evaluations of diversion and changeable message signs.
Measurement of driver responses under stress of sleeplessness, fatigue, noise, vibration, heat, and cold.
Tests of the effects of midblock conflict where there are vehicles entering and leaving from within the section.
Studies of driver information overload in urban areas where the driver must react to vehicles ahead, traffic signals, law officers, and intruding pedestrians.
Studies of signs and markings for exclusive or contraflow bus and carpool lanes.
Freeway on-ramp metering controls (markings, signals, instruction signs) and active metering operations while merging.
Studies of trade offs between longer messages on active-freeway-advisory-signs and driving errors.
Possible conflict between head up displays of automobile status (for example, windshield-projected speedometer) and highway delineations, signs, and signals.
Evaluation of proposed standards.

HYSIM Description

The highway simulator being constructed for the FHWA is composed of several subsystems, or modules, that operate in various configurations to meet simulation requirements for a range of highway research problems. Table 2 lists typical FHWA research problems to be investigated using HYSIM. The simulator is modularly designed to permit maximum system flexibility, retain partial operational capability if a subsystem fails, and facilitate modification or augmentation of system configuration or capabilities without substantial adverse impact on research schedules.

A facility floor plan is shown in figure 1. Major hardware modules are shown in figure 2. A brief discussion of the purpose and function of each module follows.

Car cab module

This module provides a realistic automobile environment for the driver. The cab, a modified 1980 Ford Fairmont, includes all controls

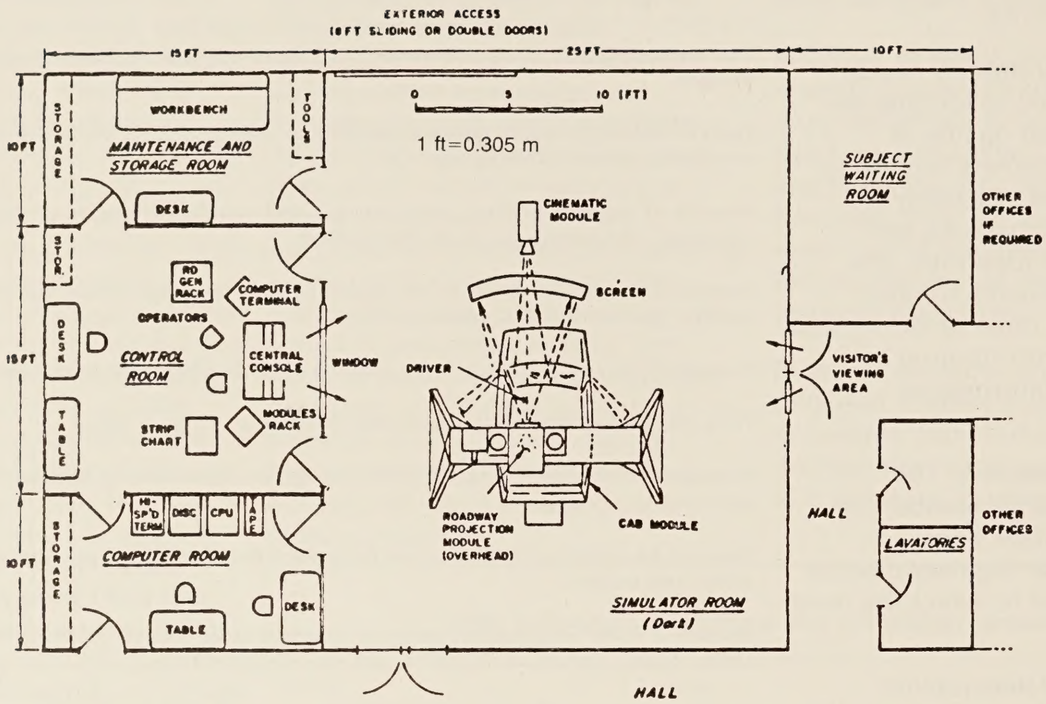


Figure 1.—Highway simulator facility.

(accelerator, brake, steering wheel), dashboard instruments, signals, and other devices normally installed in a car.

Driver manipulation of controls can be measured. The module is interfaced with the vehicle dynamics module to permit steering and speed control, the scenario computer to provide monitoring, control, and performance functions, the operator's station, and the auditory module.

Vehicle dynamics module

This module receives input from the car cab controls (steering, throttle, and brake) and the scenario computer (wind gusts and roadway disturbances). It computes the car's speed, direction, and track error, with representative vehicle dynamics that can be selected to simulate different kinds of vehicles and tire/roadway conditions. Its output goes to the scenario computer for further computation of route progress and driver performance and scoring of the drivers' performance data; to the roadway generator and/or cinematic module to provide fully or partially interactive visual displays; and to the auditory module for sound effects.

Digital roadway generator module

The effects of signs, novel delineations and lighting situations, unexpected road obstacles, and wet and icy roads on driver/vehicle control can be studied with this module and the roadway projection system. The module can display two

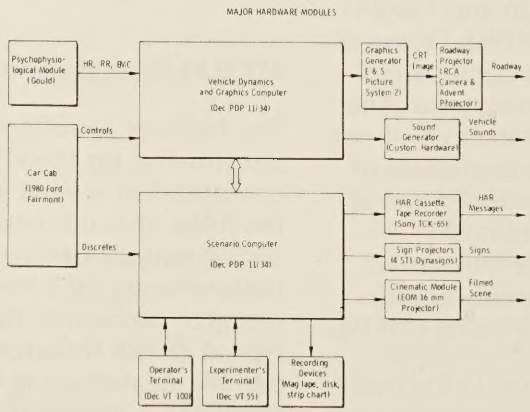


Figure 2.—Major hardware modules.

separate road lanes and various road geometrics including straight or curved sections, intersections, merges, and diverges. Traffic signals, signs, and obstacles on adjacent lanes or lanes ahead are displayed.

Perturbations to the car, such as wind gusts, which require the driver to rectify his or her course and direction by corrective steering or braking can be induced.

Cinematic module

The purpose of this module is to permit study of drivers' responses to signs, signals, and markings under variable conditions. It provides great flexibility in presentation, requiring, for example, only a quick film change to go from city streets at night to country roads at midday. It uses a quality 16 mm motion picture projector with speed and yaw control.

Auditory module

The purpose of the auditory module is to provide realism and important driving cues and attenuate unwanted sounds. Environmental sounds are those associated with the moving automobile and are important to the driver in estimating speed. They include wind noise, roadway rumble, engine sounds, and general background noise and are generated "on-the-spot" because they must change according to the speed of the automobile. Voice messages, such as highway advisory radio messages, are prerecorded. Unwanted noises must be attenuated because they interfere with the driver's performance.

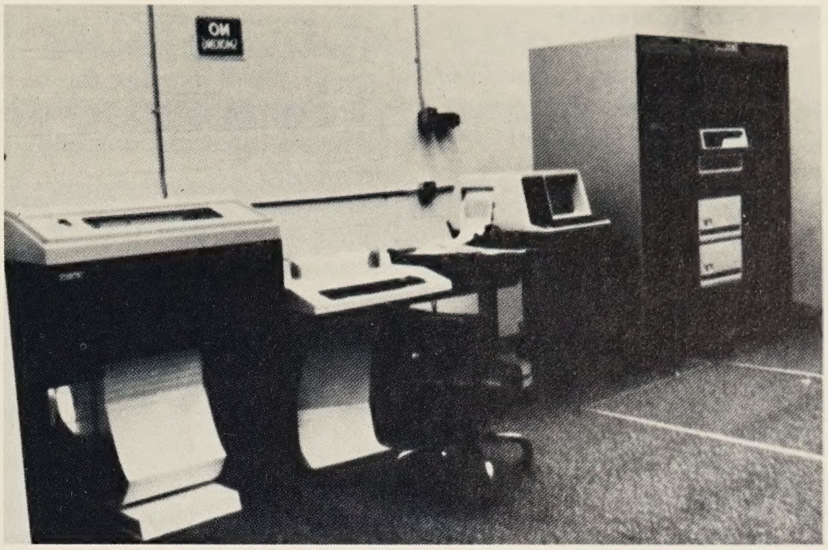


Figure 3.—Scenario computer with peripherals.

Scenario computer module

This module provides the capability for simulation, control, monitoring, and preliminary data recording and analysis (fig. 3). These functions are interfaced with the other modules. It is the "brain" of the HYSIM in that it controls and coordinates all other modules. It provides scenario control, system checkout and calibration, experiment execution, and data collection and recording. It also monitors and checks the functioning of the system during experimental runs and alerts the HYSIM operator of any faults.

Psychophysiological module

This module provides the following measures of the driver's psychophysiological state:

- Respiration (Pneumograph)—rate, depth, and pattern of inspiration/expiration.

- Blood circulation (Electrocardiogram)—heartbeat rate, heartbeat steadiness, and blood pressure (systolic and diastolic).

- Psychogalvanic reflex (PGR)—current and cumulative, expressed as percentage change.

- Muscle action potential—including electromyogram and galvanic skin response.

- Eye fixation potential (from eye marker camera or oculometer).

In addition, this module can encompass additional psychophysiological measures such as brain waves (EEG), eye blinks, pupillary response, vasodilation, salivary secretion, and metabolic rate.

Operator's station module

In this module, the system operator controls and monitors all aspects of the experiment during execution. The system operation is checked and modifications to system function are made from this location before experimental runs. Preliminary data also are analyzed. The module allows monitoring of all important system signals, data flows, and driver actions. The operator can communicate with the driver and observe the driver's actions during the experiment with an intercom and closed-circuit television.

Procurement

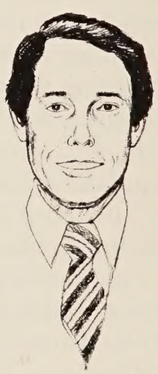
HYSIM procurement requires 2 years. The facility will be fully operational in October 1981. The simulator will be contractor operated for the first year of operation and will reside in a rented facility near the Fairbank Highway Research Station (FHRS) in McLean, Va. When the new research facility is completed at FHRS, the simulator will be moved to the laboratory. Government personnel will manage the facility during the first year and will assume full operational responsibility thereafter.

During the early part of the first year, a study will be conducted to validate the simulator's measures. In the remainder of the year, several representative research studies will be conducted.

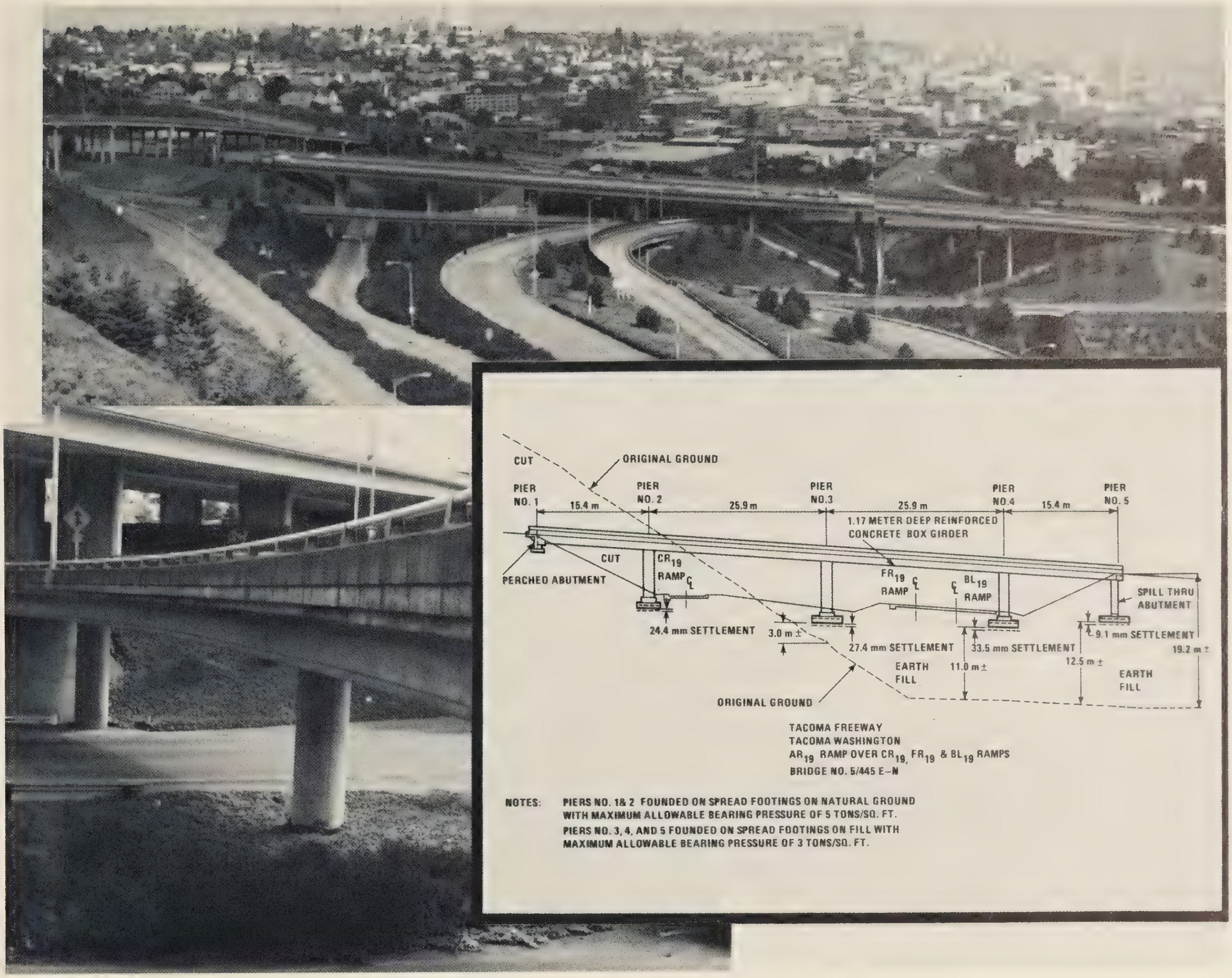
REFERENCE

(*1*) "Tenth NTEC/Industry Conference Proceedings, Nov. 15-17, 1977," Technical Report No. NAVTRAEQUIPCEN IH-294, *Technical Manuals Branch, Naval Training Equipment Center,* Orlando, Fla., 1977.

King M. Roberts is an engineering research psychologist in the Systems Development and Technology Group, Traffic Systems Division, Office of Research, Federal Highway Administration. He joined the staff at the Fairbank Highway Research Station in McLean, Va., in 1963, and since that time, has worked on a range of human engineering problems within the highway community. In addition to managing the activities necessary to acquire and operate the FHWA highway driving simulator, Mr. Roberts is working in research areas concerned with in-vehicular route guidance, advance warning of roadway hazards, and interactions between automobiles and trucks.



Analytical Studies of the Effects of Movements on Steel and Concrete Bridges



by

Hota V. S. GangaRao and Grant T. Halvorsen

An analysis of the effects of support settlement on steel highway bridges and member stresses is presented in this article. Design equations are given to help the designer determine tolerable limits for such movements. The tolerability of support settlements depends on the span length of the superstructure, the number of spans, and the stiffness of the structure.

The structural performance of several concrete systems is also evaluated for differential settlements. A simple but approximate method of analysis (relaxation method) was used in the analysis. The flexural rigidity, span length, creep and shrinkage coefficients of concrete, and rate of the foundation settlement were found to be the most important parameters in accurately determining the settlement stresses.

Introduction

This is the second of three engineering research articles on the development of tolerable movement criteria for highway bridges. The series was introduced in the June 1980 issue of *Public Roads* (vol. 44, No. 1). The significance of defining and setting design limits on the tolerable movement criteria for highway bridges was discussed in the September 1980 issue of *Public Roads* (vol. 44, No. 2). This article deals with the analysis of the performance data for steel and concrete superstructures experiencing support settlements. In addition, the analysis includes time-dependent effects on settlement stresses for prestressed concrete bridge sections. Superstructure types include slab-stringer systems with wide flange stringers, plate girders, trusses, and prestressed concrete American Association of State Highway and Transportation Officials (AASHTO) girders of varying span lengths. Results of this analysis have been used to devise exact and empirical equations to incorporate settlement effects into bridge design and to set tolerable limits of movements.

In this article bridge movements are tolerable if they do not impair architectural appearance, rideability, or structural integrity. Allowable stresses, deflection limits, and human sensitivity limits, as specified in the AASHTO Specification of Highway Bridges, are considered in assessing tolerable movements. (*1*)[1]

Performance analysis of several structure types is based on strength criteria (stress changes leading to the reduction in load carrying capacity) with respect to settlement magnitudes. Serviceability criteria (curtailed usefulness because of cracking or excessive displacements) are used to evaluate the performance under support settlements. Strength and serviceability criteria are considered independently to better understand the effects of movements.

Static Analysis of Steel Bridges

In order to evaluate the effects of movements on the serviceability or strength of a bridge structure, the kinds of movements to be considered must be defined and limits on the magnitudes of these movements must be set. Much research has been conducted by the coal industry and associated research groups to determine the effects on surface structures of vertical movements from underground mining. However, limited data have been accumulated on settlement of bridges. (*2–4*)[2]

[1] Italic numbers in parentheses identify references on page 115.

