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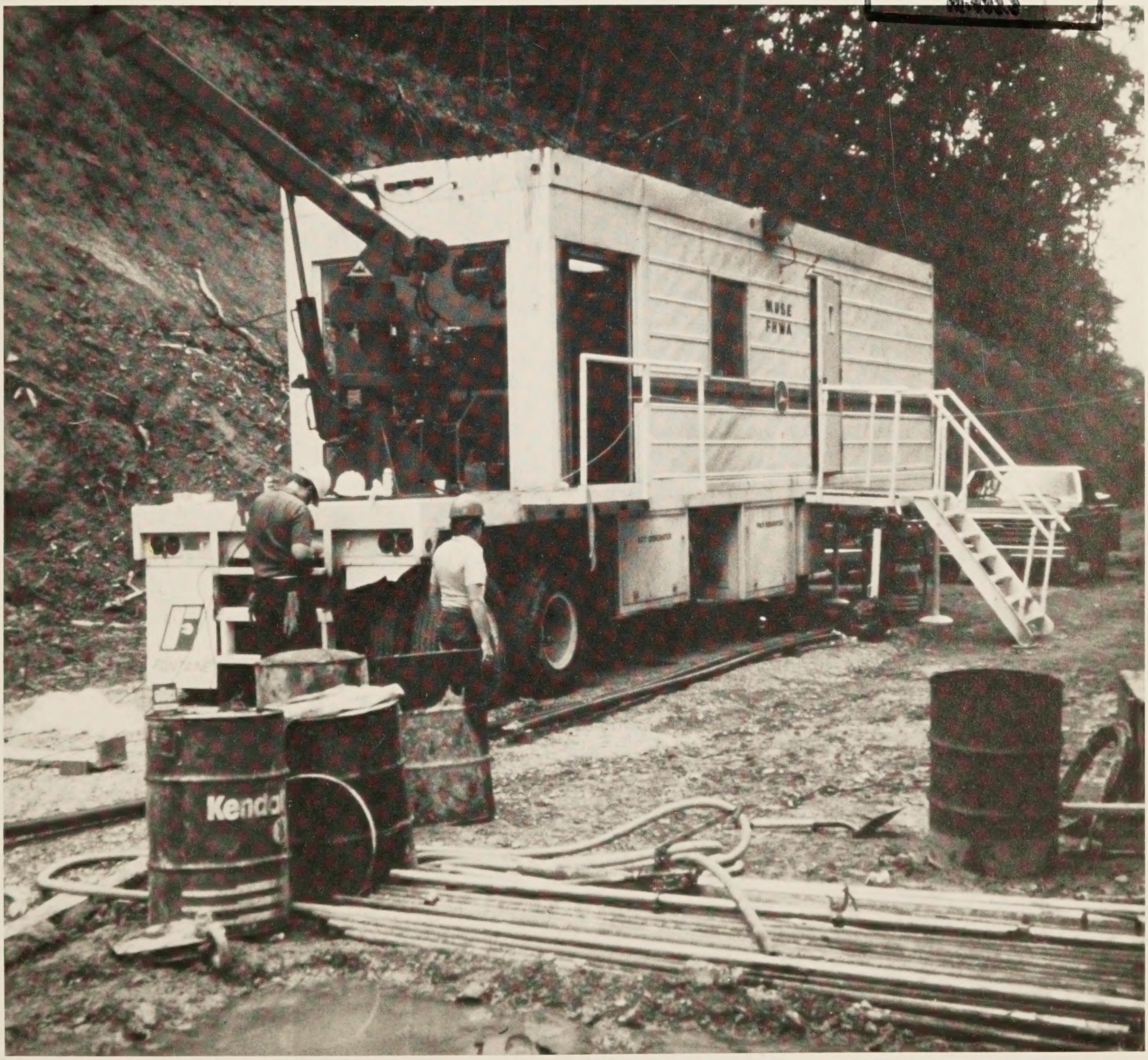
A Journal of Highway Research and Development



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COVER: FHWA's Mobile Unit for Subsurface Exploration (MUSE) in position at the Cumberland Gap Tunnel site in Tennessee.

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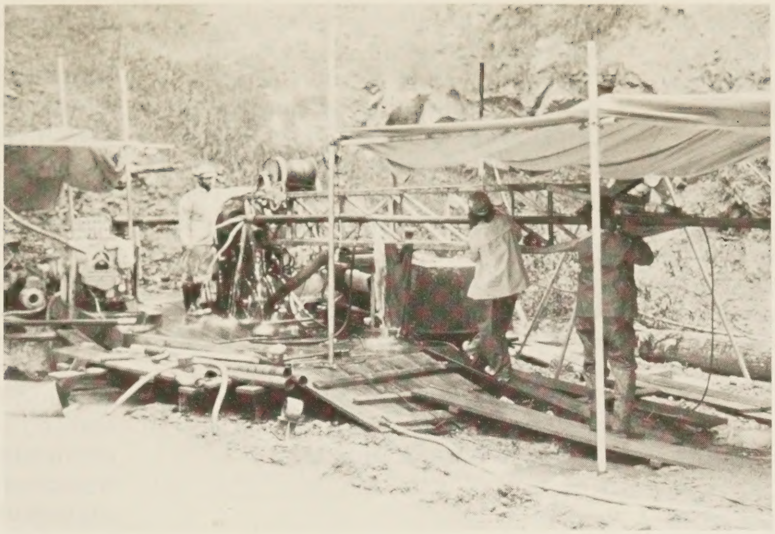

The Combination Probe—A New Instrument for Subsurface Exploration

by
Joseph C. Leifer

This article discusses the Combination Probe—a promising new tool capable of making geophysical measurements through a 3.25 in (82.6 mm) diameter horizontal *or* vertical test borehole. The physical principles used by the probe are described as are its technical features and those of its support systems. Operational experiences to date are discussed as well as the status of the system.

Introduction

As part of its highway transportation research program, the Federal Highway Administration (FHWA) has investigated tunneling technology extensively to reduce the engineering risks, and therefore the costs, associated with highway tunnel construction. Experience has shown that attempts to reduce engineering costs by minimizing investigation of a proposed tunnel site often are counterproductive. Misjudging underground structure may result in selecting the wrong tunnel excavation technique or under- or overdesigning the tunnel supports.

Present methods for obtaining pertinent information for the design and construction of a highway tunnel include assembling geophysical and geotechnical information from other nearby projects, reports from geologists and public agencies, and, sometimes, data obtained from a series of vertical borings along the tunnel alinement. Core samples taken from these borings can be of considerable help in evaluating underground conditions. However, vertical borings may be impractical when much overburden covers the region of interest. An additional drawback to vertical boring is the finite spacing between samples, because different and adverse conditions may be present and remain unsampled.

To insure safe construction and service life of a tunnel, complete and accurate geophysical data should be collected to establish the condition of the soil and rock at and around a tunnel site. Drilling a horizontal borehole through the alinement of the tunnel and passing a geophysical probe through the borehole to provide measurements that can be interpreted and converted into information relevant to the design of a tunnel will achieve this objective. However, the feasibility of this approach depends on the cost involved and the probable value of the information obtained.

Recent experience with directionally controlled horizontal drilling of 3 to 4 in (76 to 102 mm) diameter boreholes with core extractions several thousand feet (metres) deep has proven practical. (*1*)[1] This procedure currently costs approximately $100 to $200 per foot ($328 to $656 per metre), depending on geophysical conditions and depth. This expense is modest compared with the cost of an exploratory pilot tunnel or the cost of the tunnel itself.

Combination Probe System

FHWA initiated a study in February 1975 to evaluate the feasibility of developing a geophysical probe for collecting data within a horizontal borehole. (*1*, *2*) This study concluded that a well-designed probe system could determine underground structure satisfactorily using resistivity, seismic (acoustic), and electromagnetic (underground radar) sensors. Following this study, the Combination Probe System and the MUSE (**M**obile **U**nit for **S**ubsurface **E**xploration) support vehicle were developed, built, and field tested.[2]

The Combination Probe System consists of the following components: Downhole equipment, uphole equipment, logging cable and draw works, and the MUSE support vehicle (fig. 1).

[1]Italic numbers in parentheses identify references on page 11.

[2]S. A. Suhler, "A Combination Resistivity, Seismic, and Electromagnetic Geotechnical Probe System," Southwest Research Institute, San Antonio, Tex., January 1982. Unpublished report prepared for the Federal Highway Administration.

Downhole equipment

The downhole equipment consists of 3 in (76 mm) diameter stainless steel elements that screw together to form four different assemblies that function as either a short baseline resistivity probe, a long baseline resistivity probe, a seismic probe, or an electromagnetic probe. Although certain modules are used in more than one assembly, only a single sensing function can be performed by any one configuration of modules. The downhole control module regulates and distributes several low-voltage power levels to succeeding modules and controls the flow of commands and data between the uphole electronics and the downhole modules.

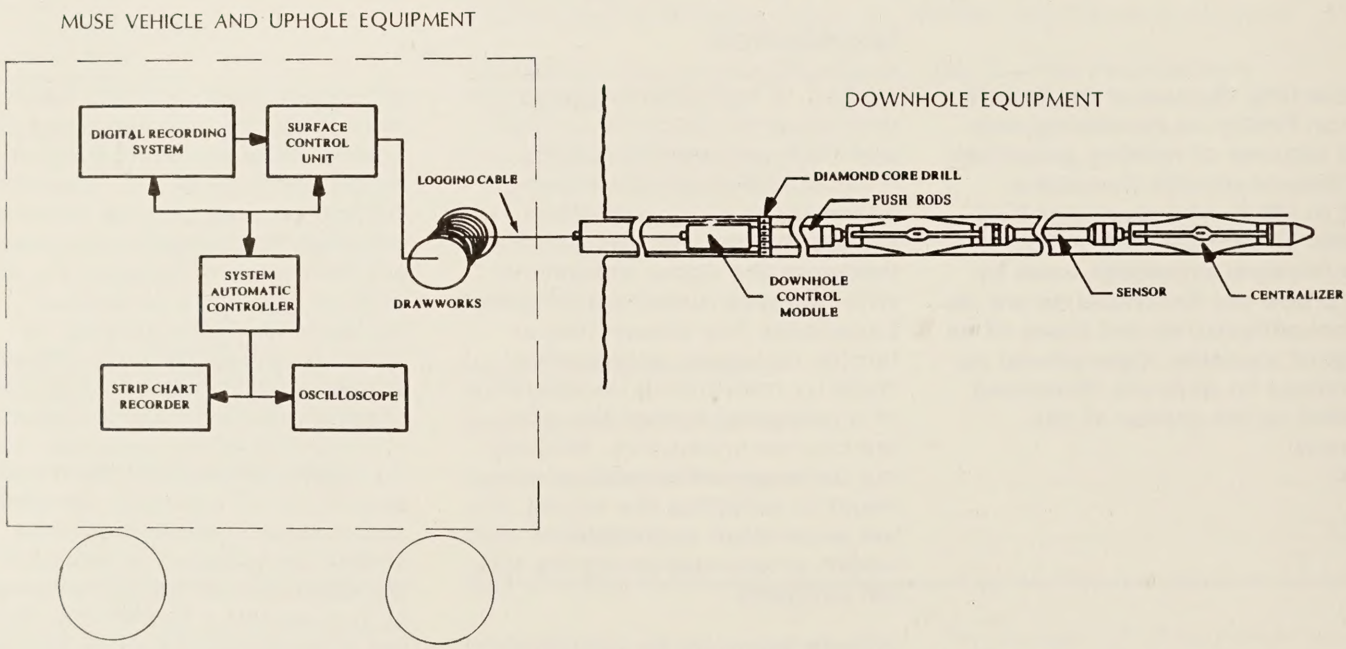

Figure 1.—General system configuration.

Uphole equipment

The uphole electronic equipment prepares and formats commands for the downhole equipment and receives acknowledgment of these commands (fig. 2). Digital data generated by the probes are received, formatted, and recorded on magnetic tape. Control panels allow manual insertion of identification codes and intervention into logging operations when necessary.

Figure 3 shows much of the uphole electronic equipment and the control panel for all three probes (resistivity [long and short], seismic, and electromagnetic). The panel is divided into three sections, one for each probe; switches are available for selecting the desired probe and the associated parameters for operation. Status lights, used with most of the controls, are activated only after the downhole probe has received and acknowledged commands. Real-time downhole data are presented on an oscilloscope that allows monitoring as logging proceeds.

Routine operations are either controlled or prompted by a desktop control computer. At the beginning of each logging run, information identifying the run and establishing technical parameters such as start and stop depth for the electrical odometer, gain settings, and spacing of readings must be manually entered into the desktop computer. At this point, the computer can be commanded to take control of the logging operation, and the data are collected on 0.5 in (13 mm) computer compatible magnetic tape. In future applications, data will be fed directly into a minicomputer system recently installed in the MUSE for concurrent recording and processing.

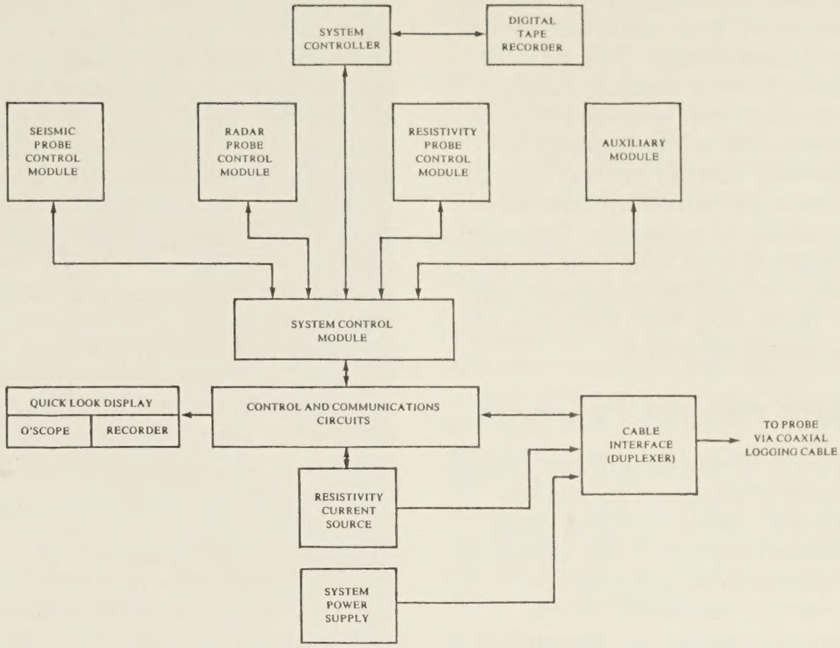

Figure 2.—Uphole equipment block diagram.

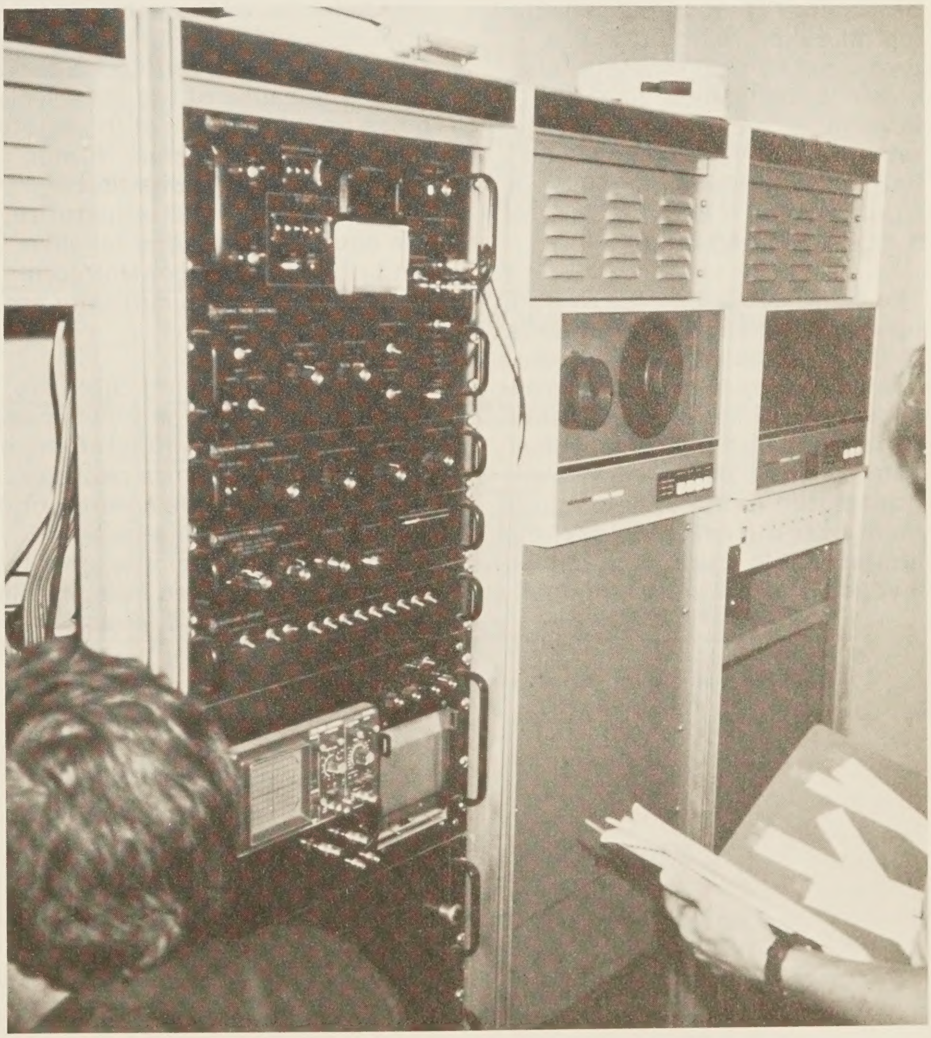

Figure 3.—Uphole electronics in the MUSE.

The onboard minicomputer system provides essentially real-time analysis of data collected by the probe assemblies. This data reduction facility detects subtle equipment malfunctions or misadjustments while logging is underway so that corrections can be made at minimum cost and disruption of schedule. Onboard processing also helps onsite geologists and tunnel designers analyze cores and formulate the underground structure.

Logging cable and draw works

The back of the downhole control module is attached to a steel-wrapped triaxial logging cable that provides 120 V d.c. power to the electronics and electromechanical elements of the downhole assembly. Because it is a critical element in the data transmission system, the logging cable was selected based on a set of required characteristics including diameter, bending radius, gross mechanical strength, and electrical resistance. Specifications include a nominal voltage rating of 600 V, a transmission line characteristic impedance of 62 ohms, and a loss at 1 MHz of 3.4 DB per 1,000 ft (305 m).

The cable is used by the system operator to transmit commands to the downhole probe assemblies to optimize probe performance. Acknowledgment of these commands and the geophysical data collected by the probes are transmitted to the uphole equipment over the cable. Finally, the logging cable, in conjunction with a winch or draw works manually controlled by the system operator, is used to remove the probe from the borehole. A set of slip-rings provides continuous electrical contact to the cable as the winch drum is rotated, and a spring-suspended pulley and potentiometer measure tension during probe withdrawal. The logging cable must be payed out and reeled in carefully to avoid damage to the level-wind and tension measuring apparatus on the winch. The probe is pumped into the borehole by hydraulic pressure exerted behind a rubber ring or "packer" next to the cable attachment on the downhole control module.

The use of digital signals to transmit all commands and data between the uphole electronic system and the downhole probes over the logging cable contributes to the Combination Probe's efficiency. Digital communication was selected because for a properly designed system, signal accuracy does not have to be degraded because of the electrical noise voltages present in remote measurement systems. Numerous integrated circuits that greatly simplify and miniaturize the equipment needed for this kind of command/control/communication system currently are available.

Signals in the logging cable are generated by high and low speed frequency-shift-keyed modems in the uphole electronics package and in the downhole control module. Multiplexers, filters, and duplexers allow a single signal wire to provide three independent simultaneous data circuits.

MUSE support vehicle

All of the previously mentioned equipment (including the probes) either is installed in or transported by the MUSE vehicle (fig. 4). The MUSE (figs. 5 and 6) is a 40 ft (12.2 m) long, 8 ft (2.4 m) wide, and 13.5 ft (4.1 m) high semitrailer that provides an operating platform for the Combination Probe as well as auxiliary functions such as power, lighting, draw works, and a crane to assist in handling the probes in vertical boreholes. The MUSE also has storage space for spare parts, facilities for troubleshooting and repairing electronic assemblies, and air-conditioned space for the system operators and the computer facility. All interior spaces are 7.5 ft (2.3 m) high and equipped with fluorescent lighting.

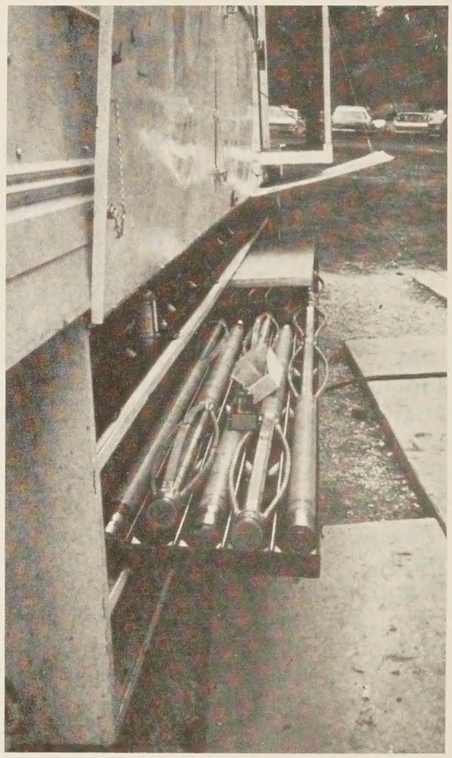

Figure 4.—Modules with fiberglass centralizer springs are stored in a drawer on the MUSE.

Figure 5.—The MUSE is manipulated into position for logging an experimental borehole at the Cumberland Gap Tunnel site.

There are three major compartments in the MUSE. At the front (towing) end is a 12 ft (3.7 m) long × 8 ft (2.4 m) wide room that holds the minicomputer, 50 Megabyte disk, nine-track magnetic tape recorder, printer, plotter, and other equipment.

The middle section, 16 ft (4.9 m) long × 8 ft (2.4 m) wide, is the main working area and contains the electronics for the probe system and storage cabinets. There also is a winch operator's position just behind a large glass window. The winch operator, who also may be the system operator, communicates with personnel at the tunnel portal who are operating the mud pump and/or handling the modules.

At the rear of the MUSE is a 5 ft (1.5 m) long × 8 ft (2.4 m) wide area that can be enclosed by steel rollup doors and is used for the winch and the cable odometers. The winch is operated by a reversible d.c. motor and is fitted with a mechanical brake. The drum will hold about 2,900 ft (884 m) of 5/16 in (7.9 mm) logging cable. At the rear of the

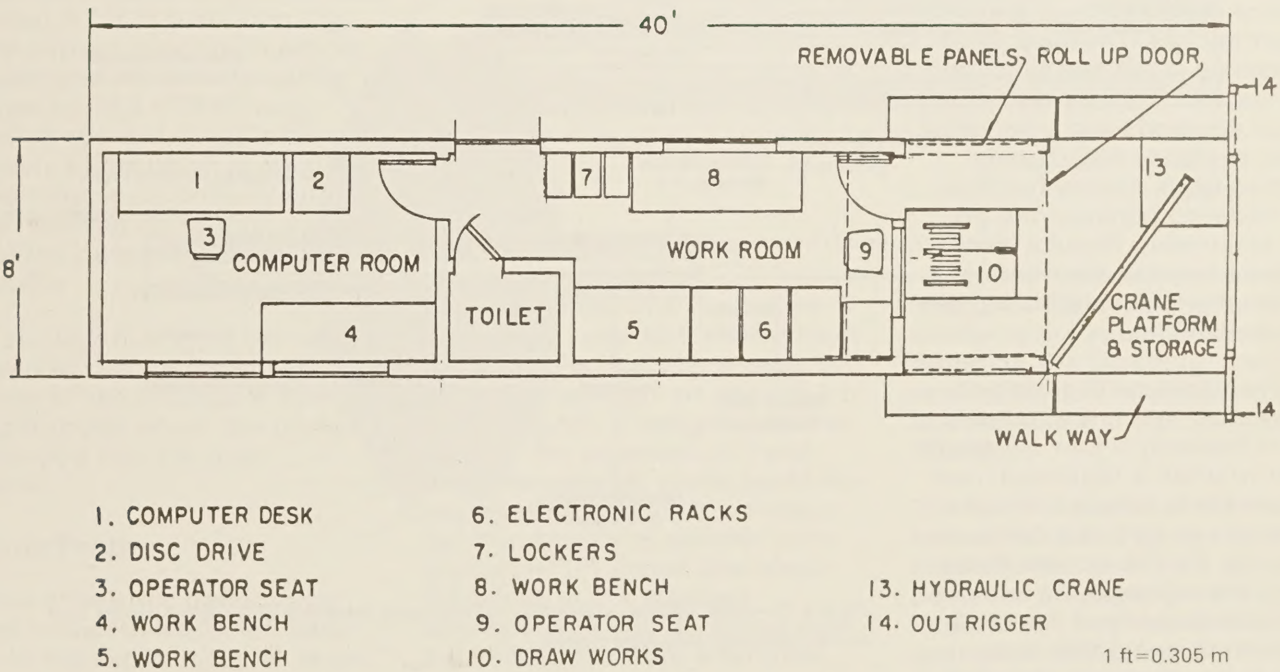

Figure 6.—Equipment layout in the MUSE.

MUSE, an electrically powered hydraulic crane is mounted on an open platform. This crane can handle 3,000 lb (1.4 Mg) loads at an operating radius of up to 14 ft (4.3 m). Maximum capacity at short extensions is 6,000 lb (2.7 Mg). The MUSE is equipped with two 12 kw diesel generators. Power wiring minimizes interaction between heavy cyclic loads, such as air-conditioning and winch and crane, and the power supplied to the probe and the computer. Environmental control is maintained by two heat pumps having a total capacity of 2 tons (7.03 kw) refrigeration. A stowable ladder permits ready access to the main entrance door of the MUSE; for added convenience, there is a demountable steel walkway surrounding the crane area.

Short and Long Baseline Resistivity Probes

Earth resistivity sensing has been used for many years to determine the presence of anomalies in subsurface structures. This geophysical technique is similar to a four-wire ohmmeter, where two wires (a dipole) measure the potential difference produced by a known constant current. The volume of earth sensed is a direct function of the distance between a current injection electrode and a potential sensing electrode pair. Spatial resolution is an inverse function of the distance between the potential electrodes. Results also are affected by the distribution and character of layers having different resistivities.

The pole-dipole earth resistivity method used by the Combination Probe is basically a four-electrode system wherein a regulated, constant current is injected into the volume of soil and rock being explored (fig. 7). The current flows through the soil and rock to a distant electrode (several hundred feet [metres] away) that is assumed to be effectively at infinity. This assumption establishes the premise that in a homogenous earth, current flow would be uniform in all directions. Potential electrodes are separated a fixed minimum distance in accordance with the needed resolving power and signal-to-noise ratio and are moved in increments near the injection electrode. The long baseline resistivity probe automates this motion by using a spatially distributed array of switched electrodes. If the earth were homogenous, the measured potential difference would decrease as the square of distance from the injection electrode. From the electrode geometry, resistivity of the earth can be calculated. Perturbations in the apparent resistivity measured in the borehole can be used to infer anomalous conditions in the soil and rock surrounding the borehole.

The short baseline resistivity probe module (only 10.43 ft [3.18 m] long) is used to determine the resistivity of soil and rock lining a borehole. The spacing between the single current injection electrode and the two potential electrodes is 3.05 ft and 3.54 ft (0.93 m and 1.08 m), respectively. Electrodes for current injection and potential readings are mounted on fiberglass springs specially designed to accommodate the size and weight of the module. At the center of each spring are small metal rollers wrapped with compliant conducting rubber, which insures an area (rather than a point) contact in hard rock. Without an area contact, current density in a dry borehole might be high enough to cause excessive heating of the rock, which would result in unstable and inaccurate readings. Resistivity can be measured in either wet or dry boreholes; the readings can be compensated to insure accuracy in highly conductive borehole fluids.