[2] "Analysis of Bridge Movements and Their Effects," thesis by J. R. Kula for Master of Science in Civil Engineering, West Virginia University, Morgantown, W. Va., 1979.

Methodology

The effect of support settlement for static loading was analyzed with the aid of the ICES STRUDL–II computer package. (*5*) The bridge superstructures were designed according to the AASHTO specifications. The assumed loading conditions for the bridges included both live and dead loads. The live loading consisted of the AASHTO HS20–44 wheel loading or its equivalent lane loading, depending on span length. (*1*) Generally, three loading conditions are investigated: dead load; live load and dead load, with the live load positioned to produce maximum negative moment; and live load and dead load, with the live load positioned to produce maximum positive moment.

The settlements of the bridge supports varied from 0 to 76 mm (3 in) in increments of 13 mm (0.5 in) or 25 mm (1 in), depending on bridge type and span length. For the two-span bridges, two settlement cases were studied: settlement of the exterior support (abutment) and settlement of the center support (pier). For the four-span bridges, three settlement cases were studied: settlement of the exterior support, settlement of the interior support immediately adjacent to the exterior support, and settlement of the center support.

Investigated were continuous two-span and four-span slab/stringer systems consisting of rolled beam spans of up to 18 m (60 ft) in length, rolled beams with cover plates up to 46 m (150 ft) in length, and plate girder spans up to 76 m (250 ft) in length. In addition, two-span continuous parallel chord truss systems, with spans up to 207 m (680 ft), and two-span continuous nonparallel chord truss systems, with spans up to 268 m (880 ft), were investigated. More specifically, the short span bridges used W30, W33, and W36 stringers, with 1.8 and 2.4 m (6 and 8 ft) stringer spacing and spans of 9, 12, 15, and 18 m (30, 40, 50, and 60 ft) (figs. 1 and 2). The intermediate span bridges used W36 stringers with cover plates, 1.8 and 2.4 m (6 and 8 ft) stringer spacing, and spans of 30, 38, and 46 m (100, 125, and 150 ft). The plate girder bridges used a 2.4 m (8 ft) girder spacing and spans of 46, 61, and 76 m (150, 200, and 250 ft). All slab/stringer systems used a 203 mm (8 in) concrete deck, and composite action is assumed between the slab and the stringers. In each bridge, span lengths were equal to reduce the variables considered.

For the two-span parallel chord trusses, span lengths of 146, 183, and 207 m (480, 600, and 680 ft) with panel depths of 15, 18, and 21 m (50, 60, and 70 ft), respectively, were investigated. A constant panel width of 12 m (40 ft) was used, and the chord dimensions were kept constant for all spans to reduce the variables

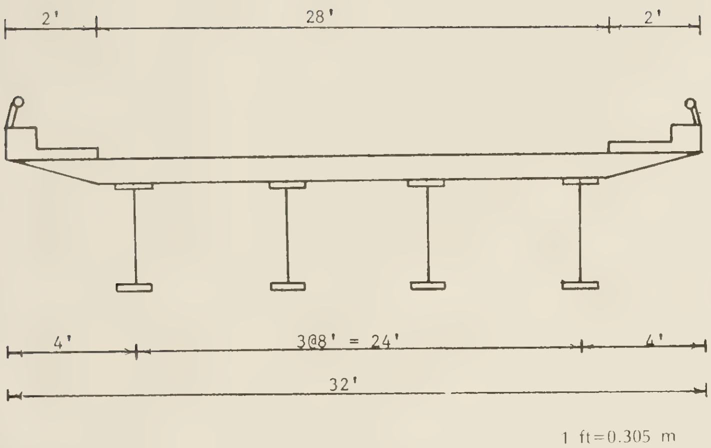


Figure 1.—Bridge cross section with 2.4 m (8 ft) stringer spacing.

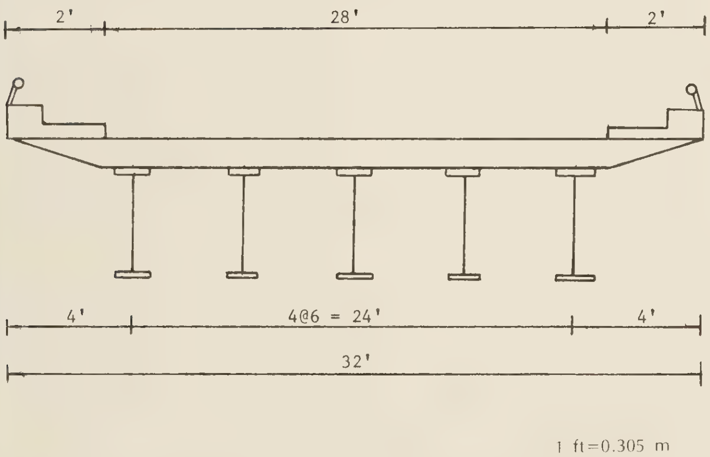


Figure 2.—Bridge cross section with 1.8 m (6 ft) stringer spacing.

considered. For the nonparallel chord trusses, span lengths of 219, 244, and 268 m (720, 800, and 880 ft) were analyzed. Again, the panel width was constant at 12 m (40 ft), but the depth of each truss varied from 24 m (80 ft) at the center support to 12 m (40 ft) at each quarter point. As the span length increased, the moment of inertia of the chords was increased to increase the stiffness of the structure. For both kinds of truss systems, the loads were applied at the panel points, assuming that the floor beams would transfer the lane loadings to the trusses at these points. All trusses were analyzed as frames to account for any "secondary" stresses that might develop.

Discussion of Results

Two- and four-span slab/stringer bridges

The moment diagrams plotted from the computer-aided analysis determined the effect of settlement on internal member stresses. Synthesis of the data revealed that two settlement conditions were critical. For the two-span bridges, the maximum negative stress occurred at the center support, with settlement at an exterior support under loading conditions that would produce maximum negative stress (fig. 3). The maximum positive stress occurred near the quarter point of the structure, with the settlement at the center support under loading conditions that produce maximum positive stress. For the four-span bridges, the maximum negative stress occurred at the center support because of settlement at an interior support under loading conditions that produce maximum negative moment (fig. 4). The maximum positive stress occurred at approximately midspan of the interior span with settlement at the center support under loading conditions that produce maximum positive moment.

In addition to the data on the effects of settlement on internal stresses, the deflected shapes of several of the structures were plotted. The plots indicate that other than the obvious effect of changing the shape of the curve at the support where the settlement occurs, settlements do not alter significantly the live load deflected shape. The slopes of the deflection plots, however, indicate that rotations from settlement may be critical in determining tolerable limits of settlements.

A synthesis of the data on two- and four-span bridges with spans of 9 to 18 m (30 to 60 ft) reveals that the effect of altering the stringer spacing is negligible. Reducing the stringer spacing reduces the load on each stringer and reduces the moment, but the effect of settlement on the moments is relatively the same in both cases investigated.

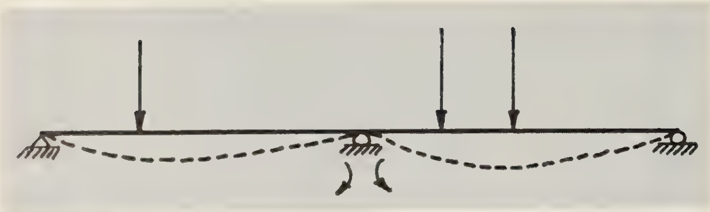


Figure 3.—Position of live load for maximum negative moment.

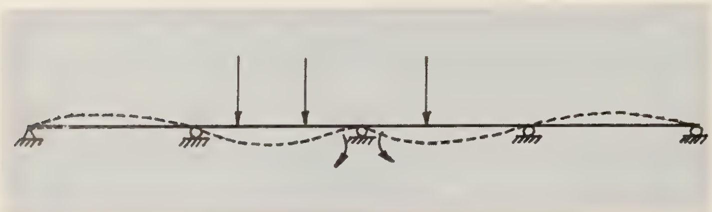


Figure 4.—Position of live load for maximum negative moment.

Support settlements up to 76 mm (3 in) have a profound effect on the internal stresses for span lengths up to 18 m (60 ft). For example, settlements of 76 mm (3 in) at an exterior support for a two-span 9 m (30 ft) bridge increased stress over 500 percent. The effect that support settlement has on a given superstructure depends on both the stiffness of the structure and the span length. There is, of course, an overlapping of effects because the overall stiffness of the structure will be reduced as the span length is increased.

The effects of changing span length versus increase in stress for settlements of 25, 51, and 76 mm (1, 2, and 3 in) on two-span continuous bridges indicate that as span length increases, stresses from settlement decrease. Similar results were obtained for four-span structures, although the increase in stress for a given settlement and span length is greater for four-span structures than for two-span structures. This is because of the higher overall stiffness of the four-span structures.

Figures 5 and 6 show the percentage increase in stress for a given settlement versus the ratio of moment of inertia to span length (I/L) for two-span structures. The plots indicate that for stiffer structures, the increase in stress from settlement is much greater than for flexible structures with long spans.

It is important to note that the percentage increase in stress for the critical positive moment case is significantly larger than for the critical negative moment case. However, it cannot be assumed that the positive moment case is more critical than the negative moment case. Each structure was designed for maximum negative moment and then checked for positive moment. Usually, the maximum positive bending stress was much less than the allowable value. Therefore a larger percentage increase in positive moment is usually less critical than a lesser increase in negative moment, assuming that the structure is designed for maximum negative moment.

As was the case with stress versus span length, the plots of percentage increase in stress versus the (I/L) ratio for four-span structures reveals that for a given (I/L) ratio and settlement, the percentage increase in stress is much greater for the four-span structures than for the two-span structures (figs. 7 and 8 and table 1). This can be explained by the continuity of the structure, which increases its effective stiffness.

The analysis of the research data indicated that no useful information could be obtained by varying the stringer spacing. Therefore, in all subsequent analyses, the stringer spacing was set at 2.4 m (8 ft). Also, the plots of (I/L) seemed to be the most indicative of the behavior of the structure, and subsequent analyses of longer span bridges were aimed at developing similar plots.

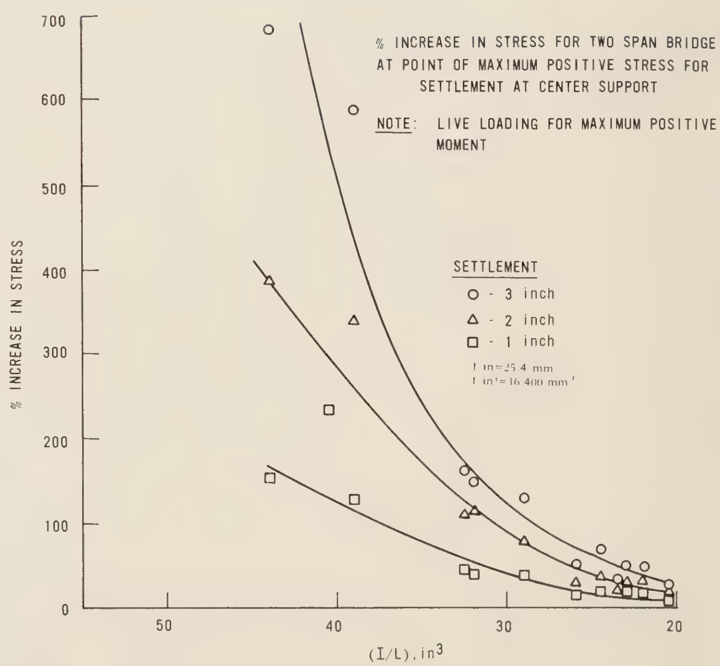


Figure 5.—Percent increase in stress versus (I/L) ratio for two-span bridges.

Two- and four-span slab/stringer bridges with cover plates

Slab/stringer systems with cover plates were designed for two and four equal spans of 30, 37, and 46 m (100, 120, and 150 ft). The cross section of the bridge in each case is the same as previously described (fig. 1) with stringer spacing set at 2.4 m (8 ft). All designs are in accordance with AASHTO specifications, with W33 sections and cover plates where needed to resist applied moments.

The computer-aided analysis of the effects of settlements up to 76 mm (3 in) revealed that the two critical settlement conditions described earlier are also critical for these kinds of bridge systems. The maximum positive and negative moment locations are the same as those identified previously for bridges with 9 to 18 m (30 to 60 ft) spans. The trends previously observed are supported by the data accumulated for spans ranging from 30 to 46 m (100 to 150 ft).

In addition to the data on settlement stresses, data were also compiled on displacements and slopes of structures deflected because of settlement. Slopes of deflected structures (rotations) that are greater than 1/1000 of a radian can cause concrete cracking in the highway bridge decks. (6) This cracking could reduce the riding quality of a bridge and may lead to deterioration of the road surface. Table 2 compares typical rotation values for spans up to 46 m (150 ft). The values given are the most critical values of rotation resulting from 76 mm (3 in) settlement of supports for two-span bridges. Thus, differential settlements of 76 mm (3 in) may not be tolerable for continuous bridges. This may conflict with the field information. (*4*) Additional field results are needed to substantiate the limits on rotations.

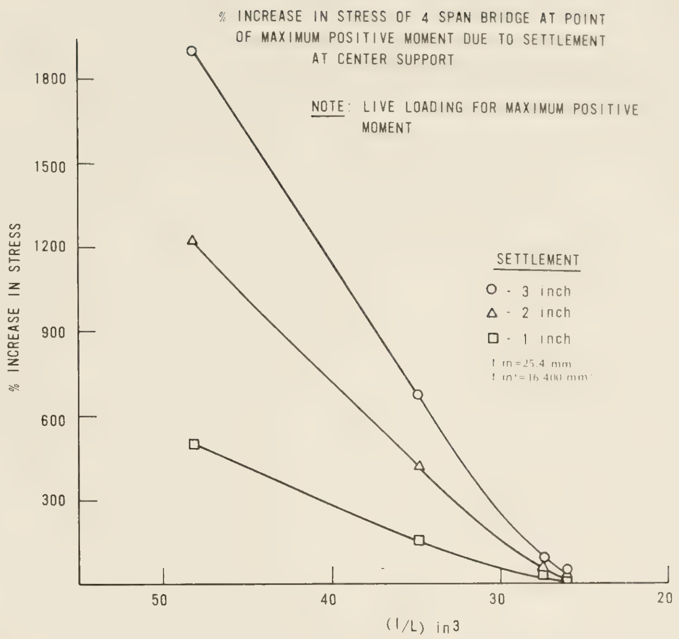


Figure 7.—Percent increase in stress versus (I/L) ratio for four-span bridges.

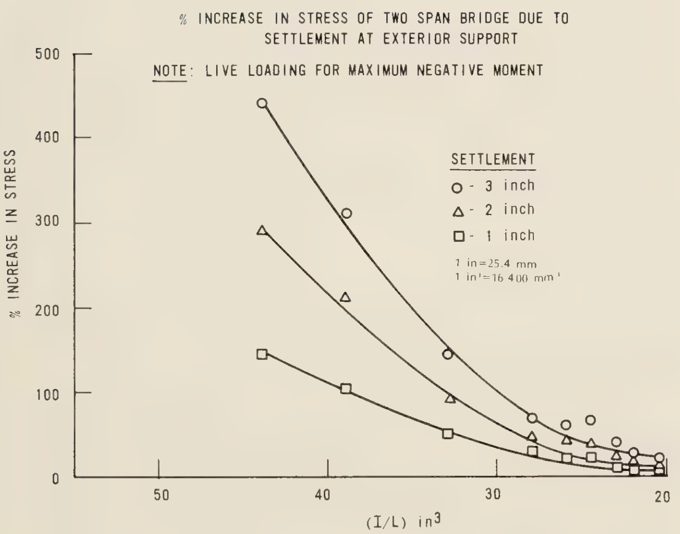


Figure 6.—Percent increase in stress versus (I/L) ratio for two-span bridges.

Two- and four-span plate girder systems

Two- and four-span plate girder systems were designed for spans of 46, 61, and 76 m (150, 200, and 250 ft). In general, the designs incorporated increasing girder depths to provide additional stiffness for resisting the increased stresses from larger span lengths.

Settlement conditions similar to those previously described were applied, and the results of the computer-aided analysis revealed that the same conditions that caused maximum positive and negative stress for shorter span bridges also caused the maximum stresses for spans in the range of 46 to 76 m (150 to 250 ft).

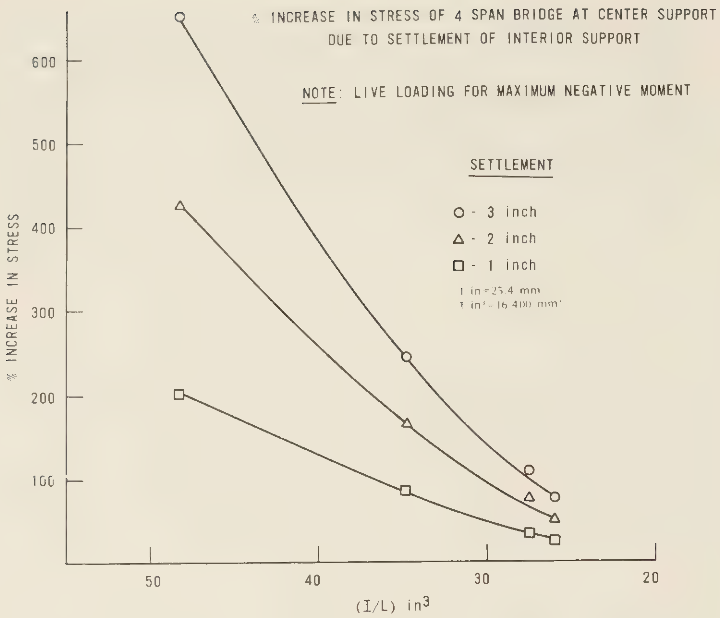


Figure 8.—Percent increase in stress versus (I/L) ratio for four-span bridges.