To avoid errors in readings caused by incidental earth currents, polarization of the electrodes, and local corrosion potentials, the "d.c. resistivity" is measured at one of three selectable low frequencies. Filtering and synchronous detection are used to reduce interference. Also,

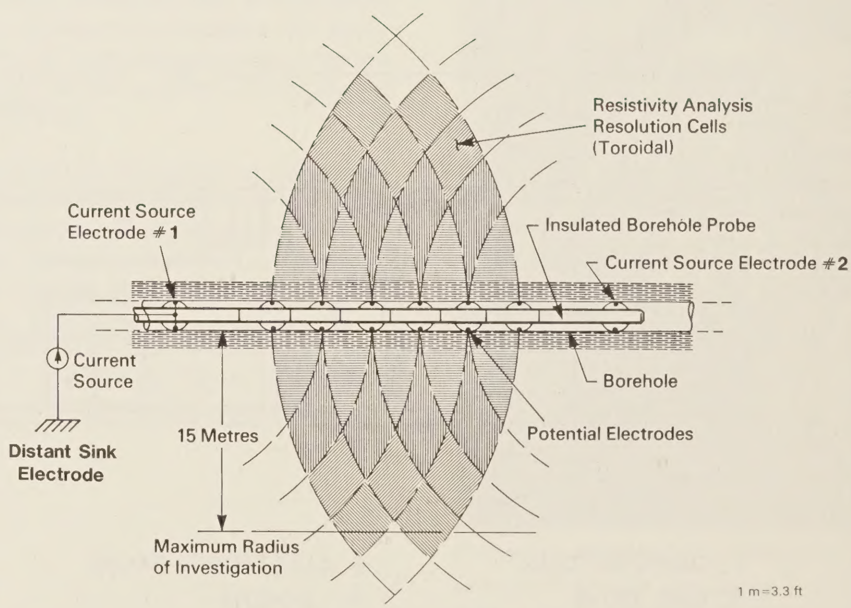

Figure 7.—Operational schematic of the long baseline resistivity probe and corresponding resolution cells.

the level of constant current used in the measurement is selected to provide a suitable voltage at the sensing dipole electrodes. Depending on soil and rock characteristics, the desired current may range up to 1 A and require a driving (or compliance) voltage of up to 500 V; therefore, caution is necessary when handling and testing the resistivity probes.

The long baseline resistivity probe operates on the identical physical principle as the short baseline probe. However, because the long baseline probe's function is to determine resistive anomalies at a radius of about 49 ft (15 m) from the borehole, the assembled probe is 103 ft (31.4 m) long but only 3 in (76 mm) in diameter. A probe this long and narrow is heavy and unwieldy and must be handled carefully to reduce bending moments. Therefore, this and other probes are assembled and disassembled module-by-module as they are inserted into and withdrawn from the borehole.

The probe consists of two 8.2 ft (2.5 m) long current injection modules at each end of a string of six potential measuring modules, each also 8.2 ft (2.5 m) long (fig. 8). The current injection modules are separated from the potential modules by 16.4 ft (5 m) long insulating spacers. Additional necessary information is obtained in inhomogenous earth by alternately injecting current on either side of the potential measuring electrodes.

Also, six 19.7 ft (6 m) pushrods are used to reduce the effects on readings of the protective steel casing through which the probes are pumped into the open borehole.

Seismic Probe

The use of seismic geophysical probes is well established, especially in the exploration for large, deep earth structures that might contain hydrocarbons. Because the Combination Probe requires

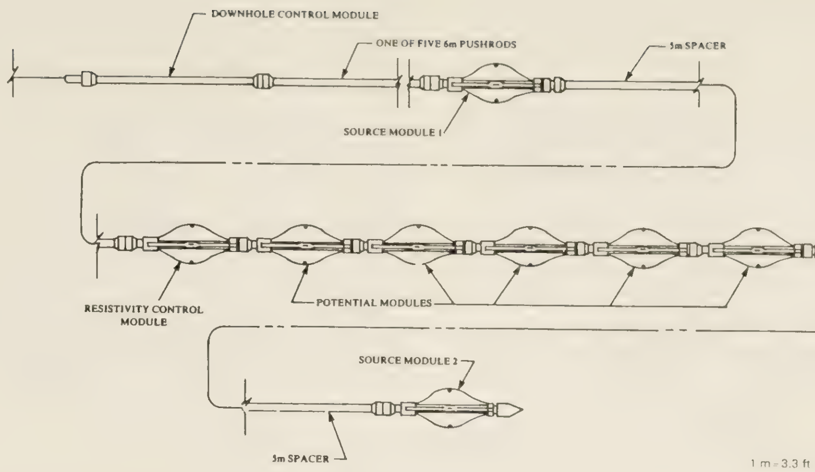

Figure 8.—Resistivity probe configuration.

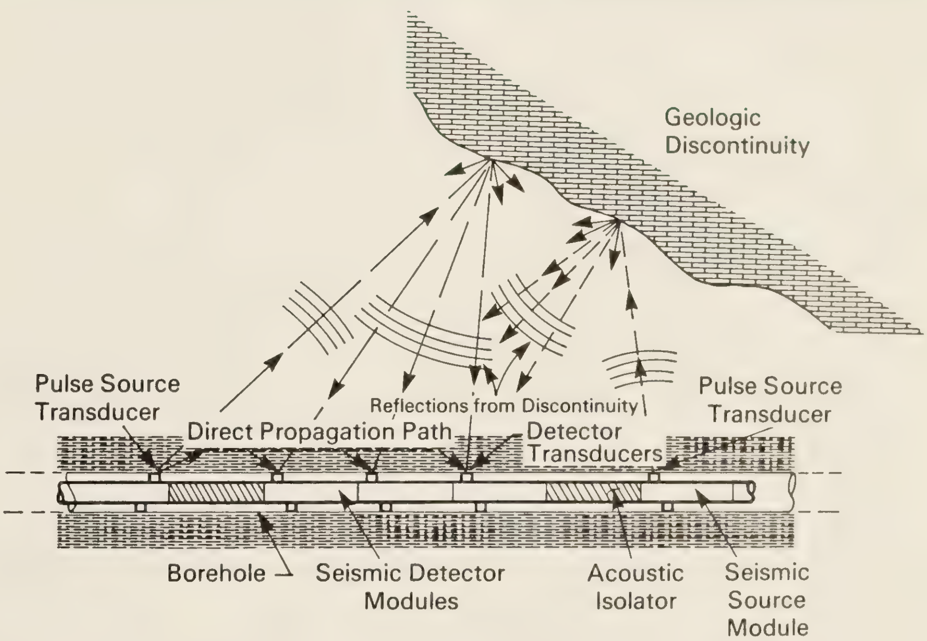

Figure 9.—Operational schematic of the seismic probe.

high resolution at a maximum distance of only 49 ft (15 m) from the borehole, acoustic signals can have a wavelength of about 3.3 ft (1 m) and still propagate satisfactorily in the expected soil and rock. The seismic probe performs two sensing functions—measuring the velocity of seismic compressional (P) waves and shear (S) waves in the adjacent borehole wall and detecting seismic reflections from interfaces, anomalies, and contrasts in geological structures at a range of up to 49 ft (15 m). The first measurement provides information on the nature of the soil and rock surrounding the borehole, and the second reveals interruptions and discontinuities in the soil and rock (fig. 9).

The seismic probe consists of two transmitting and three receiving modules that provide a variety of paths for propagation of signals into the soil and rock and for detection of reflections from discontinuities. The module configuration also speeds up operation by reducing the need to move the probe in small increments because the multiple received signals are used efficiently by available data processing routines. The probe is unusual in that it can operate in a wet or dry irregular borehole of varying diameter. This capability is achieved by packaging the five piezoelectric transducers into 1 in (25 mm) diameter cylinders. Each transducer can be pumped out of the module until solid mechanical (and thus acoustic) contact is established with the borehole wall, or as far as 1.25 in (31.8 mm). Two reaction cylinders (identical in appearance to the transducers) that are pumped out on the opposite side of each module are provided to accommodate irregular boreholes and boreholes of varying diameters.

Because sidewall contact must be made by the transducers in a dry borehole, pressure and limit switches are used to indicate whether the transducers make solid contact before all cylinders and transducers of each module are fully extended. If any transducer becomes fully extended, the system operator is signaled that the transducer's reading is questionable. Transducer retraction also is signaled, and interlock logic prevents the winch from being activated for withdrawal from the borehole before the transducers are completely retracted. Severe damage would occur if the probe were moved with transducers extended.

Under normal conditions, the hydraulic system design provides fail-safe operation by supplying the energy for retraction by a hydraulic accumulator in each module. However, for certain malfunctions, a backup, pump-driven hydraulic retraction mode is available.

Seismic pulse shape is controlled by the mechanical resonance characteristics of the piezoelectric transducer stack, which is pulsed by a 1,000 V charge applied to the parallel-connected piezoceramic disks of the transmitting transducers. Pulse energy is 0.07 J—the maximum allowable for each cylinder without danger of mechanical or electrical breakdown. The receiver stacks appear identical to those of the transmitter, but the elements are series-connected to maximize sensitivity.

To reduce the time and energy required to pump out all 15 cylinders of the piezoelectric array, a vent in each seismic module equalizes borehole and module pressures. This assures that hydraulic system pressures needed to actuate the cylinders remain constant regardless of liquid pressure in the borehole.

Highly efficient acoustic isolator modules prevent direct propagation of transmitter pulses through the bodies of intervening modules and into the receivers. The isolators provide tensile strength, electrical continuity, and metal-to-metal contact when in compression. However, when the isolators are subject to tension, a specially designed rope tension member prevents acoustic propagation through the isolators.

Electromagnetic Probe

The electromagnetic (EM) probe operates as an underground radar system, similar in principle to systems used for detecting the presence of and range to an airborne object. A radio wave is reflected from an object that is of appreciable size compared with the wavelength of the incident radiation and that has a characteristic impedance different from the propagation medium ahead of it. In the air, commonly used radar frequencies are propagated virtually without loss; however, because radar frequencies useful for detecting subterranean anomalies are heavily attenuated and delayed by many common underground materials, useful range can be very short. Also, electromagnetic wave propagation in rock is a dispersive process wherein attenuation and velocity are frequency sensitive.

The EM probe is classified as a "short pulse" radar in which pulses lasting a few nanoseconds are fed into a "transient radiating" antenna structure that radiates a wide band of frequencies. The receiver consists of an antenna and preamplifier that improves signal-to-noise ratio and matches antenna impedance to a time domain sampler. The time domain sampler converts the waveform of the 30 MHz to 300 MHz broadband radar spectrum into an identical waveform within the audio frequency spectrum. A selectable-exponent time-variable-gain amplifier is used to feed an analog-to-digital converter in the downhole module, which encodes data for transmission to the surface.

Because of the characteristics of underground propagation, the geophysical measurements made by the EM probe provide the following three basic kinds of information that can be analyzed and correlated with other measurements:

- Propagation velocity along the borehole wall, which is an important indicator of the ratio of resistivity to dielectric constant. Radial propagation velocity, a key element, can be calculated from the measured differences in time between reception of pulses reflected from discrete objects as the probe moves past them.
- Anomalies that produce reflections at a measured time (corresponding to distance) after initiation of the transmitted pulse.
- Propagation loss, which can be estimated from the maximum range at which signals of differing frequencies are detected (fig. 10).

With a conventionally designed antenna, the 3 in (76 mm) diameter of the borehole probe would make directivity (the ability to determine the radial direction of origin of a reflected signal) impossible. However, by using a transmitting antenna that incorporates high dielectric materials and a novel design principle, an unprecedented 10 DB of useful directivity has been measured under actual working conditions. This performance is possible because the transmitting antenna uses a phased, dual-dipole array and has a rotator system with eight preset rotational positions. The receiving antenna is an electrically short, omnidirectional monopole. The transmitter has a pulse repetition frequency of 300 KHz, a peak power of 780 W, and a pulse duration of 3.0 nanoseconds. However, antenna performance varies somewhat with the diameter of the borehole and the nature of the surrounding soil and rock.

Adjustments in the gain of RF preamplifiers necessitated by borehole characteristics are controlled from the surface. The directional antenna is rotated by a small electric motor and gearbox and can be commanded to stop in each of the eight preset positions. The surface unit, through the digital command system, requests control of position for successive readings. Commanded antenna position is compared with the indication given by either a fluxgate or pendulum sensor, respectively, in horizontal and vertical boreholes; logic circuits insure the antenna always rotates the shortest distance to its new position.

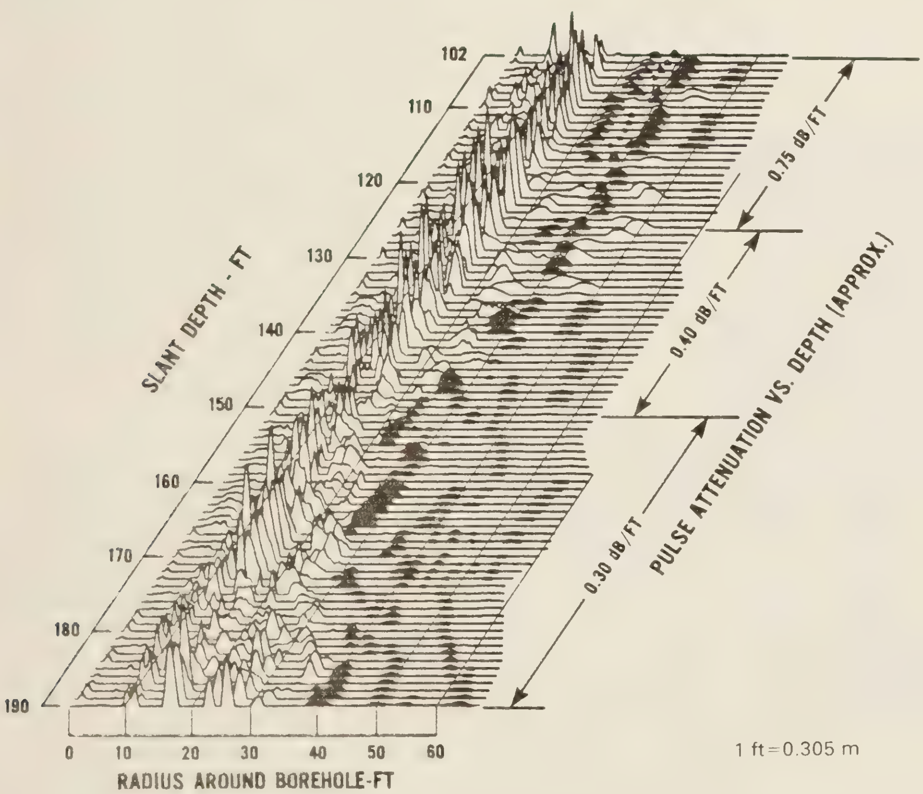

Figure 10.—Typical electromagnetic pulse reflections observed in a borehole.