Table 1.—Typical values of maximum negative stresses at the center support of two-span and four-span continuous steel bridges caused by differential settlements

Span length	Settlement	Maximum calculated stresses Two-span bridges with settlement of exterior support	Four-span bridges with settlement of first interior support
Feet	*Inches*	*ksi*	*ksi*
30	0	14.6	11.0
	1	18.8	21.0
	2	28.2	36.5
	3	38.4	50.5
40	0	—	14.0
	1	—	21.0
	2	—	28.0
	3	—	37.0
50	0	18.0	17.0
	1	22.5	23.2
	2	26.5	29.0
	3	30.0	35.0
60	0	19.0	18.5
	1	21.0	21.3
	2	23.2	24.5
	3	26.0	28.5
100	0	18.8	18.4
	3	21.2	23.0
120	0	18.0	—
	3	20.4	—
150	0	18.9	19.8
	3	21.8	21.5
200	0	20.0	19.0
	3	21.0	21.5
250	0	19.8	20.0
	3	21.2	21.3

1 ft=0.305 m 1 in=25.4 mm 1 ksi=6.9 MPa

Plots of stress versus span length for spans ranging from 9 to 76 m (30 to 250 ft) clearly indicate the effect of settlement on maximum stress (figs. 9 and 10). The plots of negative stress indicate that the stress versus span length curve becomes asymptotic around 138 MPa (20 ksi). All of the bridges described were designed for a maximum negative stress of 138 MPa (20 ksi), and the effect of settlement on these stresses was determined. As expected, the increase in stress from settlement decreased as span length increased.

Similar results were obtained and plotted for positive stresses. The positive stress plots indicate that the curve of stress versus span length goes below the 138 MPa (20 ksi) level for spans greater than 30 m (100 ft). This can be explained by the fact that each bridge was designed for maximum negative moment and checked for maximum positive moment. In the design of plate girder bridges, the depth was determined on the basis of expected negative stresses at the center support. AASHTO specifications limit the minimum depth of girders for a given span, which reduces the stress below the 138 MPa (20 ksi) level in the positive moment regions. Therefore, an increase in stress from settlement does not cause the stresses in the positive moment region to exceed the allowable 138 MPa (20 ksi) range.

Two-span parallel and nonparallel chord truss systems

Each truss was loaded with an equivalent AASHTO lane loading for live load as well as dead load. The loads were applied at the panel points, assuming that floor beams would transfer the lane loadings to the truss at these connections. The trusses were analyzed as a frame to account for any "secondary" stresses. Exterior and center support settlement conditions were considered. Increases in stress resulting from 76 mm (3 in) settlement are insignificant—less than 10 percent increase in the dead and live load stress. This follows previous trends of decreasing settlement stress with decreasing (I/L) ratio.

Design Equations

Although the results of the analysis of various steel bridges reveal much about the influence of support settlements on displacements and forces, the results cannot be obtained easily for design purposes. Design equations for displacements and moments of a continuous bridge system subjected to static loads, derived with the aid of a macro-flexibility approach, are presented below: (*7*)

Equation 1

$$W(x)=A_j\left[\sin\frac{j\pi\xi}{L}+F_{\alpha+1,j}\sin\frac{j\pi\alpha}{n}-F_{\alpha,j}\sin\frac{j\pi(\alpha+1)}{n}\right]\sin\frac{j\pi x}{L}$$

Where,

$$F_{\alpha,j}=\left(\frac{\alpha-\xi/l}{3\cot^2\frac{j\pi}{2n}+1}\right)\left[2\left(\alpha-\frac{\xi}{l}\right)^2-3\operatorname{cosec}^2\frac{j\pi}{2n}\right]$$

$$A_j=\frac{2PL^3}{\pi^4EIj^4}$$

$W(x)$ = Deflection at any point x.
L = Total span of the entire bridge = nl
l = Length between two consecutive supports.
n = Number of spans.
x = Distance between the left abutment and the point where deflection is needed.
ξ = Distance between the left abutment and the applied concentrated load.
α = Support number to the left of the applied concentrated load (fig. 11).
P = Intensity of the load.

$F_{\alpha+1,j}$ can be obtained by substituting $(\alpha+1)$ for α in the above expression for $F_{\alpha,j}$.

The moment expression for continuous bridges can be derived easily by differentiating equation 1 twice with respect to x and multiplying that quantity with the flexural rigidity (EI) of a bridge system, that is, the value of A_j for moment is modified to $(2PL/j^2\pi^2)$. In addition, equation 1 can be integrated with respect to ξ within appropriate limits to find the deflection of a continuous system subjected to uniform loading. It was found that the deflections and moments, computed from the above expressions, yielded a maximum of 5 percent and 10 percent errors, respectively, when compared with the corresponding exact solutions.

The deflection and moment values are obtained by combining the results of an evaluation of equation 1 for j=n−1, n, and n+1. For example, equation 1 is evaluated for j=3, 4, and 5 for a four-span case, and the algebraic addition results in the final deflection value. This is in excellent agreement with the exact ones. It should be noted, however, that this kind of phenomenon is different from the first term approximations that are normally encountered in the classical Fourier series solutions.

Based on the macro-flexibility approach, expressions for forces and displacements from support settlements were derived for continuous bridge systems. (*7*) Additional expressions for maximum stress increase, f_o, produced by the exterior support (abutment) settlements, Δ_o, were developed and are given below:

Equation 2

$$f_o=3Ecn^3\frac{\Delta_o\sin\frac{\pi}{n}}{L^2\left(3\cot^2\frac{\pi}{2n}+1\right)}$$

Where,
E = Young's modulus.
c = Distance from the neutral axis of the cross section to the bottom fiber.
The other terms are defined in equation 1.

The equation for the maximum stress increase, f_α, produced by the settlement, Δ_α, of any interior support (pier) is given as the following:

Equation 3

$$f_\alpha=\frac{11\,Ecn^3\,\Delta\alpha}{L^2}(Z_2+Z_3)$$

Where,
α = Interior support number (fig. 11) at which settlement is taking place; and the values of Z_2 and Z_3 can be obtained from the relationship:

Equation 4

$$Z_i=\frac{\sin^2\frac{i\pi}{2n}\cdot\sin\frac{i\pi\alpha}{n}\cdot\sin\frac{i\pi}{4}}{L^2\left(3\cot^2\frac{i\pi}{2n}+1\right)}$$

By substituting i=2 and i=3, respectively, equations 3 and 4 are valid for values of α corresponding to pier locations at or outside of the point of symmetry of the bridge. For example, for a four-span bridge, equations 3 and 4 would be valid for α = 1 and α = 2. Values for settlement of the third support would, by symmetry, be the same as those for the second support. However,

Table 2.—Critical values of rotation for 3 in settlements

Span length	Rotation	Critical rotation
Feet	*Radians*	*Radians*
30	0.0090	0.001
40	0.0070	0.001
50	0.0067	0.001
60	0.0060	0.001
100	0.0055	0.001
150	0.0076[1]	0.001

1 in=25.4 mm

1 ft=0.305 m

[1] Based on an equivalent lane loading, which produces higher deflections and rotations than truck loading.

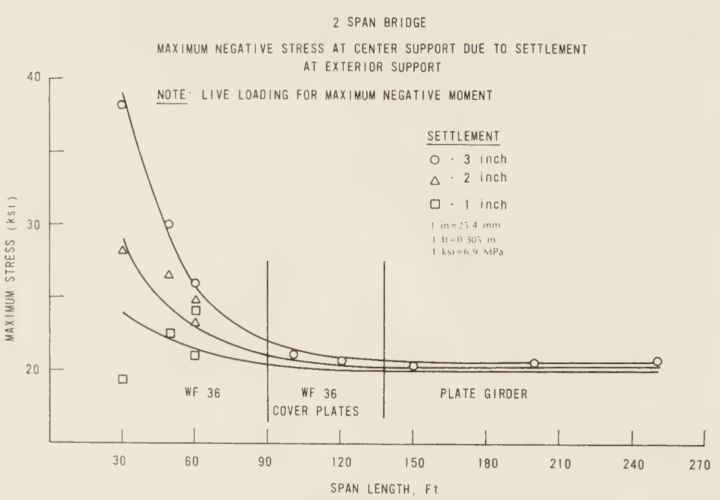


Figure 9.—Maximum negative stress versus span length for two-span bridges.

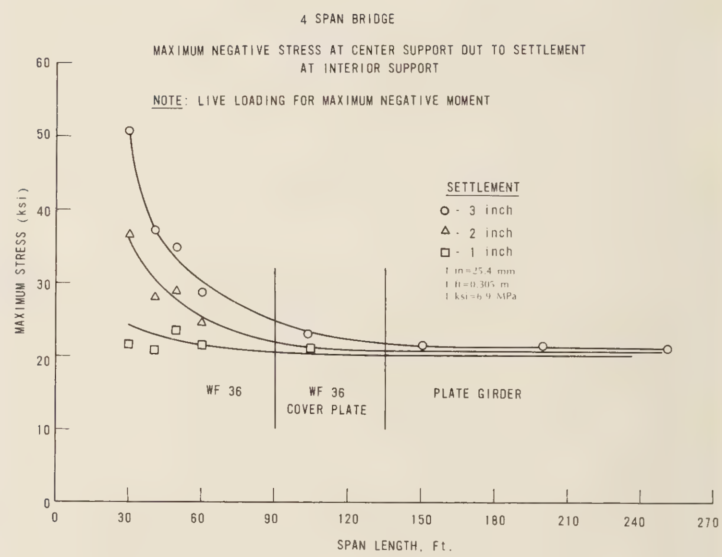


Figure 10.—Maximum negative stress versus span length for four-span bridges.

such symmetry is not readily apparent from these equations. Equations 2 and 3 represent the approximation of Fourier series solutions.

Typical stresses produced by the use of equations 2 and 3 are within 10 percent of those produced by the use of the ICES STRUDL-II programs for corresponding bridges. (5)

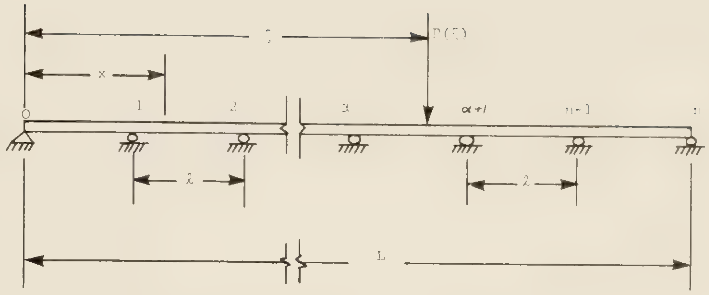


Figure 11.—Continuous bridge of n-equal spans.

Time-Dependent Analysis of Concrete Bridges

In contrast to steel bridges, concrete highway bridges are highly susceptible to effects of material shrinkage and creep. Concrete shrinkage is a deformation that is related to moisture loss within the concrete, as well as other factors independent of loading. Creep is an increase in deformation that occurs while load or stress is kept constant. These aspects of material behavior are related to mix proportions and composition of the concrete, as well as the concrete's curing and subsequent environmental history and the loading history of the structure. As a result of the changing material properties, the properties of the structure change with time. Because the structure changes with time, the changes must be accounted for in the analysis of a structure for support settlement, which is also generally a time-dependent phenomenon.

Gradual foundation settlement causes a continual variation of the stresses in a bridge superstructure. The stresses also will be modified if member properties are changing because of shrinkage or creep of the material. The combined effects of settlement and creep are illustrated schematically in figure 12 for a beam with two equal spans continuous over a central support. The geometry of the structure is shown in figure 12a. The deflection, δ, is the vertical displacement of the middle support, measured from the original position. Two settlement-time relationships are shown in figure 12b. In the first relationship, the settlement occurs suddenly and is maintained at a value of δ_1 thereafter. In the second relationship, the settlement occurs gradually with a decreasing rate. To compare the relationships, the settlement-time function δ_2 is assumed to be asymptotic to the value δ_1 after a very long time.

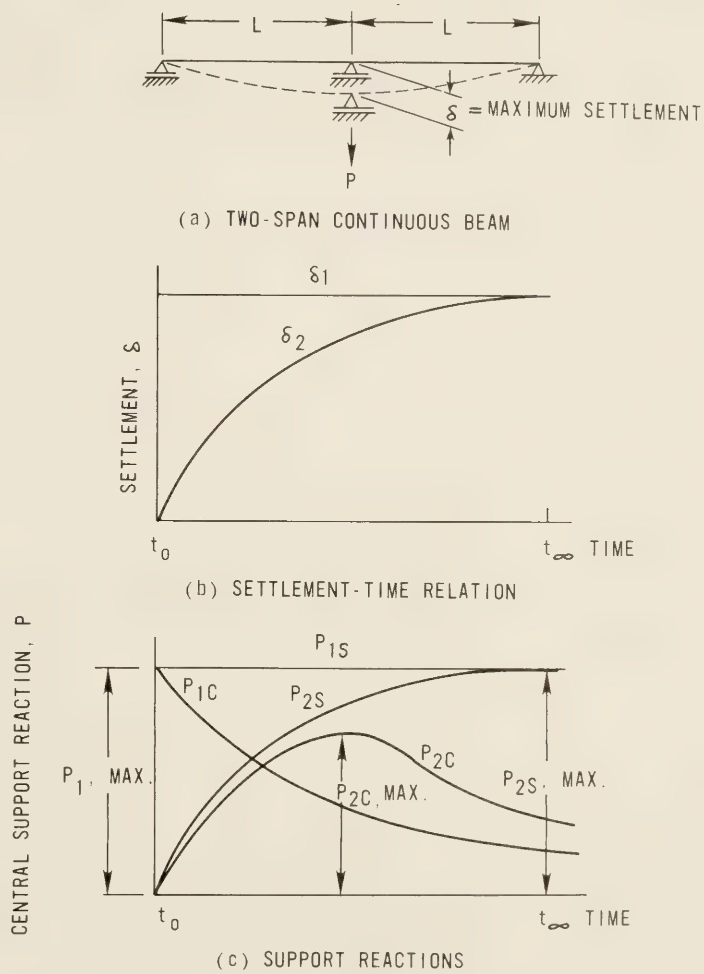


Figure 12.—Creep and settlement effects on support reaction.

If the continuous beam is made of a linear-elastic, "noncreeping" material such as steel, the reaction at the central support will have a sense as shown in figure 12a and a magnitude proportional to the amount of settlement at any time. In figure 12c, P_1S shows the reaction for a noncreeping material if the settlement is instantaneous and equal to δ_1, while curve P_2S shows the reaction for the same material if the settlement occurs gradually. If the beam is made of a material that creeps, such as concrete, the reaction will vary as shown by curve P_1C (instantaneous settlement) or curve P_2C (gradual settlement). Comparison of curves P_1C and P_1S and P_2C and P_2S shows that the effects of settlement will be countered by creep. The reduction in this reaction, and thus the stresses in the beam, can be quite dramatic. For the ideal case of instantaneous settlement, the stresses from settlement may diminish to as little as one-third of the instantaneous value.[3] If the beam is made of a creeping material and is subject to gradual settlement, the center reaction will always be less than if the material did not creep. Further, the magnitude of the maximum reaction and when it occurs will depend on the settlement-time relationship for the support and the creep behavior for the material.

A variety of methods is available for the time-dependent analysis required to study concrete bridges. The choice of the method and level of sophistication should suit the kind of structure, as well as the inherent uncertainties in the settlement, loading history, and material properties. One sophisticated analysis procedure is a general step-by-step method where time is divided into intervals, with stresses and deformations computed at the end of any interval by considering the influence of stresses applied during that interval and all previous intervals. (*8, 9*) Various kinds of construction can be considered, including composite bridges where the girder and deck have different material properties and multistage construction.[4] A computer program has been developed to implement this method. (*8*) A simplified and less sophisticated analysis—the relaxation method—can be useful for analyzing many structures, but becomes rather complex when applied to composite or segmentally erected bridges. (*10, 11*) The relaxation method is the basis for a simplified analysis procedure described later in this article. These analytical methods have been applied specifically to the analysis of reinforced concrete highway bridges for the effects of settlement. For a specific design problem, any of the finite element analysis programs with capabilities for nonlinear, time-dependent material properties may be used.

In any analysis for time-dependent stresses and deformations in concrete structures, some assumptions must be made to estimate the basic material properties. Convenient techniques for estimating creep and shrinkage behavior that reflect a range of structural and material parameters are based on much research and practice. (*12–14*) These estimates typically are presented in a standard set of conditions and a set of adjustment factors to account for other combinations of environmental conditions, member properties, and concrete mix proportions.