Operational Experience

The Combination Probe has been used in two field data collection operations. The first operation was a shakedown test at Gold Hill, Colorado, in 1981. Because the MUSE was not completed then, operations took place in a relatively shallow vertical borehole. Although some data were collected, the principal result of the exercise was experience in setting up and operating the probe and in determining which subsystems displayed difficulties with temperature variations, immersion in water, and other adverse conditions.

The second field operation took place in 1982 at the Cumberland Gap, Tennessee, tunnel site where data were collected for approximately 3 weeks with all four probes in a 1,000 ft (305 m) horizontal borehole (fig. 11).[3] Despite heavy flow of water out of the borehole and a mudslide that stopped operations for 1 week, the Combination Probe System operated successfully in the complex geology of this site.

To determine the usefulness and cost-effectiveness of the Combination Probe System for tunnel support design, an attempt was made to interpret the data collected with the probes in terms of rock drilling characteristics, rock quality, and rock strength *without* using the core samples taken to aid in interpreting the data. Although large quantities of repeatable data were collected, the interpretation was not possible because of the characteristics of the borehole and lack of experience in evaluating the probe data with the material found at ground truth.

Major impediments to interpreting the long baseline resistivity data included the frequent presence of thin bedding layers through which the borehole passed at a dip angle of 30 to 40 degrees. Presently a mathematical model for interpreting data exists only for angles near 0 or 90 degrees.

The electromagnetic and the seismic data were confused by "tube mode" propagation in which the signal reflects from elements of the probe and borehole and passes repeatedly through the volume being sensed. These repeated artifact signals cannot be removed easily even using sophisticated computer techniques because the propagation of the tube wave is affected by the varying parameters (including dispersion) of the material lining the borehole. Data analysis at this site was further hampered because the extraordinarily low conductivity of the borehole fluid (0.20 micromho) did not attenuate the borehole mode electromagnetic wave. Borehole mode acoustic waves were difficult to interpret because reference information on the acoustic properties of borehole materials was not available.

Despite these problems, excellent correlation was found between the short baseline resistivity measurements and the direct arrival EM pulses and between the seismic and EM/resistive measurements as the probe passed through different borehole wall materials. The differing borehole materials were characterized by their electrical, electromagnetic, and seismic parameters and classified on the basis of their similarity to materials whose properties are documented. However, for tunnel design it is necessary to describe the physical condition of the known materials, and in this analysis there were too many unknowns. Despite the difficulties encountered, it was possible to predict that there were no significant anomalies within about 33 ft to 49 ft (10 m to 15 m) of the borehole.

Status of the Combination Probe System

The Combination Probe System has promise as a powerful tool for reducing tunnel construction costs by collecting and interpreting geophysical data taken at prospective tunnel sites. However, potential users must gain confidence in the system before tunnel designs will be based on the system's output rather than on data collected by other means.

[3] B. M. Duff, "Field Tests of the Mobile Underground Site Evaluation System at the Cumberland Gap Tunnel Site," Southwest Research Institute, San Antonio, Tex., July 1983. Unpublished report prepared for the Federal Highway Administration.

Figure 11.—Probe withdrawal from water-filled borehole at the Cumberland Gap Tunnel site.

To make the probe a reliable and cost-effective tool for tunnel site investigation, an increasingly extensive data base must be provided from the study of probe data and ground truth. Therefore, the Cumberland Gap probe data should be reevaluated in conjunction with physical examination and measurement of the cores taken when the borehole was drilled.

A series of sophisticated algorithms and computer programs for processing data generated by the four probes has been developed recently. The short range objective of the algorithms and computer programs is to present easily understood graphs of processed probe data. The long range objective is to develop a map of the underground showing probable natural structures and materials, including estimates of size and location of inclusions and anomalies. Recent efforts concentrated on separating signal reverberations within the borehole from more distant ones in the strata of interest. Although analytical methods are advancing toward this goal, the long range objective will be met only through experience and repeated correction and refinement of estimates. Therefore, construction of the Cumberland Gap Tunnel will be an important step in the development sequence of the Combination Probe System.

Acknowledgment

The author wishes to acknowledge the dedicated efforts and professional skill of Dr. Bob Duff of the Southwest Research Institute, San Antonio, Texas, who led the work of developing the data processing algorithms and field testing the Combination Probe System.

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(*1*) L. A. Rubin et al., "A New Sensing System for Pre-Excavation Subsurface Investigation for Tunnels in Rock Masses, Volume II," Report No. FHWA–RD–77–11, *Federal Highway Administration,* Washington, D.C., August 1976. PB No. 276721.

(*2*) L. A. Rubin et al., "A New Sensing System for Pre-Excavation Subsurface Investigation for Tunnels in Rock Masses, Volume I," Report No. FHWA–RD–77–10, *Federal Highway Administration,* Washington, D.C., August 1976. PB No. 276720.

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

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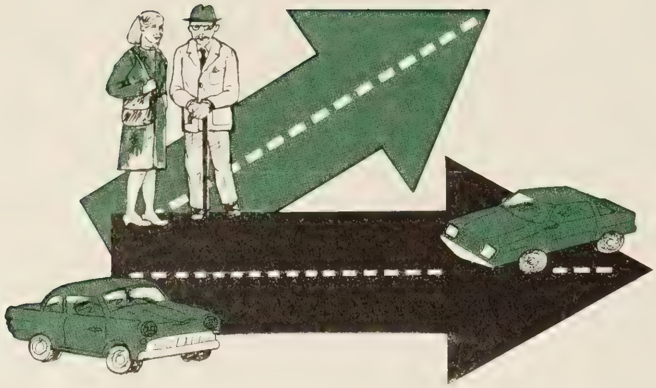

Driver Characteristics Impacting Highway Design and Operations

by
Donald A. Gordon, Hugh W. McGee, and Kevin G. Hooper

Introduction

Highway design standards should be reviewed periodically and modified as necessary to reflect changing highway conditions and findings from research on driver behavior. Recently, smaller and lighter vehicles have become prevalent on the road. Also, a larger proportion of women and older persons are driving. Female drivers now comprise approximately 50 percent of licensed drivers, and because women generally are shorter than men, the change in the composite eye height distribution may affect highway design standards. The percentage of drivers 60 years and older has increased during the past 40 years from 5 to over 15 percent. (*1–3*)[1] Because older drivers tend to react more slowly than younger drivers, requirements for sight distance at intersections and railroad crossings may have to be increased. Older drivers also tend to have poor night vision, implying a need for increased night luminance of overhead guide signs.

This article discusses the findings and recommendations of a Federal Highway Administration (FHWA) study to analyze the driver components of highway design standards.[2]

[1] Italic numbers in parentheses identify references on page 16.

[2] "Highway Design and Operations Standards Affected by Driver Characteristics," Report No. FHWA/RD–83/065, Federal Highway Administration, Wasington, D.C. Not yet published.

FHWA Study of Highway Design Standards

Highway standards literature was reviewed to identify the specific design standards involving driver characteristics. Some of the more significant American Association of State Highway and Transportation Officials (AASHTO) standards reviewed include the following:

- Geometric Design Guide for Resurfacing, Restoration and Rehabilitation (R-R-R) of Highways and Streets, 1977
- Highway Design and Operational Practices Related to Highway Safety, Second Edition, 1974
- A Policy on Design of Urban Highways and Arterial Streets, 1973
- Geometric Design Guide for Local Roads and Streets, 1971
- A Policy on Design Standards for Stopping Sight Distance, 1971
- Manual for Signing and Pavement Marking of the National System of Interstate and Defense Highways, 1970
- Geometric Design Standards for Highways Other than Freeways, 1969
- A Policy on Design Standards, Interstate System, June 20, 1967
- A Policy on Geometric Design of Rural Highways, 1965

As a result of the reviews, standards containing an explicit specification of a driver characteristic were identified (table 1). Other standards, influenced indirectly by driver characteristics, also were identified. These standards include highway design speed, pedestrian refuge island design, sidewalk width, roadway lighting, and sign and traffic signal heights.

Sensitivity analyses were performed to determine if each standard was responsive to a change in the associated driver characteristic specification. For example, it was found that a 3.75 ft (1.14 m) eye height specification provides only a 2.5 percent longer sight distance than a 3.5 ft (1.07 m) specification. Therefore, an appreciable change in the eye height

Table 1.—Standards based on an explicit specification of a driver characteristic

Standard	Driver characteristic	Specification
Sight distance, general:		
Stopping sight distance	Brake reaction time	2.5 sec
Design passing sight distance	Initial maneuver time	3.6–4.5 sec
Decision sight distance	Detection and recognition time	1.5–3.0 sec
	Decision and response time	4.2–7.0 sec
	Maneuver time	3.5–4.5 sec
Sight distance measuring criteria	Eye height	
Sight distance, specific:		
Intersection sight distance[1]		
• Case I	Perception-reaction time plus time to brake or accelerate	2.6 sec
• Case II	Brake reaction time[2]	2.5 sec
• Case III	Perception time and time to actuate an automatic shift	2.0 sec
Railroad-highway grade crossing sight distance[3]		
• Case I	Perception-reaction time	2.5 sec
• Case II	Perception-reaction time	2.0 sec
Sight distance along a ramp	Brake reaction time[2]	2.5 sec
Sight distance through a grade separation	Brake reaction time[2]	2.5 sec
Sight distance at a ramp terminal	Perception time and time to actuate an automatic shift	2.0 sec
Horizontal alinement:		
Lateral clearance to sight obstructions on horizontal circular curves	Brake reaction time[2]	2.5 sec
Vertical alinement:		
Crest vertical curve lengths	Eye height	3.5 ft
	Brake reaction time[2]	2.5 sec
Sag vertical curve lengths	Brake reaction time[2]	2.5 sec
Sag vertical curve lengths through a grade separation	Eye height	3.5 ft
	Brake reaction time[2]	2.5 sec
Traffic control devices:		
Adequate gap time for school crossing warrant for traffic signal installation	Pedestrian walking speed	3.5 ft/sec
	Pedestrian perception-reaction time	3.0 sec
Traffic signal face location	Cone of vision	40°
Yellow vehicle clearance interval	Perception-reaction time	1.0 sec
Pedestrian clearance DON'T WALK interval	Pedestrian walking speed	4.0 ft/sec

[1] Case I requires the driver to adjust speed to avoid a conflicting vehicle on a cross street. Case II involves the decision whether to continue, accelerate, or stop. Case III involves intersections controlled by stop signs.

[2] Driver characteristics that indirectly influence design standards.

[3] Case I refers to a passive grade crossing where there are no gates. Case II involves a crossing where the driver must stop.

1 ft = 0.305 m

specification would have only a minor effect on sight distance. In contrast, pedestrian signal timing (time to cross the street during the "WALK" interval) is very sensitive to the pedestrian speed of walking specification. The sensitivity analyses thus identified those standards significantly influenced by a driver or pedestrian characteristic specification.

Recommendations

Based on results of the sensitivity analyses and what presently is known about the distribution of driver characteristics, changes to specific highway standards were recommended. These recommendations, made to accommodate the 85th percentile drivers, are offered for information only; additional empirical research is underway.

Stopping Sight Distance. Drivers must have sufficient sight distance to stop a vehicle to avoid hitting an object on the roadway. The perception-brake component of stopping sight distance is the time the driver requires to see the object and apply the brakes. Time and distance also must be available for the vehicle to decelerate and come to a stop. The complete AASHTO formulation of stopping sight distance is as follows:

$$SSD = 1.47\,PV + \frac{V^2}{30(f+g)}$$

Where,
SSD=Stopping sight distance (ft).
P=Perception-braking time (sec).
V=Initial speed (mph).
f=Coefficient of friction between tires and road surface.
g=Grade (percent).

The presently accepted AASHTO value of perception-braking time is 2.5 seconds. A time of 3.2 seconds is suggested as being more appropriate. As shown in table 2, the sum of latency (the time between the appearance of an obstacle and start of the driver's response), eye movement, fixation, recognition, decision, and the motor action of applying the brake equals a reaction time of approximately 3.2 seconds for the 85th percentile driver. This reaction time was reached by summing the component reactions, which logically are part of the response.[3] At 30 mph (48 km/h), the calculated minimum sight distance would be 16 percent higher than a 2.5 second perception-braking reaction; at 70 mph (113 km/h), the calculated minimum sight distance would be 10 percent higher.

Table 2.—Perception-brake reaction time for various percentiles of driving population (Stopping sight distance, sec)

Element	Percentile of drivers 50	75	85	90	95	99
Perception						
Latency	0.24	0.27	0.31	0.33	0.35	0.45
Eye movement	0.09	0.09	0.09	0.09	0.09	0.09
Fixation	0.20	0.20	0.20	0.20	0.20	0.20
Recognition	0.40	0.45	0.50	0.55	0.60	0.65
Decision	0.50	0.75	0.85	0.90	0.95	1.00
Brake reaction	0.85	1.11	1.24	1.42	1.63	2.16
Total	2.3	2.9	3.2	3.5	3.8	4.6

Passing Sight Distance. AASHTO design policy defines passing sight distance as the sum of four distances: (1)The initial distance to the point of entrance into the left (passing) lane, (2) actual passing distance, (3) the distance between the passing vehicle and the opposing vehicle at the end of the maneuver, and (4) the distance traveled by an opposing vehicle for two-thirds of the distance of the actual passing distance of the other vehicle. The current AASHTO minimum sight distances differ from the Manual on Uniform Traffic Control Devices (MUTCD) minimum sight distances used as warrants for placing no-passing zone pavement stripes on completed highways. The current AASHTO specifications for passing sight distance are based on data collected before 1960. The performance of newer vehicle models should be represented in the standard.

Intersection Sight Distance. Because intersections differ in their requirements for driver response, three separate and distinct cases have been identified (table 1). Case I concerns sight distance at an uncontrolled intersection where the driver simply must adjust speed to avoid a conflicting vehicle on a cross street. Where a crossing is not controlled by YIELD signs, STOP signs, or traffic signals, the operator of a vehicle approaching an intersection must be able to perceive a hazard in sufficient time to alter the speed of the vehicle as necessary before approaching the intersection. The formulation for Case I sight distance provides 2.6 seconds for the driver to perceive and recognize an approaching vehicle, to decide what evasive action is necessary, to move the foot to the accelerator or the brake, and to take the evasive action. Analyses of perception-reaction times, based on summing the times for latency, eye movement, fixation, recognition, decision, and brake action (applying the brakes) suggest the 85th percentile driver would require 3.4 seconds or longer to react (table 3). The current sight distance response time of 2.6 seconds does not allow sufficient time.

[3]It is not certain whether summing the elements of perception-braking time is a valid estimate of reaction time. This process currently is being evaluated through experimental investigation.

Table 3.—Perception-brake reaction time for various percentiles of driving population (Intersection sight distance, sec)

Element	Percentile of drivers 50	75	85	90	95	99
Perception						
Latency	0.28	0.33	0.34	0.36	0.39	0.45
Eye movement	0.12	0.12	0.12	0.12	0.12	0.12
Fixation	0.20	0.20	0.20	0.20	0.20	0.20
Recognition	0.40	0.45	0.50	0.55	0.60	0.65
Decision	1.00	1.25	1.35	1.40	1.50	1.50
Brake reaction	0.63	0.82	0.92	1.05	1.21	1.60
Total	2.6	3.2	3.4	3.7	4.0	4.5

In Case II, the driver perceives a vehicle moving across the path, judges its trajectory, and decides whether to continue, accelerate, or stop. In this set of conditions for crossings not controlled by YIELD signs, STOP signs, or traffic signals, it is assumed that the operator of a vehicle on either highway must be able to see the intersection and the intersecting highway in sufficient time to stop the vehicle, if necessary, before reaching the intersection. The standard perception-reaction braking time is 2.5 seconds. Calculated perception-reaction times for the 85th percentile driver, including latency, eye movement, fixation, recognition, decision, and braking is 3.4 seconds (table 3). Again, the standard does not appear to allow the driver adequate time.

In Case III, traffic is controlled by STOP signs on the minor road of an intersection. For safety reasons, the driver of a stopped vehicle must see enough of the major highway to be able to cross before a vehicle on the main highway reaches the intersection, even though this vehicle comes into view just as the stopped vehicle starts to cross.

Railroad-Highway Grade Crossing Sight Distance. At a passive railroad grade crossing where there are no gates, the driver must scan the tracks and decide whether to stop or proceed. In making the judgment, the driver must first detect and recognize the railroad crossing and perceive that there are no gates. The driver then must search right and left and decide whether to stop or proceed. A perception-reaction time of 3.5 seconds, representing the 85th percentile driver, appears more appropriate than the 2.5 second interval now used.

Vehicle Clearance Interval. The yellow traffic signal, lasting between 3 and 6 seconds, warns the driver of an impending change to the stop (red) condition. Current practice assumes a perception-reaction time of 1 second. A more valid estimate of perception-reaction time based on the 85th percentile population decision and brake action time is 1.77 seconds.

Pedestrian Signal Timing. The MUTCD provides a WALK interval of sufficient length to allow a pedestrian progressing at 4 ft per sec (1.2 m per sec) to leave the curb and move to the center of the farthest traveled lane. At the end of the interval, vehicles on the opposing lanes receive a green indication. Studies show approximately 30 percent of the adult pedestrian population is excluded by the current specification. (*4*) The design specification for walking speed should be decreased to 3.5 ft per sec (1.1 m per sec) to allow sufficient time for the pedestrian to reach the opposite curb. The 3.5 ft per sec (1.1 m per sec) walking speed would accommodate the 85th percentile pedestrian.

Sign Letter Height. Letters on signs must be large enough to allow the driver to read the sign and carry out maneuvers indicated by the message. The rule of 50 ft (15.2 m) (distance from driver's eye to the sign) to 1 in (25.4 mm) of letter height allows drivers with 20/24 visual acuity at least 10 seconds of maneuvering time.[4] Drivers with visual acuity poorer than 20/24 would need larger letters to complete the maneuver in 10 seconds. Current sign letter height specifications included in the MUTCD do not adequately consider the needs of the elderly, whose average letter legibility is 26 ft (7.9 m) per 1 in (25.4 mm) of letter height. As much as 25 percent of the driving population may be unable to read highway guide signs in sufficient time to comfortably or safely react. Thus, the letter size on highway signs needs to be increased.

Required Research

This study is an important initial step in the scientific examination of current highway design standards. However, the extent the findings are supported by empirical (experimental) evidence needs to be determined.

[4]An acuity of 20/24 indicates that a person can read the same Snellen test letters at 20 ft (6.1 m) that a person of normal vision can read at 24 ft (7.3 m).

Empirical verification would resolve a number of uncertainties. In some cases, the driver may be able to adapt to and overcome minor geometric inadequacies. The older driver may drive less often at night and may show increased caution when doing so. The short driver can adjust the seat or sit on a pillow. The driver approaching an obviously dangerous intersection may slow in advance. The extent and effectiveness of these driver adaptions is not presently known.

Also, the method of determining driver response time as the sum of latency, eye movement, fixation, and other time intervals requires empirical verification. The driver's response time may represent an integrated reaction, which is shorter than response time calculated by adding separate acts.

In summary, this study has contributed to improving geometric design standards to reflect the changing vehicle mix and driver population on the roadway today. As a result of the sensitivity analyses conducted, empirical studies, some of which are now being undertaken, can be aimed more precisely at the geometric standards based on driver characteristics.

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(*1*) "Highway Statistics 1979," *Federal Highway Administration,* Washington, D.C., 1979.

(*2*) "Nationwide Personal Transportation Study," *U.S. Bureau of the Census,* 1969–1970.

(*3*) G. W. Marsh, "Aging and Driving," *Traffic Engineering,* vol. 31, No. 2, 1960.

(*4*) R. B. Sleight, "The Pedestrian," *Human Factors in Highway Traffic Safety Research,* T. W. Forbes, ed., *John Wiley & Sons, Inc.,* 1972, pp. 224–253.

Donald A. Gordon was a senior research psychologist in the Systems Technology Division, Office of Safety and Traffic Operations Research and Development, Federal Highway Administration. He passed away February 3, 1984. He was nationally known in research for his work on the driving process and the development of improved signs and roadway delineations. Dr. Gordon had articles published on perception in vehicular guidance, sign legibility and understanding, and the contribution of psychology to the traffic flow theory. His most recent research included overhead guide sign visibility issues, effects of changing driver characteristics, and recommendations on flagger vest designs.

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Foundation Engineering Research: Part 2—Pile Foundations

by

Albert F. DiMillio

This is the second of two articles on current efforts to improve highway bridge foundations. This article discusses research efforts to improve pile design and installation methods. Part 1, which was published in the March 1984 issue of *Public Roads* (vol. 47, No. 4), discussed spread footings and ground improvement techniques that can enhance the use of spread footings in lieu of piles. The foundation selection process also was discussed briefly to illustrate how a designer can decide whether piles are necessary.

Pile Design

Choosing the right kind of foundation is difficult and is based on soil and site conditions (described in Part 1). Scour and the relative inability of spread footings to transfer inclined, horizontal, or uplift forces and overturning moments require the use of piles in many instances.

The first problem facing the pile designer is choosing from among the 100 or more different kinds of piles. There are many variations in materials, configurations, and installation techniques. Guidelines for selecting the best pile for various situations can be found in the literature. (*1–3*)[1]

After a particular kind of pile has been selected for use, the designer then must determine the number, length, and size of the piles required. Simple guidelines are not available to design and analyze piles for the various situations that can occur in bridge foundations. The ultimate load that can be supported by a certain kind of pile depends on the strength of the soil and/or pile material. Usually the ultimate load is determined by soil failure; however, the pile itself may fail (fig. 1) if forced to penetrate dense soil or rock.

Calculating the ultimate load based on soil failure is the most difficult problem confronting the foundation designer. Because current theories for determining the ultimate load of a pile and a group of piles are not accurate enough to provide economical foundation designs, research is being conducted to define the

[1]Italic numbers in parentheses identify references on page 24.

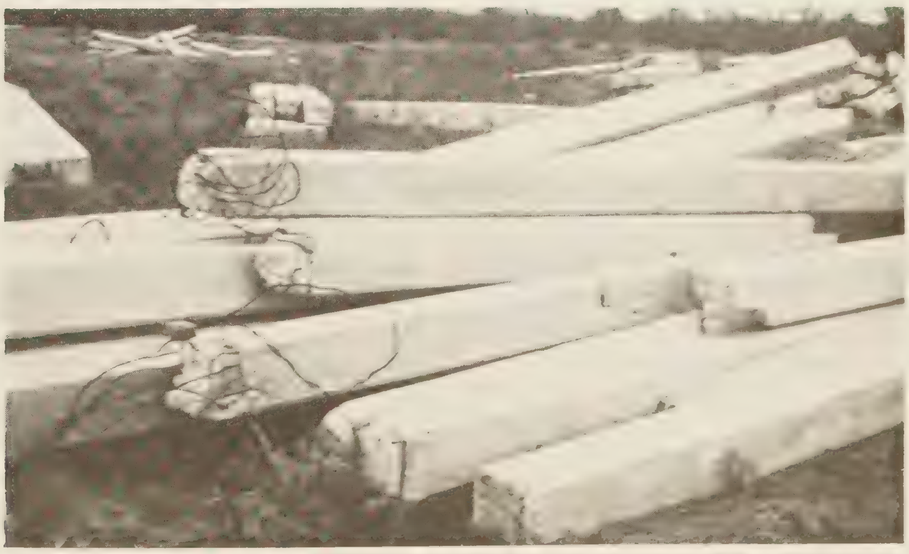

Figure 1.—Concrete piles damaged during driving.

complex mechanics of load transfer between pile and soil and among the various piles within a pile group. Load transfer in piles and pile groups is being measured in the laboratory and field to develop and refine analytical models of the pile behavior process to provide more economical design procedures.

Assuming the pile material is not overstressed, the ultimate load capacity of a pile is equal to the sum of two major soil components—point resistance and side friction. The amount of support contributed by each component varies according to the soil properties and pile dimensions. These resistances can be calculated through mathematical relationships; however, the required input data are difficult and expensive to obtain. As a result, less expensive index testing usually is performed to obtain approximate values for estimating the resistances.

Pile capacity usually is predicted using static and dynamic analysis procedures. On large, expensive structures, pile load tests often are used to verify the design loads. Recently completed experimental studies of instrumented piles load tested to failure have increased our understanding of load transfer behavior of single piles and pile groups; however, more tests are needed to confirm the new prediction methods.

The static analysis of pile capacity requires knowledge of the shear strength parameters of the soil underneath and surrounding the pile. The static load capacity of a pile is equal to the unit soil strength (estimated from laboratory and/or in situ field tests on relatively undisturbed soils) multiplied by the surface area of the pile shaft plus the bearing area of the pile tip. These computations are used to estimate the number and length of piles required at a site. It should be noted, however, that soil properties can change significantly because of the pile driving process (depending upon the type of soil involved), thus changing the load capacity of the piles.

A wave equation (dynamic) analysis also is used in the design process to determine the driving resistance that the pile will encounter and the compatibility of the pile driving equipment. The wave equation analysis is based on the stress wave from the hammer as it strikes the pile. The speed that the force pulse, developed in the pile from the hammer impact, travels down the pile depends on the pile material. Soil frictional and tip resistances attenuate the force pulse. Relationships between pile capacity and driving resistance, pile drivability, and overstressing of the piles can be determined from a wave equation analysis.

Other complicating factors involved in pile design include negative skin friction and horizontal loads on piles. Negative skin friction occurs when the soil surrounding the pile settles. The settling soil grips the pile and applies a drag load that can severely damage a structure. Research is concentrating on methods to reduce negative skin friction, including coating the piles with bitumen or other materials to reduce the frictional forces.

Horizontal forces can cause dangerous lateral loading that can fail a pile in bending or shear. Design solutions are available; however, additional, carefully controlled experiments are required to compare predictions with observed behavior to refine existing design methods.

At the outset of the pile foundation design process, semiempirical procedures are used to size the pile group for a trial design. To refine the trial design and perform cost analyses pile forces are estimated and codes, specifications, and experience are applied. As previously mentioned, load testing during the design phase of large, expensive structures often verifies and refines a trial design. Mathematical models often are used to predict or verify some aspect of the trial design such as constructability or performance.

A mathematical model of pile group behavior is a valuable analytical tool that systematically can convey engineering experiences from one site to another. Several mathematical models have been developed in recent years; however, none has been validated adequately because of the lack of precise field data on pile group behavior. This is especially true of new foundation design methods based on finite element analysis.

Pile Installation

Proper construction control of pile driving operations is as essential as good design practices. Construction control is more difficult for piles than for spread footings because the excavation and construction of footings can be observed. The construction control for pile driving involves checking the pile materials and the installation equipment. The pile inspector can visually check many requirements; however, some of the most important checks require instrumentation.