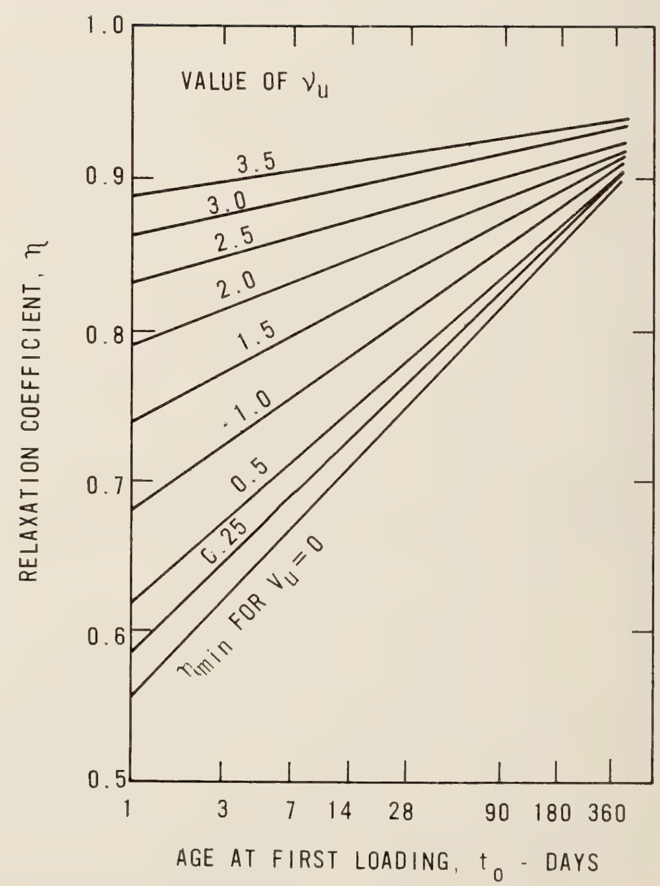


Figure 13.—Aging coefficient.

[3]"Time-Dependent Effects of Support Settlements in Concrete Bridges," thesis by F. Nikjeh for Master of Science in Civil Engineering, West Virginia University, Morgantown, W.Va., 1979.

[4]"Time-Dependent Effects of Support Settlements in Concrete Bridges," by F. Nikjeh.

Once the time-dependent material properties are known, constitutive relationships can be developed that describe instantaneous and time-dependent deformations in terms of applied loads and the effects of foundation settlement. These relationships can be stated in terms of material stresses and strains or member behavior as moments and curvatures or rotations. The relaxation method accounts for the observation that a stress applied over a period of time results in less creep deformation than an instantaneously applied stress of the same total magnitude. This behavior is partly explained by the relationship of creep to age at first loading of the concrete. An aging coefficient, η, may be used to relate the creep strains that result from gradual stress and instantaneous stress.

A relationship of the aging coefficient, η, with age at first loading, t_o, is shown in figure 13 for various values of the creep coefficient, γ_u. (*15*) For a typical range in age of loading, 20 to 30 days, and a creep coefficient in the range of 1.5 to 2.5, η does not vary significantly from a value of 0.85.

By applying the relaxation concepts and the constitutive relationships described previously, the time-dependence of bending moments subject to creep can be established. Specifically: $M(t)=M_i\Omega s$ if the bending moment is applied suddenly, and $M(t)=M_e\Omega g$ if the bending moment is applied gradually—where $M(t)$ is the bending moment at any subsequent time, M_i and M_e are the bending moments as applied instantaneously or gradually, and Ωs and Ωg are index coefficients for sudden effects and gradual effects, respectively.

The index coefficients are shown graphically in figures 14 and 15. These indices are presented in terms of the ultimate creep coefficient and the time after the bending moment is applied to the structure. Thus, these coefficients account for the effects of material properties, member geometry, and time. An additional term, the joint index coefficient, Ωj, also is required for certain kinds of construction. Values of Ωj are shown in figure 16. This term is necessary where a continuous structure is constructed from determinate components, and moments at the joints are actually induced with time. If the load applied to a structure can be broken up into components, such as dead load, live load, settlement, and effective prestress, the effects of each can be evaluated at any time in the life of the structure. To evaluate these effects, it is necessary to determine (or assume) the ultimate creep coefficient, γ_u, and the time since the application of any load effect. In the case of

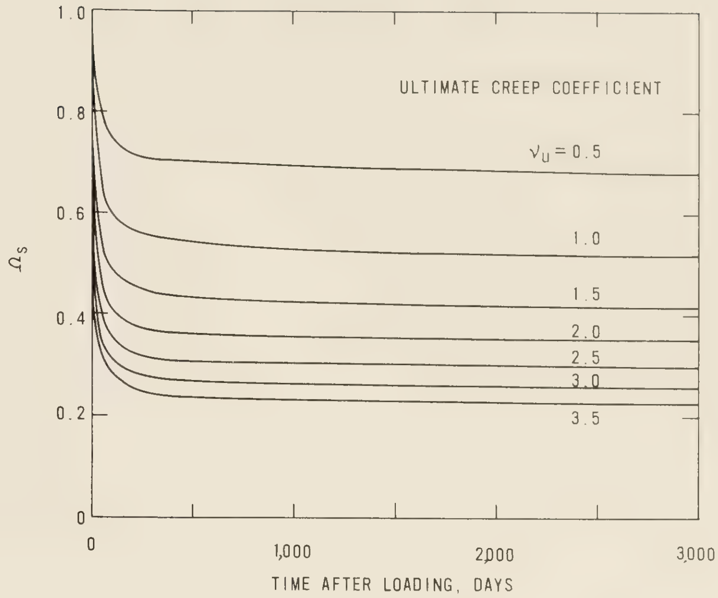


Figure 14.—Relaxation index for sudden effects.

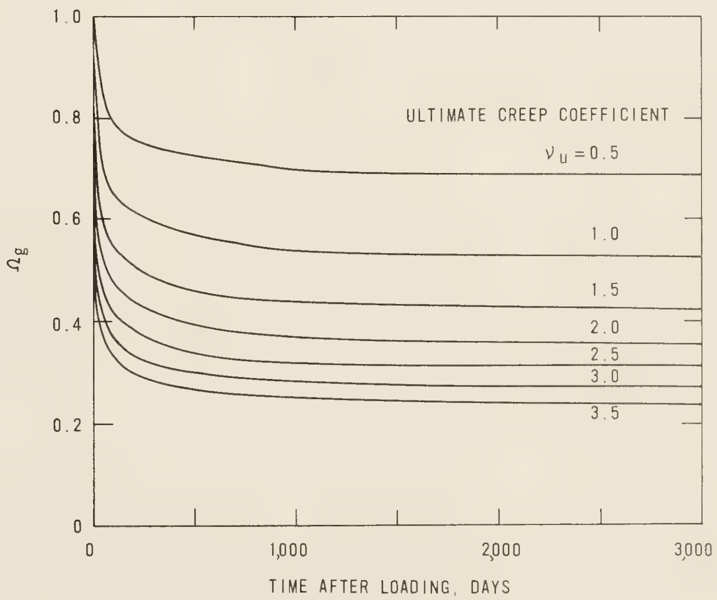


Figure 15.—Relaxation index for gradual effects.

gradual settlement, it is necessary to know when the settlement began. This method is most straightforward for noncomposite structures.[5]

For an ultimate creep coefficient, γ_u, of 1.5 to 2.5, indices Ωs and Ωg are in the range of 0.35 to 0.45 after about 1,000 days. This implies that with time the effects of creep can be expected to reduce the stresses induced by settlement by about 60 percent in many practical situations. In some cases the reduction may be even greater. In general, the stresses from gradual settlements tend to be higher than from sudden settlements because the possibilities for creep relief decrease as the concrete ages.

Other studies have confirmed the intuitive appraisal that settlement stresses are more significant in stiffer structures. Span length is possibly the most important single parameter in evaluating the significance of settlement-related stresses, because the ratio of the stress from settlement to the dead load stress varies as the term I/L^4. For example, a 25 mm (1 in) sudden settlement at the center support of a continuous beam with two 23 m (75 ft) spans would produce an instantaneous stress increase of 40 percent of the dead load stress, while the stress increase would be only 5 percent of the dead load stress in a bridge with 61 m (200 ft) spans.

The analysis of foundation movements for concrete bridges is more complicated than for steel bridges because of the additional consideration of creep and possible effects of prestressing forces. Creep will reduce overall the stresses from settlement. This reduction may be as much as 60 percent for typical concretes. Final stresses will be higher for gradual settlement because there is less possibility for creep relief of these stresses. Span length is the most useful measure of a structure's sensitivity to settlement for continuous bridges, with spans less than 30 m (100 ft) being most sensitive to settlement effects.

Summary

The effects of support settlement on internal stresses for steel and concrete bridges were determined and simple design equations for displacements and stresses were derived for steel bridges from a macro-flexibility analysis. The results of the analysis indicate that the tolerability of support settlement for a given bridge system depends on structural and geometric parameters of the system such as stiffness, magnitude of settlement, and number of spans.

[5]Examples of settlement analyses of concrete highway bridges that use the simplified relaxation method will be given in "Tolerable Movement Criteria for Highway Bridges," Report No. FHWA/RD-80/045, *Federal Highway Administration,* Washington, D.C. Not yet published.

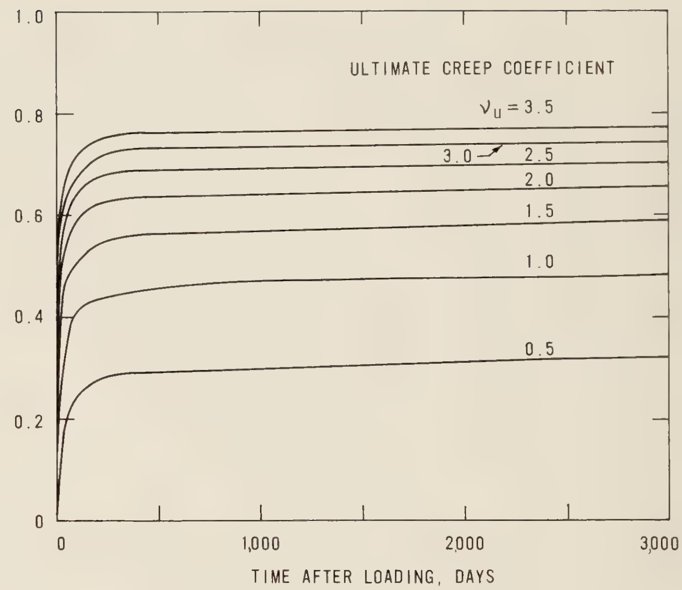


Figure 16.—Relaxation index for field joints.

In particular, the analysis of several steel bridge systems revealed the following:

- For span lengths up to 15 m (50 ft), settlements of 25 mm (1 in) or more are intolerable because of the significant increase in stress.
- For span lengths greater than 61 m (200 ft), the effects of differential support settlements up to 76 mm (3 in) are either tolerable or negligible.
- For span lengths from 15 to 61 m (50 to 200 ft), the tolerability of settlements depends on the stiffness and geometry of the structure. Differential settlements of 76 mm (3 in) may be tolerable if the stiffness to span length ratio is less than 25 and 20, respectively, for two- and four-span bridges.
- In designing continuous span bridges for spans of 15 to 61 m (50 to 200 ft), the costs of using deep foundations to limit settlement should be compared with the costs of using shallow foundations and modifying the superstructure to resist the additional settlement stress.

Time-dependent material properties must be considered in evaluating the effects of foundation settlements on concrete bridges. When settlements are also time-dependent, the magnitude and time of occurrence of maximum stresses must be determined by analysis of the particular structure. This analysis may be performed by using an analytical method suitable to the complexity of the structure and the uncertainties in the settlements and material behavior.

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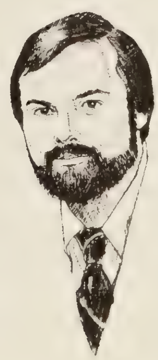
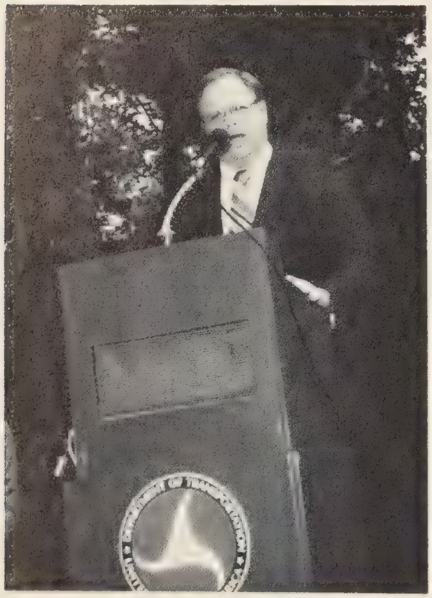


A Dream Comes True

FHWA officials and honorary guests listened to opening remarks by Dr. Love.



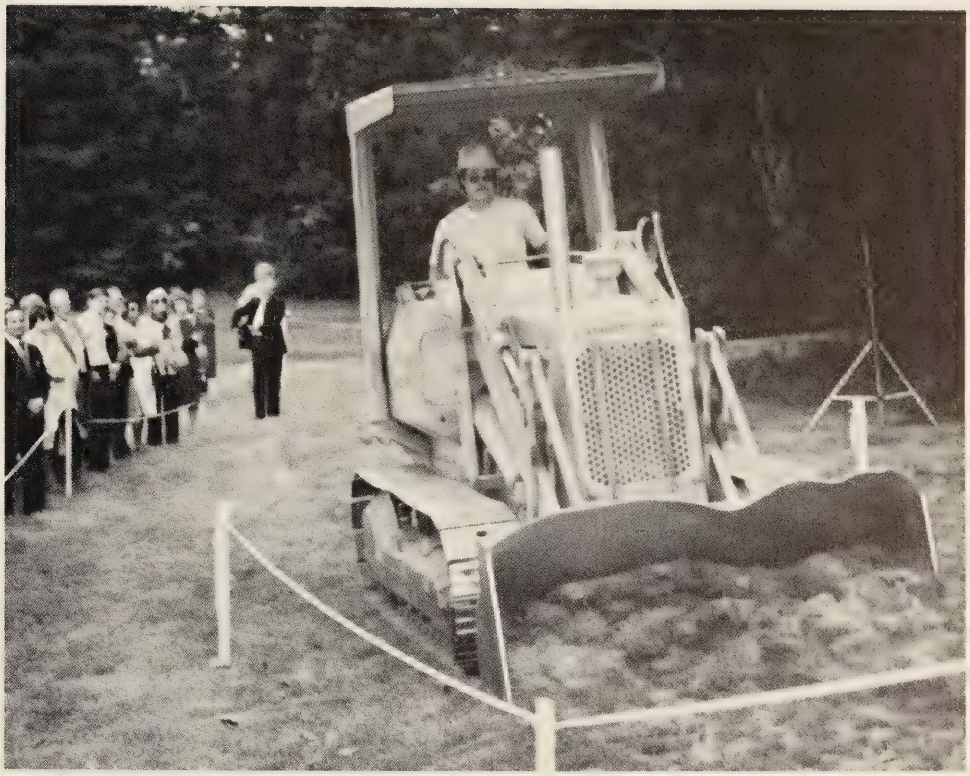
Federal Highway Administrator John S. Hassell, Jr., officiated at the dedication ceremony.

Mr. Hassell, Ms. Burke, and Dr. Love removed the first shovel of dirt.

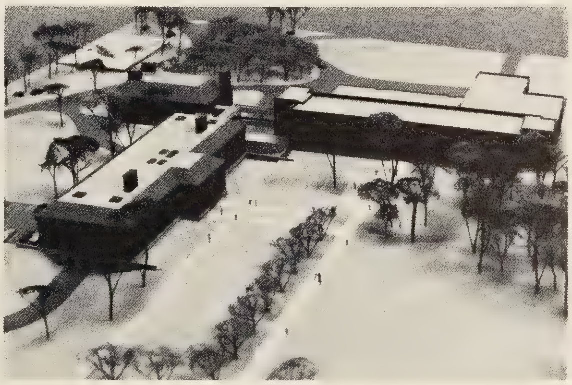
For more than 10 years, an expanded research facility at the Fairbank Highway Research Station (FHRS) in McLean, Va., has been in the planning stages. At times, the actual construction of such a facility seemed to be only a wishful dream. However, on September 17 the dream was finally realized. On that date, FHRS personnel, approximately 200 guests, and several former key R&D officials attended the groundbreaking ceremony for the expanded facility. Following remarks by Dr. Gerald D. Love, Associate Administrator for Research and Development, and Mr. John S. Hassell, Jr., Federal Highway Administrator, the first dirt was shovelled from the site of the future facility by Mr. Hassell, Deputy Administrator Alinda Burke, and Dr. Love. After this ceremonial groundbreaking with a chrome-plated shovel, a front-end loader moved in and shovelled dirt to demonstrate that the groundbreaking was not only ceremonial but the beginning of actual construction.