For example, the control of pile driving stresses and the measurement of hammer efficiency require instrumentation and equipment (fig. 2) not normally found on routine pile driving projects. Current Federal Highway Administration (FHWA) research efforts are directed at improving the accuracy, mobility, and simplicity of these instrumentation devices.

Figure 2.—Pile analyzer equipment used to monitor pile installation.

Current Research in Pile Foundations

According to a recent survey performed for FHWA, approximately 70 percent of U.S. highway bridges are supported by piles, 24 percent by spread footings, and 6 percent by drilled shafts and caissons. (*4*) Because of the high reliance on driven piles, the majority of FHWA funds for foundation research has been directed at improving design and construction procedures for driven piling. FHWA pile research is divided into the following three major areas of emphasis:

- The interaction between a single pile and the surrounding soil to develop accurate prediction methods for pile capacity.
- The behavior of groups of piles to determine appropriate efficiency factors that must be applied to predictions based on single pile design.
- Pile materials and driving systems.

Single piles in clay

Field load tests on single piles in soft clays have provided experimental data to evaluate a new and promising pile capacity predictive technique based on the general effective stress method. The effective stress method for predicting pile capacity is being validated and its accuracy and general applicability are being improved.

The accuracy of the method is limited by the assumptions made to describe the changes in effective stress between pile driving and subsequent loading. Major uncertainties arise in attempted modeling of the effective stresses after the disturbed soil has reconsolidated. Current analysis techniques are considered applicable to full displacement piles driven into normally consolidated clays.

The general effective stress method entails a sequential analysis of the major events in the life of a driven pile in clay soils. The effective stress at the pile-soil interface is traced from preinstallation through pile installation, reconsolidation, and loading. The concepts of critical-state soil mechanics are used to model the large strains resulting from complete remolding of clay soils during pile driving. The completed pile load tests on single piles in soft clays showed good correlation between measured and predicted capacities.[2]

Single piles in sands

Current methods for predicting the behavior of driven piles in sand give widely different results because of several sources of error. Variability of the soil and methods used to obtain the design parameters contribute to the different results as do the simplifying assumptions made in developing the theoretical methods.

A current study on the behavior of piles in cohesionless soils involves the analysis of data on instrumented piles tested to failure under vertical loads.[3] The data, collected through an extensive literature search, consisted of 35 pile load tests at 10 different sites. The piles were of various kinds, lengths, and diameters and included steel pipe and H—piles and prestressed concrete and timber piles. Average pile diameter was 1.29 ft (0.4 m), and average pile length was 49.6 ft (15.1 m).

Statistical analyses determined the vertical and horizontal variability of the soil at each case history site. Load transfer characteristics were analyzed for each pile without consideration of residual driving stresses. In those few cases where residual stress data were available, the load transfer characteristics were reanalyzed to

[2] "Amoco Effective Stress Axial Capacity Cooperative Program," McClelland Engineers, Inc., Woodward-Clyde Consultants, and Amoco Production Company, July 1980. Unpublished report prepared for the Federal Highway Administration et al.

[3] J. L. Briaud, L. Tucker, R. L. Lytton, and H. M. Coyle, "Behavior of Piles and Pile Groups in Cohesionless Soils," Report No. FHWA/RD—84/007, Federal Highway Administration, Washington, D.C. Not yet published.

learn the effect of residual driving stresses. It was found that residual stresses play an important role in pile design and that proper consideration of residual stresses can result in shorter pile lengths for driven piles in sand.

Pile groups

The state-of-the-art for pile foundation design does not include an accurate method for relating the ultimate bearing capacity and settlement behavior of a single pile to the behavior of a group of closely spaced piles. To develop such a method, it is necessary to conduct field load tests to failure of full-scale pile groups to obtain field data that are useful in interpreting fundamental phenomena that control the behavior of groups of driven piles.

In 1978, FHWA initiated a research program to investigate pile group behavior through carefully performed experiments. First, existing mathematical models used to design pile groups were identified and evaluated. From this evaluation, the "hybrid model"[4] was selected and used to analyze a proposed full-scale pile group to be load tested. The hybrid model, a load-deformation model, reasonably predicts load versus settlement, load transfer patterns, and load distribution to pile heads. The model was especially helpful in designing the instrumentation system for the full-scale pile group load test. The field data acquired from the load test were used to refine the new method, the FHWA PILGP1 computer program, a modification and refinement of the hybrid model. (*5*)

Pile groups in clay

To develop the field data required to verify and refine PILGP1, 11 instrumented steel pipe piles were driven into a very stiff, saturated, overconsolidated clay soil at the University of Houston, Texas, campus. The outside diameter of the piles was 10.75 in (273 mm), with a wall thickness of 0.365 in (9.27 mm). The piles were driven closed-ended 43 ft (13.1 m) deep. Nine piles were driven in a 3×3 square array on a spacing of three pile diameters (fig. 3). The two remaining piles were driven apart from the group to serve as controls.

Figure 3.—Nine-pile group of steel piles at Houston, Texas, load test site.

Each of the 11 piles was instrumented with full bridge strain gage transducers and mechanical telltales to monitor load transfer from the piles to the surrounding soil. Four of the group piles and one control pile were instrumented with piezometers and lateral total pressure cells. The surrounding soil also was instrumented with piezometers and vertical ground movement devices. Electronic load cells measured applied loads, and settlements were measured at each pile head. Three-dimensional translations were measured on the massive concrete pile cap that was suspended off the ground.

The load testing program consisted of 11 compression and 6 uplift (tension) tests, all of which were carried to ultimate failure loads. The control piles were load tested in compression at three time intervals to study the effect of soil setup. Each control pile also was tested in uplift. The nine-pile group was tested in compression at three time intervals to assess setup characteristics of the group. Four of the group piles were tested in uplift; however, these tests were preceded by compression tests on two smaller groups of four and five piles. The five-pile subgroup was formed by cutting away the four corner piles, leaving the four edge piles and the center pile. The four-pile subgroup was formed by removing the center pile. (*6*)

Pile group prediction symposium: In conjunction with this load testing study, a pile group prediction symposium was held to identify various design methodologies used by governmental agencies, consultants, and universities in the United States and to predict the behavior of the experimental pile group and control piles in the load test study. These predictions were compared with the measured results from the research study.

Each member of the symposium was required to give a detailed description of the methodology used to predict the behavior of the experimental pile group and control piles. The compilation of methodologies described in the symposium proceedings is a valuable reference for practicing engineers.[5] Initial predictions were based on pretest information describing the foundation soils (field and laboratory test data), pile dimensions, and test loading procedures. The results of the load tests on the single control piles were given to the members after they had submitted their initial predictions so that, if necessary, they could modify their predictions for the experimental pile group based on the control pile results.

[4]M. W. O'Neill, O. I. Ghazzaly, and H. B. Ha, "Assessment of the Hybrid Model for Pile Groups," paper presented at the Transportation Research Board Meeting, Washington, D.C., January 1979.

[5]"Proceedings of the Pile Group Prediction Symposium," November 1980, Report No. FHWA–TS–84–203, Federal Highway Administration, Washington, D.C. Not yet published.

Pile groups in sand

A pile group load test study was performed on a group of eight timber piles in sands at the Locks and Dam No. 26 near Alton, Illinois, as part of an evaluation of several rehabilitation schemes for the distressed locks and dam structures (fig. 4). The timber piles were instrumented to measure load transfer and deformation up to and including the failure load.[6] The acquired field data were used to refine the PILGP1 program. A second series of pile group load tests in sands currently is under design.

Inservice monitoring of pile groups

In addition to the pile group load tests to failure, four full-scale field projects were initiated to observe pile group behavior under working loads. The short and long term behavior of the inservice piles has been and will continue to be compared with analytical predictions made by PILGP1.

One of the projects is located on the Natchez Trace Parkway in Mississippi, where a pile-supported bridge abutment was instrumented to obtain load transfer data on a group of six steel piles (12 HP 53) in soft clay and silt. Another project is located at Fort Belvoir, Virginia, where soil and pile conditions are similar to those at the Mississippi project. Instrument readings were taken weekly during the first year at each site, monthly during the second year, and will be taken quarterly for several years.

[6] "Axial Load Tests on Monoliths M5 and M6," Woodward-Clyde Consultants, Chicago, Ill., Feb. 20, 1980. Unpublished report prepared for U.S. Army Corps of Engineers and the Federal Highway Administration.

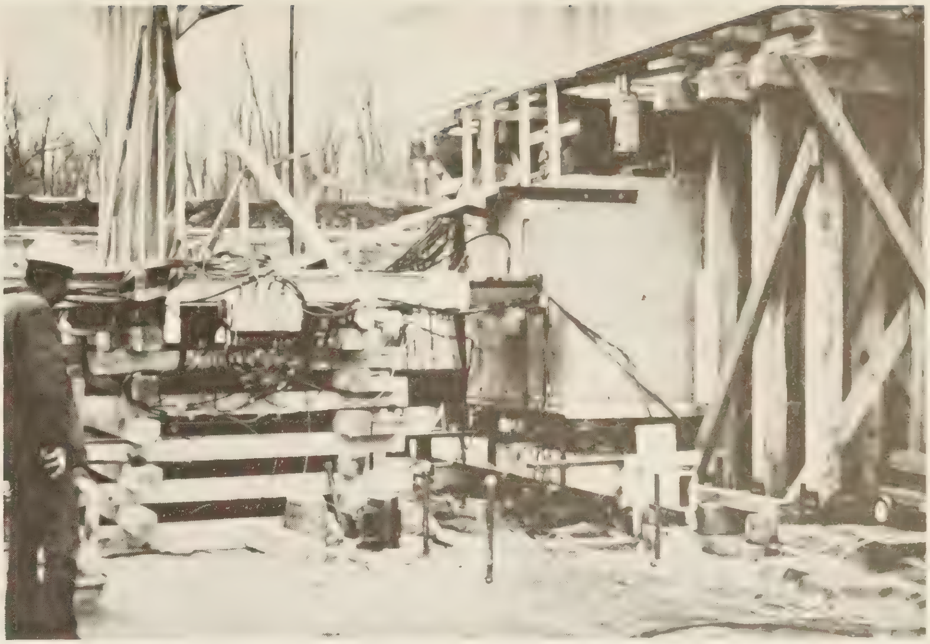

Figure 4.—Pile load test on group of eight timber piles at Locks and Dam No. 26 near Alton, Illinois.

The third site, at the Mocks Bottom overcrossing in Portland, Oregon, near Swan Island, is underlain by a thick compressible clayey silt deposit. High downdrag loads were expected because of the approach embankment loads on the compressible soil. A bitumen coating was used to reduce downdrag loads by about 90 percent. Although the bitumen-coated piles cost about 15 percent more than the uncoated piles, fewer piles were required. (7) Pile instrumentation included settlement and load transfer monitoring.

The fourth site is located at the West Seattle Freeway in Washington where a group of twelve 24 in (610 mm) diameter concrete piles supports a pier in medium-dense sands. The bridge pier and pilecap were instrumented to measure the amount of load transferred to the pile cap, and each pile was instrumented to measure load transfer from the pile cap to the top of the piles. Three piles were instrumented for load transfer along the entire pile length of 100 ft (30.5 m).

PILGP1

The development and verification of PILGP1 is based primarily on the full-scale field tests of pile group behavior under both working loads and failure conditions. Because of the many variables involved, numerous full-scale field tests need to be conducted to provide a statistically meaningful data base. However, the costs involved in full-scale field testing significantly restrict the number of tests that can be conducted. The alternatives to full-scale field testing are model field testing and laboratory model studies and centrifuge model testing.

Model testing

FHWA initiated a comprehensive investigation of scale effects between model and full-scale pile groups at the Turner-Fairbank Highway Research Center (TFHRC) in McLean, Virginia. The scaling factors identified in this study will be used to establish relationships between load deformation behavior of reduced-scale and full-scale piles and pile groups. These small-scale tests will provide much data to validate PILGP1 at less cost than full-scale field testing.

The first series of laboratory model tests are patterned after the timber pile field study at Alton, Illinois. The sandy soil at the Alton test site was matched as closely as possible at TFHRC. Model load tests will be run on single piles and pile groups at 1/20, 1/15, 1/10, and 1/3 of full scale. A minimum of three load tests will be performed for each scale. Each pile is instrumented with strain gages to measure load transfer (fig. 5).

Figure 5.—Calibration of model timber pile load cells.

The second series of laboratory model tests will be patterned after the full-scale load test on steel pipe piles in clay. Model tests will be run at 1/15, 1/10, 1/6, and 1/4 of full scale. The laboratory model tests on the 1/20, 1/15, and 1/10 scales are performed in a steel tank 5 ft (1.5 m) in diameter and 5 ft (1.5 m) deep. Because the 1/6, 1/4, and 1/3 scale models are too large to be tested in the laboratory test mold, outdoor test pits were constructed at the TFHRC site.

Centrifuge model testing

Small-scale models permit parametric studies at reasonable cost and allow soils and other conditions to be carefully controlled; however, it is difficult to achieve similitude between corresponding stresses and strains in the model and prototype. The response to load of a small pile and a large pile cannot be modeled by any simple, direct relationship derived by ordinary dimensional analysis. The question of scale effects must be resolved before any useful relationships for pile design can be developed.

Models of large heavy structures where gravity is a principal loading factor are not effective indicators of prototype behavior because the state of stress in the model caused by self weight will be much lower than in the prototype. If the model can be placed in an artificially high gravitational field, the state of stress limitation can be counteracted almost entirely. A centrifuge apparatus provides the necessary accelerated gravity rate to load test the model under simulated gravitational forces. However, to accurately measure stresses and strains, the centrifuge must be able to accommodate a model that is large enough to handle the required instrumentation. The larger centrifuge capacity provides more accuracy in direct modeling of large prototype structures.

A pilot study validated the feasibility of using centrifuge techniques for corroborating the PILGP1 mathematical model.[7] In a recently completed larger study the centrifuge was used to test models of the full-scale pile groups that were load tested to failure in the previously described field studies. The combination of centrifuge model testing and small-scale laboratory testing of conventional models at TFHRC will provide valuable physical data to establish relationships between pile groups of varying scales in the same environment.

Allowable Stresses in Piles

In addition to the ability of soil or rock to carry the load transferred from a pile, the load capacity of the pile also is important. The load capacity of a pile is governed by its structural strength and, to a lesser extent, by the surrounding environmental conditions. The structural strength is a function of the allowable stress levels that apply to the particular pile material and the cross-sectional area of the pile.

To provide a factor of safety against failure, allowable stress levels normally are specified as a percentage of the peak strength value of the pile material (for example, steel, concrete, or timber). Allowable stress levels for piles vary significantly because of different building codes in different jurisdictions. Significant controversy has arisen concerning the allowable stress levels used in the highway industry because the choice of allowable stress values greatly affects the dimensional analysis and thus the economy of the foundation system. Competition between materials producers is keen and significantly affected by changes in the allowable stress codes.

[7] "Centrifuge Model Testing of Axial Pile Behavior," University of Colorado, March 1981. Unpublished report prepared for the Federal Highway Administration.

A recently completed comprehensive investigation for FHWA of allowable stresses in piles developed recommendations for improved specifications. (*8*) A comprehensive review was made of all factors that affect the structural strength of piles subjected to axial and bending forces. Moment-thrust diagrams were developed for each kind of pile investigated, and Load Factor/Resistance Factor design concepts were used rather than the conventional factor of safety concept.

As part of the overall review, the investigators studied the codes and specifications of an array of national and foreign organizations and code bodies. The basis for each code was reviewed and corroborative data were assembled to develop improved values of allowable stresses. The claims of material suppliers for increased allowable stress levels also were evaluated and documented. Some of the claims were found to be overstated, and all of them ignored at least one or more of the important factors that govern the structural strength of a pile.

In general, it is recommended that current allowable stress values be decreased for concrete and timber piles and increased for steel piles. The new procedures for determining appropriate allowable stress levels on a case-by-case basis according to the major factors governing structural strength of piles are a significant improvement over the previously unsubstantiated blanket stress levels. Reasons for each suggested change to current methods for determining allowable stress values are documented. (*8*)

Performance of Pile Driving Systems

The high cost of pile foundations prompted FHWA to seek more efficient pile driving systems as well as more efficient design methods. Current efforts are improving concepts for measuring pile driving performance during construction. In addition to direct measurements of output, researchers will investigate the usefulness and practicality of installing gages and other instrumentation on the pile driving system to determine performance rating, spot the cause of erratic or inadequate operation, and evaluate the significance of erratic behavior in terms of performance capability.

The most promising innovative concept will be selected and implemented to verify its suitability. The prototype system will be field tested on State and Federal highway construction projects.

Lateral Loads on Piles

Several analysis and design methods for pile groups under lateral loading have been proposed, but none has been validated using carefully performed field experiments on full-scale structures. The theory of elasticity and a number of "efficiency" formulas often are used as analytical methods for lateral load design. One of the most promising methods is called the Poulos-Focht-Koch (PFK) procedure. This method will be evaluated through field load tests. Data from the load tests also may be used to develop an improved method of analysis.

The main objectives of the field load tests are to provide high quality field data on the performance of a full-scale pile group under lateral loading and to compare measured response with that predicted by the PFK method. The load test will be performed at the FHWA pile group test site at the University of Houston, Texas, campus using the same piles and some of the instrumentation used in the vertical load tests previously described.

Residual Stresses in Piles

A pile subjected to a hammer blow first will move downward, then rebound, and finally reach equilibrium after vibrating. The oscillatory nature of the pile penetration causes reversals in the direction of the pile-soil friction; the pile-soil friction resists the penetration of the pile under the hammer blow and then reverses to resist the rebound of the pile. The soil under the pile point pushes the pile back up while the pile decompresses elastically. The friction reversal reduces pile rebound and keeps the pile tip prestressed against the soil. When pile driving is complete, a significant amount of residual stress can be locked in the pile, especially in cohesionless soils.

Most pile design methods do not consider the presence of residual stresses in piles. These methods are based on load tests in which the instrumentation used to measure load transfer was zeroed after the pile was driven, thus measuring the difference between the actual stresses and the residual stresses in the piles. Analysis of pile load test data where residual stresses were measured has shown that the longer the pile, the larger the residual stresses; and, the more compressible the pile, the larger the residual stresses. The distribution of residual stresses was found to be directly related to the distribution of ultimate stresses in the piles. A new design method considering residual driving stresses was developed from the analysis of load test data.[8]

Additional research will be conducted to verify and refine the residual stress theory developed under previous research. Single piles representing a wide range of relative stiffness values will be instrumented for residual stress measurement before and after driving. Each pile will be load tested to failure, and predictions will be compared with measured behavior. The design procedures will be adjusted to incorporate residual driving stresses as warranted by the experimental analysis.

Conclusion

In the United States, pile foundations are used to support highway bridges nearly three times as often as spread footings. Because piles are much more expensive than spread footings, new and improved methods for using spread footings in lieu of piles must be developed. Also, the high cost of piles warrants intensive research to improve current design and construction procedures. The combination of reduced reliance on piles and more efficient engineering procedures when piles must be used will significantly reduce bridge construction costs.

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(*7*) G. Machan and L. R. Squier, "Bitumen Coating on Piles to Reduce Downdrag," *Public Roads,* Vol. 46, No. 4, March 1983, pp. 142–153.

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[8] J. L. Briaud, L. Tucker, R. L. Lytton, and H. M. Coyle, "Behavior of Piles and Pile Groups in Cohesionless Soils," Report No. FHWA/RD–84/007, Federal Highway Administration, Washington, D.C. Not yet published.

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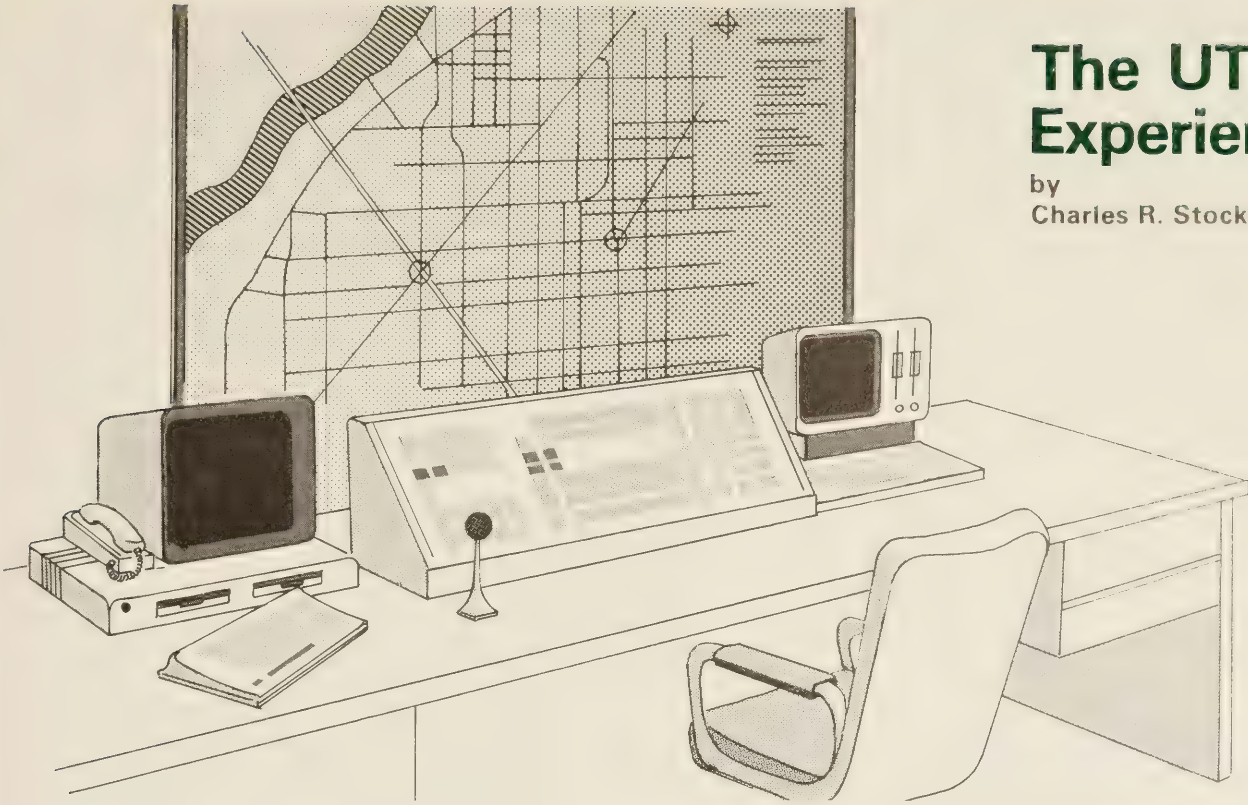

The UTCS Experience

by
Charles R. Stockfisch

The Urban Traffic Control System (UTCS), a computer-based traffic control system, has had an enormous impact on the traffic engineering community. Over the past 10 years UTCS has directly and indirectly impacted the areas of computer software and hardware and system design, installation, operation, and maintenance.

Of course, industry-developed software has had significant impacts on the traffic engineering community as well, but without the UTCS program serving as a stimulus, the state-of-the-art in computer signal control would have developed quite differently.

UTCS Development

In the early days of computerized signal systems, between 1960 and 1967, only a few cities (for example, San Jose, California; Wichita Falls, Texas; Toronto, Canada; West London, England; and Glasgow, Scotland) had operational computer-based signal systems. These cities actually established the "basic design approach" to computer signal control. By 1967 significant advances in traffic signal control equipment, communications, traffic surveillance techniques, and traffic control theory warranted development of more sophisticated traffic control systems. Because of these early accomplishments and the development of signal optimization technology such as SIGOP and TRANSYT, the Federal Highway Administration (FHWA) initiated research in 1968 to develop advanced control strategies and operational computer software.

The primary objectives of the FHWA research included the following:

- Develop and test, in the real world, new computer-based control strategies that would improve traffic flow.
- Document system planning, design, installation, operation, and maintenance to assist traffic engineers with installing their own systems.
- Stimulate modernization of traffic control equipment.

The Washington, D.C., project

FHWA's research effort resulted in the development and testing of the UTCS in Washington, D.C., between 1970 and 1976. In 1972, the system became operational with 114 intersections and 460 vehicle detectors. In 1974, the system was expanded to 200 intersections and 546 vehicle detectors.

Three levels of software sophistication were developed: First Generation Control, which uses prestored timing plans developed by offline optimization techniques; Second Generation Control, which computes online timing plans based on current traffic; and Third Generation Control, which computes online timing plans on a cycle-by-cycle basis but uses no common cycle length. Only the First Generation Control has subsequently been developed by FHWA. However, Second Generation Extended software, installed in Overland Park, Kansas, has been documented by FHWA.[1]

The First Generation software was written in assembly language to facilitate experimentation. To reduce the programing required to implement the software in other cities and on different brands of computers, FHWA converted the software to FORTRAN and demonstrated it in Washington, D.C., in 1973.

In 1975, an FHWA-sponsored conference of Federal, State, city, and industry representatives recommended that the following improvements be made in the original UTCS FORTRAN software:

- The software should operate more efficiently in state-of-the-art computers.
- The software should be modular so that the code could be more readily modified to meet special needs of individual users.
- Other functional enhancements, such as automatic restart, online data base updates, and operator interface language.

First Generation UTCS Extended and Enhanced

FHWA recognized the need to test the FORTRAN version of the software in a city that had different traffic operational characteristics than Washington, D.C., and in a system containing a state-of-the-art minicomputer. This led to the pilot installation in Charlotte, North Carolina, in 1976. Some of the changes recommended at the 1975 conference were made as part of the Extended software project in Charlotte. The software was successfully demonstrated and became operational in 1978. This resulted in the current FHWA-supported version of UTCS known as UTCS First Generation FORTRAN IV Overlay Software (Extended Version).

In further response to the 1975 conference recommendations, FHWA initiated the First Generation UTCS Enhanced project in 1977. After several delays, the Enhanced software was tested on a Honeywell computer in Broward County, Florida, in 1982, and was conditionally accepted. The documentation was given limited distribution in May 1983 as a working draft.[2] The final testing recently was completed in Birmingham, Alabama, on a Perkin-Elmer computer. The software in Birmingham controlled approximately 250 intersections and 185 detectors. Distribution of the final software and documentation is expected later this year.

Impacts From UTCS Installations

Over the years, UTCS has directly and indirectly impacted both the public and private sectors of the traffic engineering community. The knowledge gained from UTCS installations has influenced traffic engineers to install advanced computer-based signal systems and influenced consultants and manufacturers to develop advanced hardware and software products.

Operational impacts

The incentives for developing computer-controlled signal systems reach far beyond the reduction of motorist frustration caused by delay and congestion. For example:

- If $350 million were invested over the next 3 years there would be a potential annual savings of 250 million gal (946 m^3) of fuel.
- The investment would reduce carbon monoxide emissions by 600,000 tons (544 Gg) per year and hydrocarbon emissions by 1,200 tons (1 Gg) per year.

There are additional potential benefits including transportation costs for motorists using signalized networks. Potential benefits include up to a 20 percent reduction in accidents, a 5 to 50 percent reduction in vehicle stops, a 10 to 30 percent reduction in traffic delay, and a 10 to 45 percent reduction in travel time.

For a city that has 500 signalized intersections, a population of 1 million, and an average number of miles (kilometres) traveled, the cost of a $1.5 million computer-controlled signal system can be recovered in about 2 months based on fuel savings alone.

The development and dissemination of the UTCS concept has provided other benefits. For example, the introduction of these new concepts has paved the way for the acceptance of microcomputer-based traffic controllers and surveillance equipment, computer optimization modeling to compute timing plans, and computer simulation modeling to evaluate alternative strategies. Today, few traffic engi-

[1] R. W. Kessmann, "Overland Park Traffic Control System Software Documentation," FHWA contract DTFH61–82–C–00004, July 1983. Not yet published.

[2] "Applications Manual, Urban Traffic Control System," FHWA Safety and Traffic Implementation Division, HRT–20. Draft implementation package.

neers would plan a traffic control system without including some aspect of computer processing technology.