Occupancy of the new facility is scheduled for fall of 1982. For the first time since the Federal Highway Administration Offices of Research and Development (R&D) were established, all headquarters R&D personnel will be housed in one location. The $6.5 million new building will be located adjacent to the existing FHRS building and will provide approximately 7400 m² (80,000 ft²) of additional laboratory, office, and support service space. New laboratories to be constructed include a driver simulator laboratory, pavement components laboratory, highway communications laboratory, human factors laboratory, major structural testing laboratory, bridge foundation research laboratory, and hydraulics laboratory. These new laboratories will enable FHWA researchers to conduct studies requiring unique facilities. In addition, outdoor testing space and a vehicle preparation area will be available. An accelerated pavement testing facility and wind tunnel research facility will include the latest technological advances.

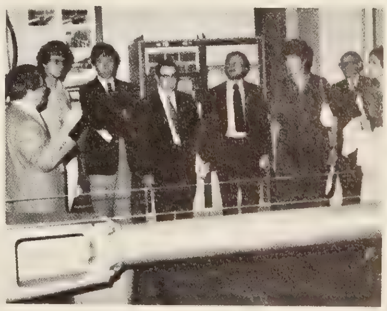
A scale model of the new facility was on view at the groundbreaking. An open house of the existing FHRS facility was held along with the groundbreaking, and the guests enjoyed the many demonstrations and exhibits of R&D's programs and projects.



As the crowd watched, a front-end loader arrived to begin actual construction.

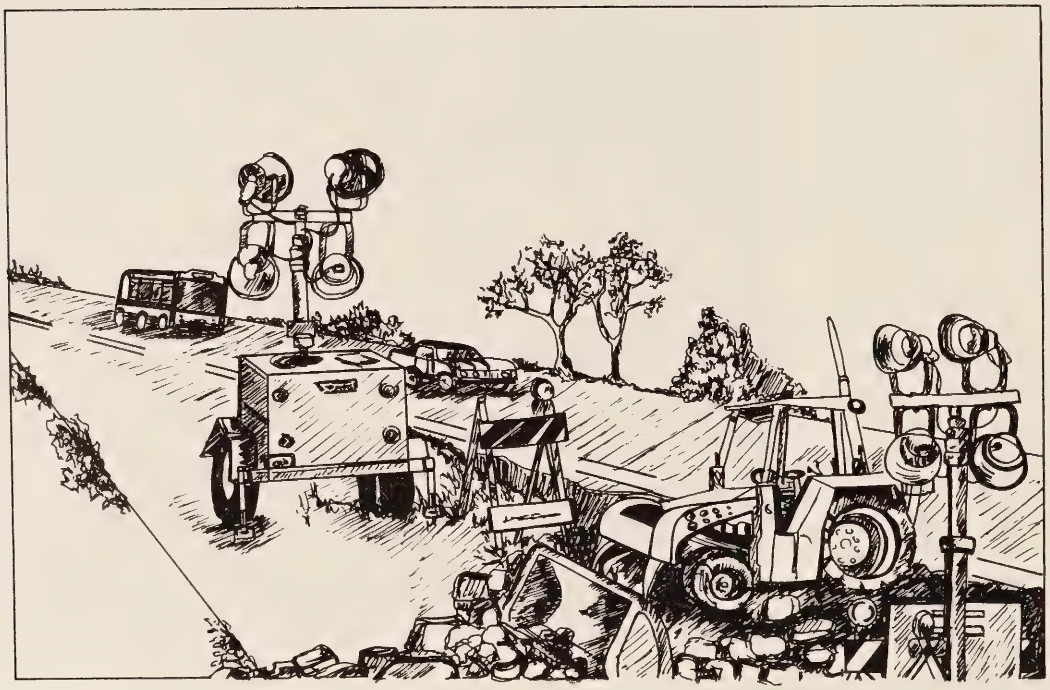


The architectural model of FHRS was displayed during the ceremony. The new facility (right) will be constructed next to the existing annex building (upper left) and perpendicular to the existing main building (lower left).



Mr. Charles F. Scheffey (left), Director of the Office of Research, discussed R&D exhibits during an open house tour of the existing building.

Economic Feasibility of Using Artificial Lighting in Construction Zones



by
Harry S. Lum

Introduction

Night traffic accidents are a serious highway safety problem. Although the number of night accidents is less than the number of day accidents, night accidents are more severe. Nearly 55 percent of all fatal accidents occur at night. (*1*) [1] Slower responses to emergency calls, reduced visibility, fatigue, and intoxication, usually more prevalent at night, contribute to night accident severity. Little is known about how the many factors affecting night driving are related to driver-vehicle performance and traffic flow.

Artificial roadway lighting has been used to reduce night accidents, but the relationship between lighting and reduced accidents is not fully understood. Before-and-after accident studies have not identified the advantages or disadvantages of lighting. Because lighting intensity, visual complexity, and traffic volume often are not considered, statistical relationships are not always valid. The Transportation Research Board recognized this knowledge gap and listed the effect of illumination on traffic operations and safety as a problem deserving high priority for research. (2)

Because traffic engineers generally agree that artificial lighting reduces night accidents, lighting has been suggested as a countermeasure to reduce night traffic accidents in construction zones. It may be prudent to use lights at those construction zones where drivers' expectations conflict with road geometry, such as on sharp curves or at sudden lane drops. Lighting also would be useful at construction zones on narrow bridges in rural areas where there is little or no room on the roadway or roadside for drivers to recover from their errors. These suggested practices adhere to the following principle, involving tort liability, that applies in most States:

[1] Italic numbers in parentheses identify references on page 122.

In the absence of statute, it has been held that there is no general duty of a State or other governmental unit to install or provide highway signs, lights, or markings. However, the duty to provide warnings, lights, or markings may arise where the particular highway presents an [foreseeable] unusual, dangerous condition. (*3*)

This article discusses an investigation of the economic feasibility of lighting an entire construction zone to reduce night traffic accidents. Specifically questioned was if an entire construction zone were artificially illuminated and the number of night accidents during construction were reduced to the same level as that before construction, would the decrease in accidents justify the additional cost of installing artificial lighting? To answer the question, traffic accident data before and during construction were examined and then the number of night accidents that presumably could be reduced by artificial lighting was calculated. A benefit/cost analysis was used to determine the economic feasibility of artificially lighting construction zones to improve motorist and pedestrian safety.

Construction Accident Data Analysis

A 1978 Federal Highway Administration (FHWA) study on accidents and speed in construction zones revealed only three research reports on construction zone accidents. (*4*) All three reported an increase in accident rates in construction zones. Data from seven States throughout the United States were then collected and analyzed to examine the extent of construction-related traffic accidents. These States were selected on the basis of ease and cost of obtaining reliable accident records and diversity of geographical area.

Table 1 shows that in States A and B the total number of accidents actually decreased during construction. However, when data are combined for all seven States, there is a 7.5 percent increase in the total number of accidents in construction zones with a corresponding 9.4 percent increase for night accidents. The percentage of total night accidents to total accidents remained constant at 30 percent for before and during construction. The combined number of fatal and injury accidents increased approximately 5 percent during construction, and the percentage of these accidents to the total number of accidents remained constant at 29 percent before and during construction. Accidents during construction involving fixed objects and rear end, head-on, and turning maneuvers increased greatly while right angle and ran-off-the-road accidents during construction decreased greatly. A breakdown by kind of night accident and severity was not available.

Table 1.—Number of accidents before and during construction in seven States (*4*)

State and time	Before construction	During construction	Change
			Percent
A			
Day	971	892	−8.2
Night	565	592	+4.7
	1,536	1,484	−3.4
B			
Day	NA[1]	733[2]	NA
Night	NA	280[2]	NA
	1,023	1,013	−1.0
C			
Day	1,075	1,216	+13.1*[3]
Night	744	778	+ 4.6
	1,819	1,994	+ 9.6
D			
Day	708	723	+ 2.1
Night	271	353	+30.3*
	979	1,076	+ 9.9
E			
Day	1,540	1,726	+12.1*
Night	659	729	+10.6
	2,199	2,455	+11.6*
F			
Day	340	439	+29.1*
Night	183	196	+ 7.1
	523	635	+21.4*
G			
Day	61	91	+49.2*
Night	32	37	+15.6
	93	128	+37.6*
All States			
Day	4,695	5,087	+8.3*
Night	2,454	2,685	+9.4*
	8,172[4]	8,785[4]	+7.5*

[1] NA = Not available.

[2] Not included in "All States" day and night totals.

[3] * = Statistically significant at the 5 percent level. The statistical test used was the Chi-Square test for 1 degree of freedom under the null hypothesis that there is no difference in the number of accidents before and during the period of construction.

[4] Includes State B totals.

Although table 1 shows a statistically significant increase in the total number of accidents during construction, differences in the number of accidents among the States vary. For example, State C had a significant increase in the number of day accidents but a nonsignificant or chance increase in the number of night accidents. State D, on the other hand, experienced the opposite pattern. Both State A and State B showed a chance reduction in the total number of accidents.

Construction Accident Rates

Table 2, which incorporates traffic volume and mileage data, ranks the States by percent of change in the mean accident rates (which include both day and night accidents) before and during construction. Because traffic volume data during construction were not available, mean accident rates for these periods were computed using before construction volume data. Therefore, the percent of change in the mean accident rates could be higher or lower depending upon whether traffic volumes or mileage increased or decreased during construction. Despite this shortcoming, table 2 compares accident rates. For example, the table shows that the mean accident rate decreased during construction in State A and increased greatly in State F. All of State A's construction projects involved rural interstate routes; most of State F's projects involved urban interstate routes.

Data presented in tables 1 and 2 do not prove that the number of traffic accidents increases during construction. Moreover, examining the construction zone accident rates by individual project shows that for many projects the total number of accidents did not increase significantly during construction. (*4*) In fact, for some projects the total number of accidents decreased during construction. These reductions may have been the result of reduced traffic volumes. In addition, published reports do not indicate whether the traffic control plan and safety devices in construction zones were adequate and properly maintained and used.

In summary, no guidelines establish and no reports document how and when the roadway should be artificially illuminated in construction zones for motorist safety. Most construction is performed during the day, and when performed at night, lighting is used primarily to aid work activities. Because reliable, meaningful data on construction-related night traffic accidents are scarce, many simplifying assumptions had to be made in the economic analyses described below.

Economic Analysis

Tables 3 and 4 summarize the accident and cost data used in the economic analysis. Costs in table 4 include purchase, installation, and removal of light posts, assuming that the installed lighting is temporary. (*5*) The cost data were obtained from a few utility companies and municipalities and therefore are not necessarily representative of cost data throughout the United States. Volume purchasing and geographical differences in pricing could change the costs. This analysis showed that the data were insufficient to analyze nationwide lighting costs.

Tables 3 and 4 show that the cost of lighting a hypothetical 9.6 km (6 mile) road construction project for 230 days is $322,200. Assuming that the lighting equipment could be recycled 10 times, the average cost of lighting a construction project would be $32,220. The corresponding benefit of $1,900 per project (from table 3) from reducing accidents assumed attributable to lighting a construction zone yields a benefit/cost ratio of approximately 0.06. Even if the equipment were recycled 20 times, the benefit/cost ratio would still be less than 1. One shortcoming is the assumption that construction takes place simultaneously on all 9.6 km (6 miles) of the hypothesized project. If work progressed 1.6 to 3.2 km (1 to 2 miles) at a time, the benefit/cost ratio would increase but still would be less than 1.

Fatal accidents in construction zones were not included in the calculation of the benefit/cost ratios because it could not be demonstrated that fatal accidents increased or decreased at construction sites. For the seven States where accident data were collected, the number of fatal accidents decreased slightly in three States, increased slightly in one State, and remained the same in two

Table 2.—Rank of changes in mean accident rates before and during construction in seven States (*4*)

Rank	State	Number of projects	Mean accident rate Before construction	Mean accident rate During construction	Change
					Percent
1	B	9	227.6	206.9	−9.1
2	A	15	120.2	116.7	−2.9
3	D	16	268.2	290.7	+8.4
4	E	10	280.0	309.2	+10.4
5	C	10	189.3	217.5	+14.9
6	G	5	208.8	287.4	+37.6
7	F	10	265.3	428.4	+61.5
Total		75			

Table 3.—Accident data and assumptions for the economic analysis

Item	Information source and assumptions
1. Average duration of a construction project .....230 days	Based on a survey of 79 construction projects in seven States. (*4*)
2. Average project length ..... 9.6 km (6 miles)	Based on a survey of 79 construction projects in seven States. (*4*)
3. Number of night injury accidents before construction 711	Based on 65 projects; data not available on 14 projects. Estimated as 30 percent of total accidents with injury before construction. (*4*)
4. Number of night injury accidents during construction 746	Based on 65 projects; data not available on 14 projects. Estimated as 30 percent of total accidents with injury during construction. (*4*)
5. Increase in number of night injury accidents because of construction 35	Item 3 subtracted from item 4.
6. Number of property damage only (PDO) accidents before construction 1,415	Based on 65 projects; data not available on 14 projects. Estimated as 30 percent of total PDO accidents before construction. (*4*)
7. Number of PDO night accidents during construction 1,568	Based on 65 projects; data not available on 14 projects. Estimated as 30 percent of total PDO accidents during construction. (*4*)
8. Increase in the number of night PDO accidents because of construction 153	Item 6 subtracted from item 7.
9. Average increase in number of night injury accidents per project 0.44	Item 5 divided by 79.
10. Average increase in number of night PDO accidents per project 1.94	Item 8 divided by 79.
11. Cost of increased number of night injury accidents per project $1,401	Injury accident cost of $3,185 from reference 6.
12. Cost of increased number of night PDO accidents per project $485	PDO accident cost of $250 from reference 6.
13. Total cost of number of increased night accidents (injury plus PDO) per project $1,900	Item 11 plus item 12. Value rounded to nearest $100.

Table 4.—Cost data and assumptions for the economic analysis

Item	Information source and assumptions
1. Unit cost of purchasing and installing wooden pole, foundation, and bracket .....$598[1] each	Mounting height of 12.2 m (40 ft); poles located on one side of roadway.
2. Unit cost of luminaire .....$188 each	High pressure sodium, 400 watt.
3. Unit cost of purchasing and installing wiring .....$3/m ($1/lin ft)	Overhead as opposed to underground wiring—costs approximately twice as much.
4. Unit cost of energy and maintenance .....$0.50 per day	Includes energy and minor replacement or repair of lamps.
5. Cost of poles, foundations, brackets, and luminaires $41,658	30.5 m (100 ft) spacing for 1.6 km (1 mile) and one side only, for a total of 53 units.
6. Cost of wiring $5,280	1.6 km (1 mile) of wiring.
7. Cost of energy and maintenance $115	For 230 days. (See item 1 in table 3.)
8. Cost of removing poles $6,625	Estimated at $125 per pole.
9. Total cost of lighting 1.6 km (1 mile) of roadway for 230 days $53,700	Sum of items 5–8. Figure rounded to the nearest $100.

[1] Dollar values are taken from reference 5.

States. One State has no "before" accident data. The fatal accident statistics were aggregated over all construction projects, and there was no way to determine whether the increase or decrease in the number of fatal accidents occurred during the day or at night. Also, there was no way to determine whether traffic control devices used at the construction zones were maintained and used properly.

Summary

This study on the economic feasibility of lighting construction zones for motorist safety has synthesized past highway lighting and accident studies. Accident data used in this study covered the years 1974 to 1975, while cost data covered the years 1975 to 1976. This study modeled a "typical" construction zone assumed to be 9.6 km (6 miles) long with a construction duration of 230 days. These characteristics were representative of 75 reported construction projects for which accident data were available. Unfortunately, the lack of available data relating accidents to the kind of construction performed, kinds of roadways, and traffic and area characteristics limited the analysis. It was concluded that continuous lighting of all construction projects throughout the construction zone is not a cost-effective traffic control safety measure. However, if there are unusual or hazardous conditions in the construction zone, a realistic policy would be to spot light those locations to give adequate warning to motorists.

REFERENCES[2]

(1) "Accident Facts," *National Safety Council*, Chicago, Ill., 1978.

(2) "Operation and Maintenance of Transportation Facilities: Statements of Research Problems," Special Report No. 158, *Transportation Research Board*, Washington, D.C., 1975.

(3) "Liability of State and Local Governments for Negligence Arising Out of the Installation and Maintenance of Warning Signs, Traffic Lights, and Pavement Markings," Research Results Digest No. 110, *National Cooperative Highway Research Program, Transportation Research Board*, Washington, D.C., April 1979.

(4) J. L. Graham, R. J. Paulsen, and J. C. Glennon, "Accident and Speed Studies in Construction Zones," Report No. FHWA-RD-77-80, *Federal Highway Administration*, Washington, D.C., December 1978. PB No. 273290.

(5) M. S. Janoff et al., "Effectiveness of Highway Arterial Lighting," Report No. FHWA-RD-77-37, *Federal Highway Administration* and the *Federal Energy Administration*, Washington, D.C., July 1977. PB No. 273527.

(6) "1975 Societal Cost of Motor Vehicle Accidents," *National Highway Traffic Safety Administration, U.S. Department of Transportation*, Washington, D.C., December 1976.

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

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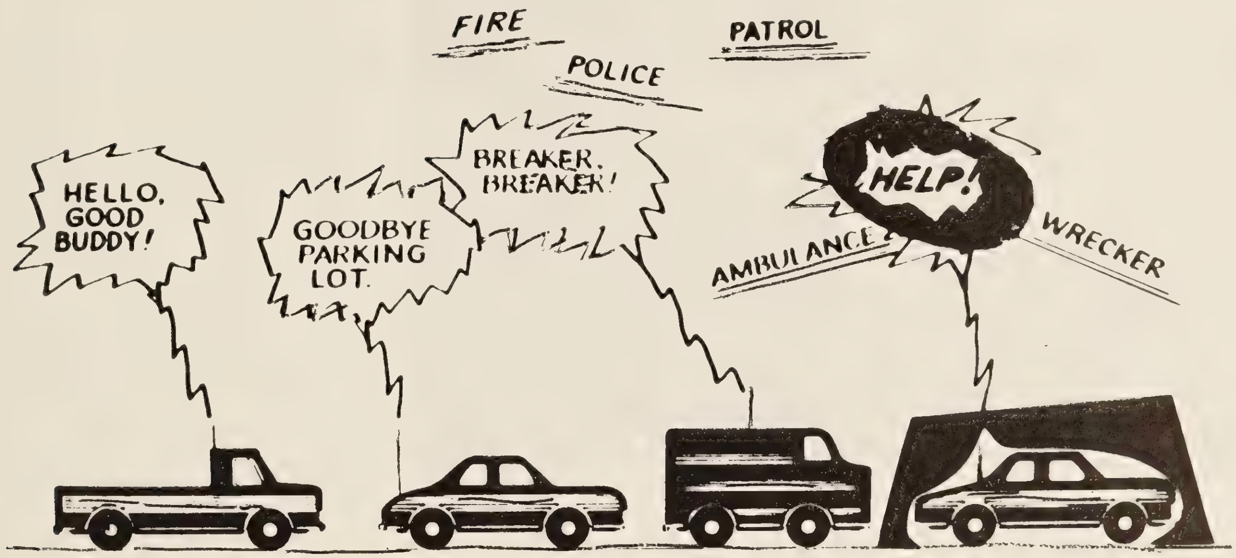
CB Motorist Aid— State of the Art

by
Frank J. Mammano



Introduction

Stranded highway motorists must be detected and assisted quickly to enhance transportation safety. Motorist aid communication systems provide the means to detect stranded motorists and provide the required response mechanism. The demand for improved motorist aid communications has steadily increased since the 1920's. The arrival of the rural freeway has made emergency communications a necessity because of the difficulty in reaching service facilities or commercial telephones. Two motorist aid systems are in general use today—call boxes (which include telephones) and Citizen Band (CB) radios.

The approximately 50 call box systems (fig. 1) in the United States, which allow motorists to call for assistance or report accidents, have been shown to be valuable. However, to use a call box the motorist must leave the vehicle and become a highway pedestrian. Call box systems require patrol cars to respond to the call so officers can determine the assistance needed. Also, these systems are susceptible to vandalism and false alarms. The call box systems have proven to be very expensive on a per call basis, and some systems have been abandoned.

The need for a positive response system that is less expensive and safer has led the Federal Highway Administration (FHWA) of the U.S. Department of Transportation (DOT) to research in-vehicular two-way motorist aid systems. These systems would allow motorists to remain in their automobiles and use a mobile transceiver to access a call box. A 450 MHz transceiver initially was selected for the system but was discarded because it was not cost effective and because of the emergence and phenomenal growth of a 27 MHz CB. The CB, popular for its entertainment value as well as its use in emergencies, prompted the FHWA researchers to investigate the use of the CB as a motorist aid and communication system. Today more than 14 million CB's are licensed by the Federal Communication Commission (FCC). The results of the investigation recommended that the feasibility of an automatic telephone interconnect system using digital signaling and automatic transmitter identification system concepts be studied further. This system will be discussed in more detail later in this article.

Figure 1.—Call box.

CB users are concentrated in the urban areas during peak traffic periods. Recognizing the need for emergency communications, the FCC reserved CB channel 9 for emergency assistance. Several national volunteer citizen groups were organized to provide motorist assistance by monitoring at mobile and fixed stations that interfaced with public safety agencies. The participation by volunteer organizations and public service agencies followed. After carefully evaluating and assessing the opportunities provided by the CB, FHWA/DOT supported the use of CB by public agencies and the development of system improvements through the National Emergency Aid Radio (NEAR) program. This expanded the use of the CB by public agencies.

The NEAR program is sponsored by the National Highway Traffic Safety Administration (NHTSA) and promotes the use of CB radio in emergency situations. Over 30 States actively participate in the NEAR program. Public safety agencies, volunteer groups, motor carriers, professional driver groups, and individual citizens monitor emergency calls. The program is coordinated by public safety, law enforcement, or other government agencies. These programs have been evaluated, and the results clearly establish the value of a two-way, in-vehicular radio and the need for a well-planned system.

Some State agencies and private groups actively are pursuing CB motorist aid programs by the following methods:

- Organizing volunteer groups to monitor more stations to provide an intense local area coverage.
- Using the CB as an input for a wide-area motorist information system.
- Involving State law enforcement agencies.
- Extending the effective transmission range of CB through equipment enhancement.

Table 1 summarizes the characteristics of these methods.

Volunteer Monitoring

Volunteer monitoring of emergency calls is carried out by various groups including CB'ers using their radios on an irregular basis, independent local clubs, and organized local groups affiliated nationally to monitor emergency and public service communications. Some groups specialize in base station monitoring operations, while others rely on mobile operations. One national organization—Radio Emergency Associated Citizens Team (REACT)—has approximately 30,000 members and 1,500 teams in the United States and Canada. There are also local volunteer groups such as Radio Users Send Help (RUSH), established in Georgia to monitor channel 9 and relay motorist needs to the appropriate responding agency.

Control Center Stations

Many volunteer and private groups operate emergency base stations on a regular basis. A favorable location can provide nearly complete coverage of a given area and become the focal point for local emergency and motorist aid communications.

This centrally organized operation can provide excellent coordination and communication with the broadcast media, law enforcement agencies, and emergency services.

The Michigan Emergency Patrol (MEP) in Detroit, and the SHADOW Network, Inc., in Chicago, New York, and Philadelphia are examples of current control center operations. The MEP (fig. 2) is a volunteer organization that operates a base station from a building in central Detroit and communicates with CB mobiles throughout most of the

Detroit metropolitan area. Support is provided by several local broadcast stations, and in return the MEP provides traffic bulletins to the stations.

The SHADOW Network, Inc. (fig. 3), a private organization, uses airplanes, helicopters, radio-equipped mobile units, high powered telescopes, and "SHADOW boxes" (CB receivers at listening posts) to report traffic conditions. The information goes directly to a SHADOW studio. Direct voice reports from the drivers and pilots are available to subscriber radio stations. This information can be provided on a teleprinter, then broadcast at regular intervals on various radio stations, or can be broadcast live in an emergency. The reports are transmitted at times specified by the subscriber stations and, according to the contract, information must be aired within 4 minutes.

The system also uses various remote SHADOW boxes by listening to the CB chatter on channel 19. However, a system is being developed that would make it capable of automatically accepting an emergency call on channel 9, which would automatically ring an alarm at the SHADOW studio so that the proper responding agency could be alerted.

Police Monitoring

Police monitoring of emergency calls varies among the States. In some States all or nearly all patrol cars are equipped with CB radios (full development). In other States most of the equipment has been purchased by the troopers themselves and operating policies range from allowing any legal communication to only receiving communication (partial development). Some States have very limited use of the CB for emergencies and motorist aid.

Police use of the CB is controversial in several States. Some citizens vigorously support police installation of emergency CB monitors, while others want the monitors removed. Some claim that the police are listening to the CB in order to arrest and harass those who use the CB to avoid radar. The problems of a police-monitoring system could be minimized if the public believed that the system was designed to help motorists in emergencies and provide a two-way flow of useful information.

Enhanced CB Equipment

The Michigan Broad Emergency Assistance Radio (BEAR), developed by the State, and the Citizens Band Automatic Interconnect Digital System (CB/Aids), developed by the FHWA, are two examples of enhanced CB systems.

Table 1.—Characteristics of various CB motorist aid applications

CB motorist aid applications	Communications		Coverage		Response time[1]	False alarm rate[1]	Communication coordination[1]	Motorist confidence[1]	Special features
	One-way	Two-way	Time	Area					
Volunteer monitoring		X	Less than 24 hours	Limited	Fair	Average	Limited	Fair	Good samaritan, mobile and or base operation
Control center operations[2]	X	X	24 hours	Broad	Good	Low	Excellent	Good	Radio station broadcasting, CCTV, speed call telephone, UHF-FM
Police monitoring		X	24 hours	Broad	Excellent	Low	Excellent	Excellent	Security, assurance, good samaritan
BEAR	X	X[3]	24 hours	Broad	Good	Low	Excellent	Good	Extended geographical coverage, good samaritan
CB/Aid System	X	X	24 hours	Broad	Excellent	Very low	Excellent	Excellent	Automatic identification, extended geographical coverage, digital signaling, priority messaging, good samaritan

[1] Ratings are relative to each other.
[2] These could include listening post systems.
[3] The distressed motorist may not have two-way communication if a good samaritan places the call.

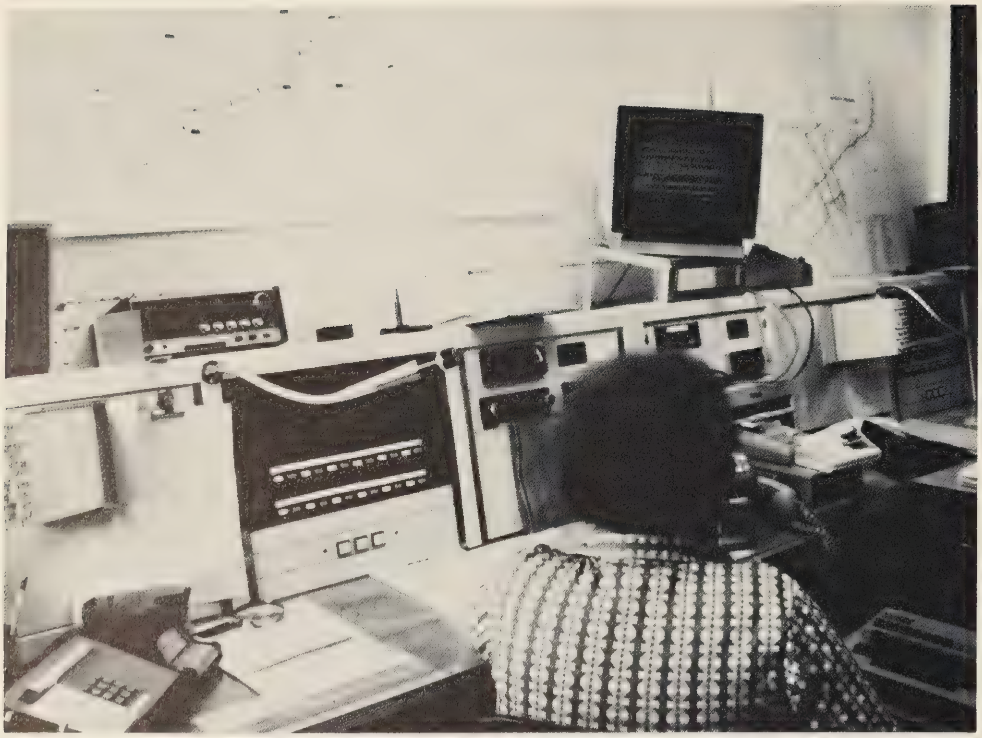


Figure 2.—Michigan Emergency Patrol control center.

Figure 3.—SHADOW Network control center.

The BEAR System

The Michigan BEAR system (fig. 4), which uses the CB radio to assist motorists, consists of 10 remote repeater (relay) stations along a 224 km (139 mile) stretch of I–96 between Detroit and Grand Rapids. The unattended remote stations are linked by dedicated leased telephone lines to the Michigan State Police in East Lansing.

When a motorist equipped with a CB needs help or sees a vehicle that needs help, the motorist can report the problem on channel 9. The message is picked up by a remote station and transmitted by telephone lines to a police dispatcher. Assistance instructions are relayed by police radio, private based telephone line, or the Law Enforcement Information Network teletype from the dispatcher to one of five State police posts. The dispatcher also communicates extensively with volunteer group base stations.

The BEAR system requires that the dispatcher monitor a display board to which all 10 remote stations are connected (fig. 5). Each remote station has a dedicated telephone line that terminates at the police station. The remote stations have a tone squelching technique, allowing the motorist with the strongest signal to access the system. Once the motorist has made contact with the police monitor, the monitor advises the motorist to standby while the monitor turns to channel 19 to listen and call to verify that a motorist needs assistance. Upon confirming the distress call, the monitor returns to channel 9 and talks with the distressed motorist or acknowledges the contribution of the "good samaritan"—the motorist who reported a vehicle needing assistance. All calls received over the system have to be monitored, which could place a burden on the monitor during peak traffic periods.

The CB/Aids System

The CB/Aids system was initiated to determine the technical feasibility of an automatic telephone interconnect device that can be used with a CB radio link for motorist emergencies and assistance.

A pilot program was established in DeKalb County, Ga., approximately 16 km (10 miles) northeast of Atlanta. Volunteer motorists who live, work, or travel frequently in the pilot program area were provided with digital adapters (system access devices) to test the system. About 100 digital adapters were used by the volunteers for the test. Three remote stations were established to provide a coverage area of approximately 185 km^2 (71 mi^2). This area included heavy industrial, commercial, and multiple- and single-family residential areas. The coverage area (fig. 6) included Interstates 85, 285, and 20, U.S. Highways 23, 29, 78, and 155, and numerous State and county roads. The emergency monitoring agency, the DeKalb County police communications center, included a central control unit (CCU) to record time of call receipt, user identification number, remote station identification number, and time of call completion. The police station monitor kept a log for each call and the volunteer motorist also recorded call data. These data were used to determine the quality and response times of the system.

The basic concept (fig. 7) of the system is that motorists can establish communications from their vehicles to a monitoring system through an automatic radio to telephone connection. To make connection, the motorist uses a digital adapter attached to the CB radio (fig. 8). Two pushbuttons on the digital adapter make it possible to select either an "emergency" (E) or "assistance" (A) monitor. By pressing the appropriate button and then keying the microphone, the motorist transmits a short burst of data, including the motorist's identification number and the E or A call identification. Decoding circuitry on a CB base station transceiver at an unattended remote station receives and stores the data. After verifying the data, the circuitry then automatically dials either the emergency monitoring telephone number or the assistance monitoring telephone number. The audible telephone touch tone codes then are transmitted to let the motorist know that the call is being placed. The remote station monitor communicates directly with the motorist using standard radio procedure. After determining the response required and insuring the request is processed, the monitor places the telephone on a hook to

Figure 4.—Michigan BEAR.

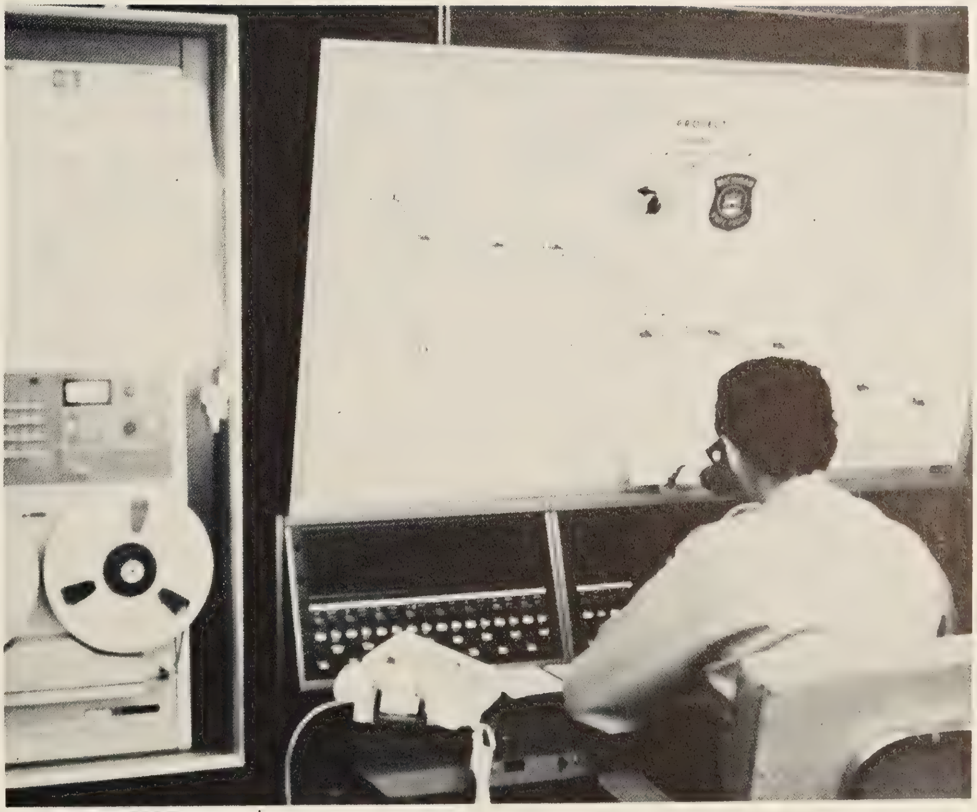


Figure 5.—BEAR control center.

terminate the call. The system automatically resets to receive the next call.

Because the system transmits the motorist's identification numbers, prank calls are minimized. The police monitor can call for the proper assistance without verifying the call. Also, the CCU does not have to be monitored continuously except during peak traffic periods. A motorist without a digital adapter can access the system through a motorist who has an adapter. An extra feature of this system is that assistance calls can be routed to volunteer monitors, thus freeing the police for emergency calls.

Summary

It is apparent there are not sufficient funds to install and maintain motorist aid call box systems on all our highways. The CB radio and other forms of two-way in-vehicular communications are probably the most convenient and safest methods for requesting motorist aid. Although the CB appears to be the most popular system, it is not without problems. All kinds of radio interference and the lack of continuous monitoring of the emergency channel may prevent a disabled motorist from obtaining assistance. In order for the CB to be an effective motorist aid system, fully organized, well-structured systems for emergency medical service and motorist aid are required. Federal, State, and private organizations are attempting to provide these systems; the NEAR program provides the structure for such systems.

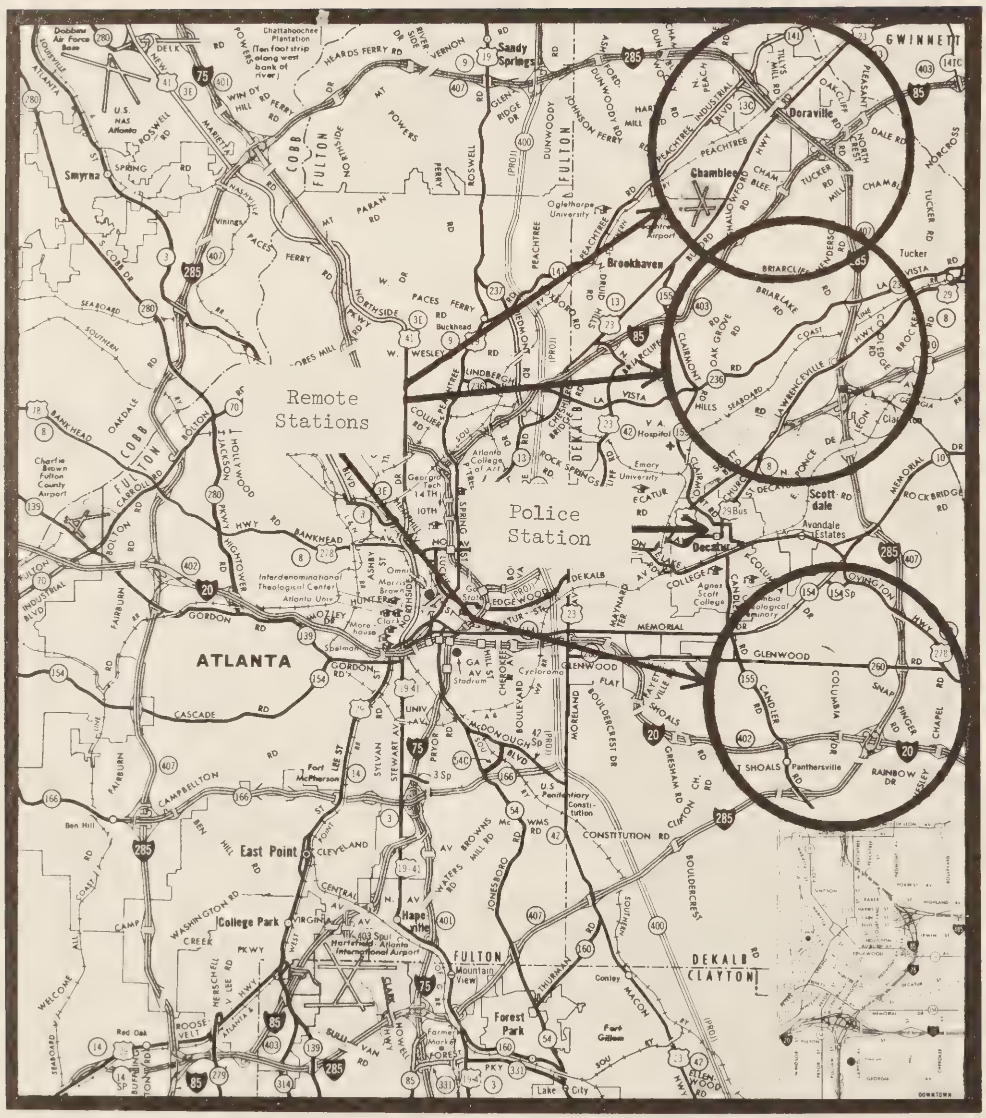


Figure 6.—CB/Aids coverage area.

Michigan is planning to expand its BEAR system to include additional remote stations and larger coverage areas. Georgia plans to install the CB/Aids system at another location and operate it in conjunction with its RUSH motorist aid program.

As discussed above, FHWA/DOT has endorsed the use of the CB as an immediate means for achieving in-vehicle communications and has supported improving automatic identification and automatic signaling. However, although these features have made the CB more valuable to the motorist, the most effective motorist communication system requires fundamental changes in system design that cannot be achieved in the present 27 MHz band. The CB is a desirable interim system while planning a more adequate system in the 900 MHz band. The proposed 900 MHz band for personal use radio would provide many features not available with the CB—selective signaling, paging, FM, automatic channel monitoring, automatic transmitter identification (ATIS), interface with public telephone network, mobile relay/repeater, special use channels, digital communications, and automation.

The FHWA will investigate further other uses of the 900 MHz band for driver information communications,

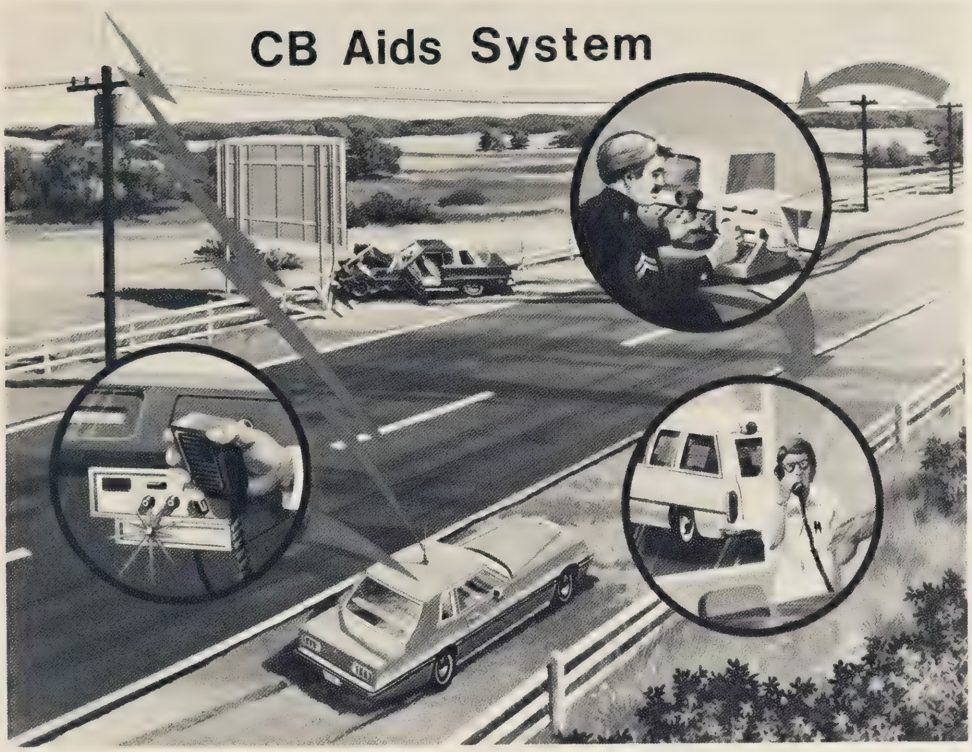


Figure 7.—Basic concept.

including highway advisory radio, vehicle guidance, and motorist aid. Eventually, an integrated driver information system would coordinate and combine the motorist communication requirements into an integrated vehicular electronics package.

Figure 8.—CB digital adapter.

Available Literature

Additional information on motorist aid systems is available in the following publications.

"Motorist Aid Transceiver," Report No. FHWA-RD-76-123, *Federal Highway Administration*, Washington, D.C., 1976.

"The Role of Citizens Band Radio Service and Travelers Information Stations in Civil Preparedness Emergencies," Report No. TM-5752/002/01, *Defense Civil Preparedness Agency*, May 1978.

"Motorist Information Systems and Services," Report No. 600, *Transportation Research Board*, 1976.

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"Citizens Band Communications Manual," *National Highway Traffic Safety Administration*, September 1976.

"State of the Art, Motorist Aid Systems," Implementation Package 76-11, *Federal Highway Administration*, Washington, D.C., August 1976.

"Driver Information and Motorist Aid Hardware," Proceedings of the International Symposium on Traffic Control Systems, Report No. UCB-ITS-P-79-2, *Institute of Transportation Studies, University of California*, August 1979.

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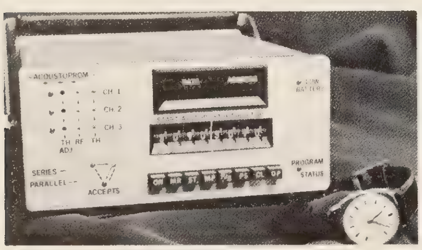
Frank J. Mammano is an electronics engineer in the Traffic Systems Division, Office of Research, Federal Highway Administration. He is project manager for FCP Project 2L, "Detection and Communications for Traffic Systems," and is responsible for programs on the self-powered vehicle detector, the passive bus detector, highway advisory radio, communications for computerized traffic control, CB motorist aid, and in-vehicular route guidance.

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 Division. The reports are available from the address noted at the end of each description.

Acoustic Emission Methods for Flaw Detection in Steel in Highway Bridges, Phase I (Report No. FHWA-RD-78-97) and Phase II (Report No. FHWA-RD-78-98)

by FHWA Structures and Applied Mechanics Division



These reports present an improved method for field structural inspections of highway bridges based on engineering interpretation of records of the high frequency acoustic emissions (AE) that accompany any crack growth in metal structural members.

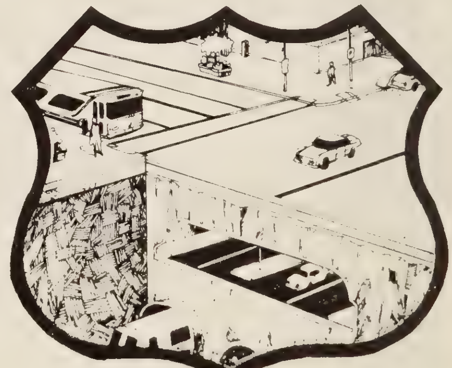
The Phase I report describes a portable, battery operated prototype AE detection and recording system that was evaluated in the laboratory and field. It is a practical concept for continuous, long term unattended monitoring of AE from known active cracks in specific areas on inservice highway bridges. Through various AE source isolation techniques, the area of surveillance on a steel member can be confined to the area of a crack or suspect region. The field data are stored digitally in a plug-in, solid state memory to provide a permanent time based AE record that can be retrieved with a separate readout module. The contents of the memory are not destroyed in the event of power loss. At the end of a test period, the instrument can remain inservice in the field with a fresh memory while the data from the filled memory are analyzed. The solid state memory then can be reused.

An improved field bridge AE monitoring system described in the Phase II report includes amplifiers, source isolation circuits, digital memories, and memory programing controls housed in a small package weighing only 2.3 kg (5 lb). Three low power tuned amplifiers and three AE sensors complete the field measurement system. The field tests have demonstrated that the system functions adequately under adverse weather conditions. The source isolation system is relatively easy to set up and is effective even with very high background noise. The system will perform unattended on battery power for as long as 3 weeks if programed to alternate from operational status to standby during a prescribed portion of each day to conserve power.

The reports are available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock Nos. PB 80 147390 and PB 80 143852).

Cut-and-Cover Tunneling, Executive Summary, Report No. FHWA-RD-79-61

by FHWA Structures and Applied Mechanics Division



This report and others in its series (Vols. 1–3 and supplemental volume) describe analyses made to determine the most efficient and economical cut-and-cover construction technology for building a transportation tunnel in an urban area under various given conditions. Eleven major activities common to cut-and-cover construction were identified and alternate ways of performing these activities were analyzed. The construction costs and surface disruption costs were compared for three cases of cut-and-cover tunnel construction—a 610 m (2,000 ft) long four-lane highway tunnel, a 213 m (700 ft) long underground rail transit station, and a 610 m (2,000 ft) long rail transit tunnel section. Each case was analyzed with other situation variables, including three depths of construction, three ground support systems, two types of bracing, and five underground soil and ground water site conditions common to many U.S. cities. Comparisons also were made between open cut and inverted construction. Forty situation combinations were considered.

The study showed that no single construction method is best for every cut-and-cover tunneling situation, but in some instances, the costs of alternatives are essentially equal. Estimated costs were based on 1974 prices in Washington, D.C., and may be updated to present costs by a procedure provided in the reports.

Construction costs were affected more by site conditions than by any other factor. Once the site has been fixed, judicious selection of the ground support system has the greatest potential for cost reduction because this can influence the requirements for underpinning adjacent buildings, for ground water control, and for the permanent tunnel structure.

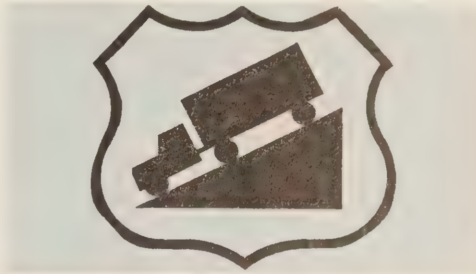
In most cases, there is nearly a straight line relation between total construction cost and depth of construction except that costs generally increase more than linearly with depth when tieback bracing is used. In 34 of the 40 construction situations evaluated, internal bracing was less costly than tieback bracing. Tiebacks are preferred for wider structures because the relatively high initial cost of tieback installations does not vary significantly with increased width of the excavation; the cost of internal bracing increases substantially.

The cost of surface disruption with cut-and-cover construction must be considered in planning. The reports describe how relative evaluations can be made. The inverted cut-and-cover method of construction is less disruptive but generally more costly than the open cut method. In an urban site highly sensitive to surface disruption with minimal utility relocation work the inverted method could be the most effective technique.

The reports are available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock Nos. PB 80 183791 [Report No. FHWA-RD-79-61], PB 257014 [Vol. 1, Report No. FHWA-RD-76-28], PB 80 183809 [Vol. 2, Report No. FHWA-RD-76-29], PB 80 183817 [Vol. 3, Report No. FHWA-RD-76-30], and PB 80 185317 [Supplemental Vol., Report No. FHWA-RD-76-139]).

Feasibility of a Grade Severity Rating System, Report No. FHWA-RD-79-116

by FHWA Environmental Division

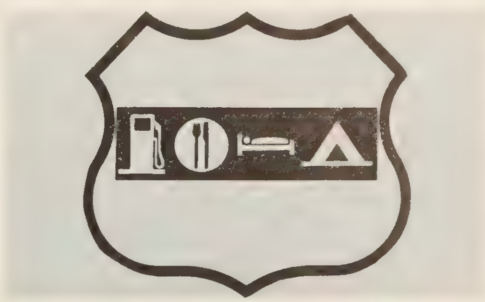


This report describes the development of a system of rating downgrades based on grade severity. The research included a literature review, development of a truck downgrade braking model, a series of field tests to validate the model, and final development of a prototype grade severity rating system. With the rating system, a downgrade may be assigned a numerical rating based on the grade severity. A speed selection model also was developed that can be used to determine a safe descent speed based on vehicle weight and the grade severity rating.

Limited copies of the reports are available from the Environmental Division, HRS-43, Federal Highway Administration, Washington, D.C. 20590.

Motorists' Needs for Service Information on Interstate and Federal-Aid Primary Highways, Report No. FHWA/RD-80/010

by FHWA Traffic Systems Division

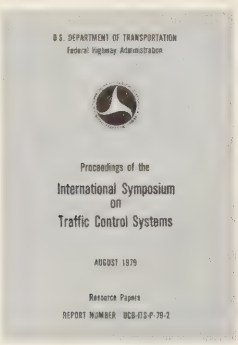


This report contains the results of a comprehensive review of motorists' needs for information on travel-related goods and services. The technical, economical, administrative, and legal problems of information systems are described. A conceptual prototype information system designed to satisfy information needs by using existing information transmission techniques was developed. This prototype system evaluated existing goods and services information systems. The report identifies problems in meeting information needs, recommends the implementation of an information system, and describes additional research needs.

Limited copies of the report are available from the Traffic Systems Division, HRS-31, Federal Highway Administration, Washington, D.C. 20590.

Proceedings of the International Symposium on Traffic Control Systems, Report No. UCB-ITS-P-79-2/4-7

by FHWA Traffic Systems Division and the University of California

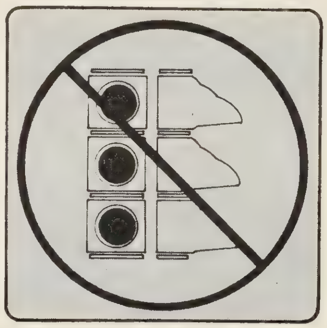


The proceedings of the International Symposium on Traffic Control Systems, held in Berkeley, Calif., in August 1979, are bound in five volumes. Volume 1 contains resource papers, prepared prior to the symposium, on the four themes of the symposium—concepts and strategies, control equipment, driver communication and information, and analysis and evaluation. These four themes are respectively covered in Volumes 2A, 2B, 2C, and 2D. Each volume contains approximately 12 papers. Keynote addresses, user representatives' comments, and discussion summaries also are included.

The proceedings are available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock Nos. PB 80 105091 [Vol. 1], PB 80 180680 [Vol. 2A], PB 80 180698 [Vol. 2B], PB 80 180706 [Vol. 2C], and PB 80 180714 [Vol. 2D]); and the University of California, Institute of Transportation Studies, 108 McLaughlin Hall, Berkeley, Calif. 94720.

Criteria for Removing Traffic Signals, Report No. FHWA/RD-80/104

by FHWA Traffic Systems Division



This report develops a decision process for predicting safety, traffic flow, energy consumption, and cost impacts that can be expected from a traffic signal removal. Criteria are included from experiences involving the removal of traffic signals at over 200 intersections. The report suggests a procedure that should be used to remove a signal from operation.

Limited copies of the report are available from the Traffic Systems Division, HRS-33, Federal Highway Administration, Washington, D.C. 20590.

In Situ Determination of the Chloride Content of Portland Cement Concrete Bridge Decks, Report No. FHWA/RD-80/030

by FHWA Materials Division

A nondestructive measurement procedure has been developed for rapid, local, in situ determination of chloride content in portland cement concrete bridge decks or other reinforced concrete members at the level of the outermost mat of reinforcing steel. A prototype instrument, based on the procedure and built as a self-contained vehicle, has been developed and evaluated in the laboratory and field.

Laboratory chloride determinations on over 100 concrete test specimens show that the method is insensitive to water/cement ratio, moisture content, reinforcing bar location and pattern, aggregate type, and presence of overlays or membranes (unless they contain polyvinyl chloride). The chloride content at a specific depth can be measured, regardless of moderate variations in surface chloride content and with detection limits of 10 to 200 parts per million of chloride, depending on the calibration model. Field trials conducted on five bridges in Texas confirmed that up to six nondestructive chloride measurements per hour can be made with enough sensitivity to permit on-the-spot decisions on bridge deck repair or maintenance.

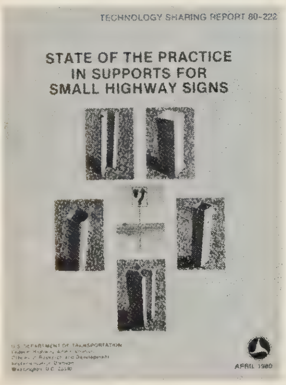
The report is available from the Materials Division, HRS-21, Federal Highway Administration, Washington, D.C. 20590.

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

The following are brief descriptions of selected items that have been recently completed by State and Federal highway units in cooperation with the 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.



State of the Practice in Supports for Small Highway Signs, Report No. FHWA-TS-80-222

by FHWA Implementation Division

This report is part of a study undertaken to survey existing small sign supports, evaluate the crashworthiness of present support systems and proposed systems, and determine the cost effectiveness of systems. This report discusses criteria for selecting a sign support system. The system must have a low material cost, low installation and maintenance cost, be easy to install, and not present a hazard to the motorist. Statistical data on sign use, design data, cost data, and maintenance data are included.

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

Crash Tests of Small Highway Sign Supports, Report No. FHWA/RD-80/502

by FHWA Implementation Division

This report describes 22 full-scale crash tests conducted to evaluate the impact performance of widely used support systems for small roadside signs. Promising new support systems also were evaluated. All systems were of the single post kind, with one exception—one system had a vertical post and a back brace. The report also summarizes the results of recent crash tests on small sign support systems.

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

Crash Tests of Single Post Sign Installations Using Subcompact Automobiles, Report No. FHWA/RD-80/503

by FHWA Implementation Division

Three full-scale crash tests were conducted to evaluate behavior of a subcompact vehicle following impact with widely used signposts. Each test vehicle weighed 880 kg (1,940 lb), and impact speed was approximately 97 km/h (60 mph) in each test. Two tests involved impact with a 4.4 kg/m (3 lb/ft) steel U-post, and the other test involved 4.4 kg/m (3 lb/ft) steel U-posts bolted to form an 8.8 kg/m (6 lb/ft) back-to-back section.

Impact with the 4.4 kg/m (3 lb/ft) post in a dry soil resulted in a vehicle change in velocity above the limiting value. Impact with the same post in a wet soil resulted in an acceptable vehicle velocity change because the post was pulled out of the ground. Impact with the 8.8 kg/m (6 lb/ft) post resulted in a vehicle change in velocity that greatly exceeded the limiting value; after impact the vehicle rolled.

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

Crash Tests of Rural Mailbox Installations, Report No. FHWA/RD-80/504

by FHWA Implementation Division

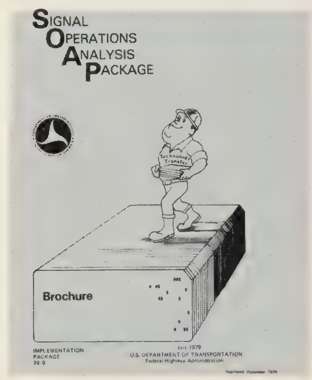
Five full-scale crash tests were conducted to evaluate the impact behavior of rural mailbox installations. Three of the five tests involved commonly used wood post supports, two of which were single box installations and the third was a four-box installation. The other two tests involved two promising new support concepts that use standard steel pipe. Both tests were on single box installations.

Results showed that installations with multiple boxes mounted on boards are a serious hazard to motorists because the board can easily penetrate the windshield. Results also showed that a pipe support post performs more satisfactorily than does a wood post. Careful attention must be given to the box-to-post attachment to prevent separation during impact, and thus minimize the potential for windshield penetration by the mailbox.

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

Signal Operations Analysis Package (SOAP), Implementation Package 79-9

by FHWA Implementation Division



SOAP is a computer program that offers the traffic engineer a practical method for evaluating traffic signal timing. SOAP includes information required for designing signalized intersections, an evaluation of intersection effectiveness, and a detailed analysis of the variation of the measures of effectiveness under changing operating conditions. Typical applications of SOAP include analysis of pretimed versus traffic actuated control and coordinated versus isolated operation; determination of optimal signal phasing patterns and cycle splits and offsets; and identification of optimum cycle lengths and the need for protected left turn intervals.

The report may be purchased for $2.50 from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Stock No. 050-001-00151-9).

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 or Transportation Department; National Cooperative Highway Research Program (NCHRP)—Program Director, National Cooperative Highway Research Program, Transportation Research Board, 2101 Constitution Avenue, NW., Washington, D.C. 20418.

FCP Category 1—Improved Highway Design and Operation for Safety

FCP Project 1V: Roadside Safety Hardware for Nonfreeway Facilities

Title: Cable Guiderail Tension. (FCP No. 41V3284)
Objective: Determine cause for loss of tension in guiderail and establish corrective actions.
Performing Organization: New York State Department of Transportation, Albany, N.Y. 12232
Expected Completion Date: November 1982
Estimated Cost: $71,000 (HP&R)

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

Title: Survey System for Road Roughness, Rutting, and Topography. (FCP No. 31W3014)
Objective: Evaluate performance of system for inventory of road surface topography provided in instrumented test trailer. Evaluate signal processing software and make modifications as needed. Develop and validate a procedure for predicting hydroplaning potential of pavements using topographic data generated.
Performing Organization: Pennsylvania State University, University Park, Pa. 16802
Expected Completion Date: April 1983
Estimated Cost: $395,000 (FHWA Administrative Contract)

FCP Category 2—Reduce Congestion and Improve Energy Efficiency

FCP Project 2J: Practicality of Automated Highway Systems

Title: Systems Studies of Automated Highway Systems. (FCP No. 32J1056)
Objective: Identify and analyze automated highway systems design configurations that would reduce costs and ease implementation. Emphasize reducing guideway costs.
Performing Organization: General Motors Corporation, Warren, Mich. 48090
Expected Completion Date: July 1981
Estimated Cost: $398,000 (FHWA Administrative Contract)

FCP Project 2K: Metropolitan Multimodal Traffic Management

Title: Quality of Traffic Service. (FCP No. 42K1074)
Objective: Investigate the feasibility of calibrating the "two-fluid model" from aerial photographs.
Performing Organization: University of Texas, Austin, Tex. 78712
Funding Agency: Texas State Department of Highways and Public Transportation
Expected Completion Date: August 1981
Estimated Cost: $68,000 (HP&R)

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

FCP Project 3F: Pollution Reduction and Environmental Enhancement

Title: Development of Traffic Information for Estimating Mobile Source Emissions for Air Quality Modeling. (FCP No. 43F3582)
Objective: Develop a procedure for estimating mobile source emissions for air quality modeling. Evaluate sensitivity of mobile source emission models to input data, and relate emissions from mobile sources to urban air quality.

Performing Organization: Texas Transportation Institute, College Station, Tex. 77840
Funding Agency: Texas State Department of Highways and Public Transportation
Expected Completion Date: August 1981
Estimated Cost: $141,000 (HP&R)

FCP Category 4—Improved Materials Utilization and Durability

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

Title: Use of Foamed Asphalt in Bituminous Stabilization of Base and Subbase Materials and Recycled Pavement Layers. (FCP No. 44C4283)
Objective: Develop design procedures for using foamed asphalt as a stabilizing agent in subbase, base, and recycled pavement layers. Examine durability characteristics of foamed asphalt mixtures.
Performing Organization: Purdue University, West Lafayette, Ind. 47907
Funding Agency: Indiana State Highway Commission
Expected Completion Date: June 1983
Estimated Cost: $83,000 (HP&R)

FCP Project 4F: Develop More Significant and Rapid Test Procedures for Quality Assurance

Title: Correlation of Construction Quality Criteria With the Performance of Asphaltic Concrete Pavements. (FCP No. 44F2084)
Objective: Investigate the performance of asphaltic concrete that has comprehensive construction quality data available. Determine the effect of construction quality criteria on performance, and determine if a relationship exists between variability of various quality criteria and the performance of the finished pavement.
Performing Organization: Louisiana Department of Transportation and Development, Baton Rouge, La. 70804
Expected Completion Date: December 1982
Estimated Cost: $68,000 (HP&R)

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

FCP Project 5B: Tunneling Technology for Future Highways

Title: Evaluation of Site Exploration Predictions and the Instrumentation and Monitoring of Soft Ground Tunneling. (FCP No. 35B3132)
Objective: Design experiments and instrumentation plans, laboratory and field tests, and analyze information developed at the Mt. Baker Ridge tunnel site to evaluate methods for site exploration. Determine if new methods are superior to conventional ones, and predict soil-tunnel interaction.
Performing Organization: Shannon and Wilson, Inc., Seattle, Wash. 98103
Expected Completion Date: January 1984
Estimated Cost: $648,000 (FHWA Administrative Contract)

FCP Project 5D: Structural Rehabilitation of Pavement Systems

Title: Developing Repair Methods for Continuously Reinforced Concrete Pavement. (FCP No. 45D2594)
Objective: Identify the predominate forms of continuously reinforced concrete pavement deterioration in Minnesota, and evaluate various repair and rehabilitation concepts.
Performing Organization: Minnesota Department of Transportation, St. Paul, Minn. 55155
Expected Completion Date: April 1984
Estimated Cost: $75,000 (HP&R)

Title: The Effect of Preventive Maintenance on Pavement Serviceability. (FCP No. 45D2644)
Objective: Determine the effects of localized large scale preventive maintenance program on overall serviceability of highway system and on subsequent reconstruction needs. Determine cost effectiveness of various preventive maintenance procedures in restoring serviceability and retarding the decline of serviceability.
Performing Organization: New York State Department of Transportation, Albany, N.Y. 12232
Expected Completion Date: June 1982
Estimated Cost: $83,000 (HP&R)

Title: Pavement Performance Model Development. (FCP No. 35D4222)
Objective: Analyze pavement characteristics and develop performance models for establishing functional criteria for roadways. Complete work in two phases: Collect and analyze information and data and formulate mathematical models; and program and exercise selected models, field verify and test models, and prepare manuals.
Performing Organization: Austin Research Engineers, Austin, Tex. 78746
Expected Completion Date: April 1983
Estimated Cost: $268,000 (FHWA Administrative Contract)

FCP Project 5J: Rigid Pavement Systems Design

Title: Improved Rigid Pavement Joints and Load Transfer Devices. (FCP No. 35J2032)
Objective: Develop improved load transfer devices for rigid pavement joints and optimize their design, construction, and performance through analysis techniques to model pavement joints as they are affected by traffic load and environmental stresses. Study load transfer device configurations to optimize joint performance. Perform

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laboratory tests of recommended load transfer devices.
Performing Organization: Construction Technology Laboratories, Skokie, Ill. 60077
Expected Completion Date: April 1982
Estimated Cost: $221,000 (FHWA Administrative Contract)

FCP Project 5K: New Bridge Design Concepts

Title: The Behavior of the 90 Degrees Bar Bend 4-Bolt Group Anchor Bolt Installations. (FCP No. 45K1092)
Objective: Evaluate the effect of bolt spacing, making a bolt group, and load type. Develop rational design relationships.
Performing Organization: North Carolina State University, Raleigh, N.C. 27650
Funding Agency: North Carolina Department of Transportation
Expected Completion Date: December 1981
Estimated Cost: $66,000 (HP&R)

FCP Project 5L: Safe Life Design for Bridges

Title: Application of Photon Tomography to Inspection of Bridge Weldments. (FCP No. 35L2051)
Objective: Evaluate ability of tomograph to detect weld defects. Develop methods of using tomography for field inspection of welded structures.
Performing Organization: University of Texas, Austin, Tex. 78712
Expected Completion Date: September 1981
Estimated Cost: $98,000 (FHWA Administrative Contract)

FCP Category 0—Other New Studies

Title: Lime Modified Soil to Increase Subgrade Stability. (FCP No. 40M1704)
Objective: Evaluate routine Statewide use of lime modification of subgrade construction.
Performing Organization: Illinois Department of Transportation, Springfield, Ill. 62764
Expected Completion Date: March 1983
Estimated Cost: $80,000 (HP&R)

Title: Use of High Pressure Liquid Chromatography to Determine the Effects of Various Additives and Filler on the Characteristics of Asphalt. (FCP No. 40M3632)
Objective: Ascertain the quantitative effects on the chemical and physical characteristics of asphalt brought about by specific additives and fillers. Determine optimum mixtures for recycling asphalts.
Performing Organization: Montana State University, Bozeman, Mont. 59717
Funding Agency: Montana Department of Highways
Expected Completion Date: March 1982
Estimated Cost: $106,000 (HP&R)

Title: Evaluation of Concretes Using Various Types of Coarse Aggregates. (FCP No. 40M5514)
Objective: Determine various properties of plastic and hardened concrete using limestone and sandstone coarse aggregates. Compare the results with properties of concrete incorporating presently used class A gravel.
Performing Organization: Louisiana Department of Transportation, Baton Rouge, La. 70804
Expected Completion Date: April 1982
Estimated Cost: $66,000 (HP&R)

New Publication

Work Zone Traffic Control—Standards and Guidelines is intended for persons involved in planning, designing, installing, maintaining, and inspecting traffic control for all work activities on or along streets and highways open to public travel.

The handbook consists of two parts. The first part is a reprint of *Part VI, Traffic Controls for Street and Highway Construction and Maintenance Operations,* of the 1978 edition of the *Manual on Uniform Traffic Control Devices.* This part presents principles and prescribes standards for the design, application, installation, and maintenance of various traffic control devices. Requirements for color, size, shape, and location of the devices are included, and need for the devices is discussed.

The second part of the handbook contains supplemental guidelines on operation and application of traffic control devices in work zones. The guidelines explain how to apply the standards to various work zone situations. This part has many illustrations and examples of common conditions and problems encountered in traffic control.

The handbook may be purchased for $5 from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Stock No. 050-001-00175-6). It also may be purchased at a bulk rate of $200 per 100 copies.

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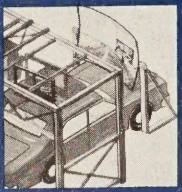

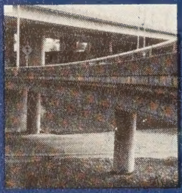

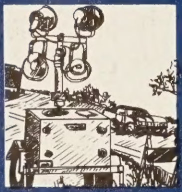

public roads

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