The UTCS concept also has influenced users during the evaluation process when selecting an appropriate software package. Most users specify UTCS software for one or more of the following reasons: (1) With UTCS software, a jurisdiction knows in advance what package it will get, (2) it knows it will get software that has been used before, (3) it knows it will avoid the software development process, and (4) it knows it will get the source code written in FORTRAN and all the documentation, which will allow the jurisdiction to make the desired modifications.

A number of cities have been influenced by Washington, D.C.'s, experience with communication costs and have avoided specifying leased telephone interconnect because of escalating telephone rates. When communication techniques were being evaluated for the Washington, D.C., system, time division multiplexing (TDM) was a new technology and not much was known about its reliability and cost. Because of this, frequency division multiplexing (FDM) was used. Today, TDM has become a proven technology, costs have decreased, and reliability has increased. TDM is being selected for many new installations.

In the early computer-controlled systems wall maps were the principal means of interfacing with the system. Gradually, computer terminals are taking over this function, and wall maps are best used as public relations displays. However, some users still believe wall maps are essential for troubleshooting and for checking system status.

The use of control panels also has declined for some of the same reasons. Again, some users still find them useful as training tools, but most users avoid nonflexible control panels because they are difficult to modify for special functions in the software.

The Washington, D.C., experiment also influenced detector technology significantly by promoting loop detectors instead of point detectors because loop detectors were better adapted to vehicle presence measurements needed as part of the timing plan selection algorithm.

Longitudinal and lateral placement procedures have been established to avoid selecting detector installation locations that can produce data errors. In addition, UTCS installations have provided much guidance on detector installation and testing procedures. It also has been learned that the number of detectors in a computerized system is highly variable and depends on the kind of control being implemented, characteristics of the street network, the number of critical intersections, and the number of links that characterize peaking conditions.

Soon after the UTCS project began in Washington, D.C., it became apparent that an automated method of evaluation would be needed to measure the effectiveness of the various strategies being tested. Therefore, in 1971, a traffic simulation model called UTCS−1 was developed. This model, presently known as NETSIM, was used in Washington, D.C., to evaluate detector placement, transition strategies, timing patterns, and alternative methods of granting priority to buses. When the model was developed, no one realized the impact it would have. Today there are about 130 NETSIM users in the United States and abroad. The model has been enhanced over the years to simulate microscopic vehicle movements in networks to measure improvements in fuel consumption, traffic noise, and actuated traffic signal operation. Consultants and some State highway departments (Michigan, Utah, and Kentucky) are using NETSIM to evaluate alternative street designs, shopping center designs, and traffic signal timing plans before the designs and plans actually are implemented. In addition, the NETSIM model has influenced the development of four other models in the TRAF family of simulation models. These models can simulate networks, freeways, and two-lane rural roads. NETSIM is written in ANSI FORTRAN 66 and is portable to all mainframe computers. It is one of the most powerful evaluation tools available today.

The UTCS software will interface with any kind of controller, whether it be pretimed, semiactuated, fully actuated, microprocessor-based, or eight-phase dual ring. The software also is compatible with distributed multilevel applications because of its modular design and flexibility.

Other examples of UTCS impacts can be seen in the use of the Integrated Motorist Information System (IMIS) software. The UTCS Extended software is being used in the $40 million IMIS project on Long Island, New York. The software will control traffic signals on parallel frontage roads, parallel arterials, and crossing arterials when traffic is diverted to avoid congestion on corridor freeways. The software development cost was saved because the UTCS package was already in the public domain.

The Texas State Department of Highways and Public Transportation used the UTCS documentation as a starting point to set up data base procedures, detector processing, signal sections, and intersection switching between sections for the Flexible Advanced Computer Traffic Signal (FACTS) software program. If the UTCS documentation had not been available, this would have been a very difficult task and would have taken much longer to complete.

The New York Department of Transportation is establishing a statewide UTCS users group comprised of State and local jurisdictions that own UTCS installations. Automated data processing support will be provided by the State's central computer service facility. The users group already has shown that modifications to the UTCS software can be provided at less cost than equivalent contractor services.

Nonoperational impacts

In the early days of computer-controlled signal systems, most traffic engineers did not realize that real-time process control, with its constantly changing data base and outside electrical noise interferences, was quite different from batch processing control. Neither did they understand what it took to install, operate, and maintain these systems in a city street environment. Fortunately, the UTCS development influenced a number of high technology firms and individuals to enter the traffic control field. Many of these people had no previous experience with complex traffic signal systems. However, today technical experts in private industry, universities, and local and Federal Government are working together to develop and improve software products for traffic control.

In 1974, FHWA required UTCS software be given "primary consideration" to qualify for Federal-aid funding. (*1*)[3] In the period that followed, non-UTCS software products became available that had a high probability of efficient performance. The FHWA requirement then was changed so that any software determined to be technically adequate to operate a system could qualify for Federal-aid funding as long as the bids were cost competitive.

The development and application of UTCS has helped create a market for traffic control hardware and services estimated to be in excess of $40 million annually. The demand for products and services from industry is growing as the demand increases for more advanced and efficient traffic control systems.

Another major impact of the UTCS development is in the area of training. FHWA has trained thousands of local and State traffic engineers and industry and university people. Over 100 courses have been conducted throughout the United States over the last 10 years. Typical courses have included instruction on computer control systems including computer hardware, software, communications, detectors, and controllers; feasibility studies; system installation; and management. Other courses have included instruction on traffic optimization and simulation models—essential for timing plan generation and evaluation.

Also impacted has been the traffic engineer's vocabulary. Terms, such as "section," "critical intersection control," "traffic responsive system," and "systems manager," have evolved with special meanings relating to computer signal control.

FHWA has produced and distributed thousands of copies of operators and maintenance manuals, system descriptions, software documentation, computer tapes, and specifications to States, cities, consultants, manufacturers, and universities in the United States and abroad. This material has had far reaching effects on the installation of computer-based traffic signal systems. The level of knowledge has increased significantly and will continue to increase as new material becomes available.

Another major impact is in the area of procurement. Current FHWA regulations require that traffic surveillance and control system projects be based on a traffic engineering analysis on a scale commensurate with the project scope. (*2*) For example, the traffic engineering analysis includes requirements for the following:

- Defining the existing system and resources.
- Incremental costs for alternative systems.
- Consideration of various procurement and start-up methods.
- The need for special control functions.
- Consideration of existing laws.
- Development of an operations plan including provisions for construction management, acceptance testing, system operation and maintenance, and institutional agreements.

[3]Italic numbers in parentheses identify references on page 29.

Conclusion

Over 40 cities now have, or soon will have, some form of UTCS or UTCS-based system with 30 or more intersections. In those cities where UTCS has been installed, traffic flow and operational efficiency have improved and fuel consumption and air pollution have been reduced. In most cities, savings to motorists have exceeded the original investments within a few months of operation.

UTCS has achieved its original objectives and has filled an important national need. However, more work is needed in computer traffic control to reduce capital and operating costs, make better use of microcomputers, achieve greater flexibility and portability of the software, make better use of traffic-responsive control as an operational mode, and improve the availability of documentation and training.

Looking back at the impacts of the UTCS Experience it is apparent that UTCS was a worthwhile undertaking and that additional development is warranted. However, because of declining funds available for urban traffic control projects and an apparent lack of a constituency for new development in this area, FHWA's future role probably will be concentrated on testing and evaluation of existing software and software support and maintenance rather than on new development of online programs.

Acknowledgment

The author would like to acknowledge the assistance of traffic engineers in the cities of Fort Lauderdale, Florida; Clearwater, Florida; New Orleans, Louisiana; and Washington, D.C.; assistance from the Texas State Department of Highways and Public Transportation and the Florida Department of Transportation; assistance from the traffic engineer in Dade County, Florida, and assistance from FHWA staff. Also thanked are representatives of Sperry Corporation, Honeywell, Inc., JHK & Associates, Kessmann & Associates, KLD Associates, Inc., PRC Voorhees, and PAWA-Winkelmann Associates.

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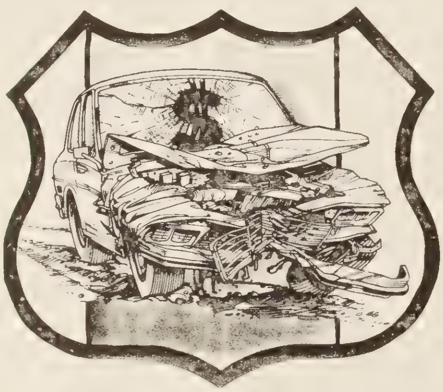
Recent Research Reports You Should Know About

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

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

Alternative Approaches to Accident Cost Concepts, Executive Summary and State-of-the-Art Report, Report Nos. FHWA/RD–83/078–079

by Safety and Design Division



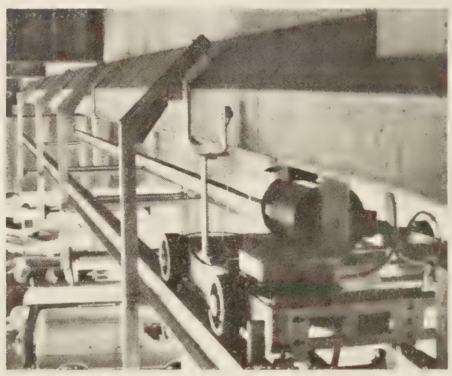
Highway accident cost derivation has been a source of controversy for many years. Not only has the appropriateness of various categorical elements (such as fire and police costs) been debated, but also the amount of detail and appropriate procedures for deriving particular elements.

These reports document the state-of-the-art in accident cost methodology, critique prominent schools of thought, provide two study plans for determining state-of-the-art accident costs (one based on the capital-cost approach and the other on the willingness-to-pay approach), and make recommendations for future research needs.

The reports may be purchased from NTIS.

Signal Enhancement and Interpretation for the Detection of Flaws in Reinforcing Steel in Prestressed Concrete Bridge Members, Report No. FHWA/RD–83/081

by Structures Division



The possibility of failure of in-service prestressed concrete bridge members from deterioration of the high strength prestressing steel led to the development of a prototype instrumentation system for rapid, in situ inspections of steel in prestressed concrete bridges using a magnetic field disturbance approach. This report summarizes recent progress in signal enhancement that has significantly improved detection capability. Typical records and results obtained during laboratory and field evaluations are discussed.

The report may be purchased from NTIS.

Design and Construction of Low Water Stream Crossings, Executive Summary, Report No. FHWA/RD—83/015

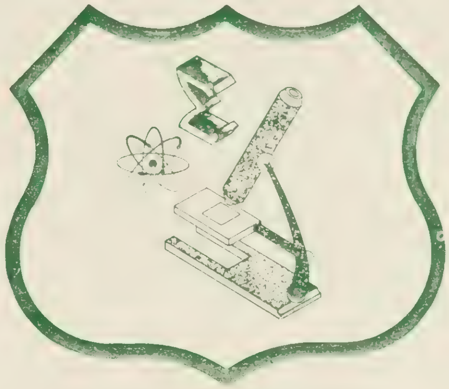
by Pavement Division

This report summarizes the results of a 2-year investigation of commonly used low water stream crossings. The objective of the study was to gather information that would aid county and local engineers in selecting appropriate low water stream crossing structures. Different kinds of commonly used low water stream crossings are described and the characteristics of their efficient designs are discussed. Selection factors are listed, as are a set of general and specific design considerations. A method based on economic risk analysis is recommended for the final selection and design of fords, vented fords, and low water bridges. Design examples are provided for each of these three structures.

The report may be purchased from NTIS.

Time-to-Corrosion of Reinforcing Steel in Concrete Slabs, Volume 5, Report No. FHWA/RD—83/012

by Materials Technology and Chemistry Division



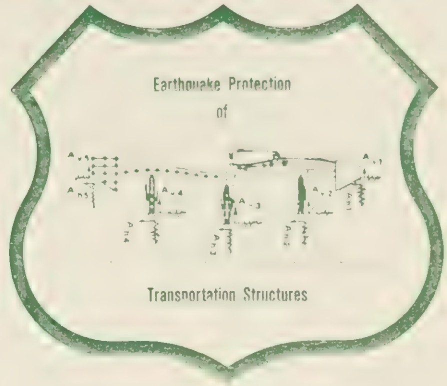
This report presents the findings of an outdoor exposure study of concrete slabs containing either epoxy-coated rebars or calcium nitrite as a corrosion-inhibiting protective system. The tests were performed under conditions that simulated those found in typical highway bridge decks, and the results are compared with those obtained on uncoated reinforcing steel.

Findings of the study indicate that both epoxy-coated reinforcing steel and calcium nitrite reduce the corrosion rate more than an order of magnitude, and thus should provide long term protection against corrosion-induced damage on properly engineered and constructed structures in severe salt environments. Some of the variables that affected the performance of the slabs were the chloride content in the calcium nitrite experiment, and the selective coating of the upper mat only, versus the coating of both mats in the coated rebar experiment.

The report may be purchased from NTIS.

Seismic Retrofitting Guidelines for Highway Bridges, Report No. FHWA/RD—83/007

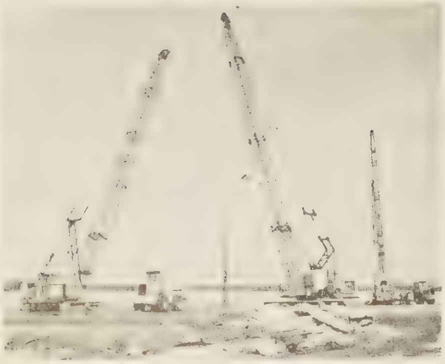
by Structures Division



This report contains guidelines for the seismic retrofitting of typical highway bridges in all parts of the United States. The guidelines are based on both the observed performance of bridges during earthquakes and on recent research conducted in the United States and in other countries.

The comprehensive guidelines include a preliminary screening procedure to identify candidate bridges for retrofitting, methods for evaluating an existing bridge in detail (calculating seismic capacity/demand ratios for each potentially vulnerable bridge component, assessing the consequences of a design earthquake at the bridge site, and selecting the most appropriate retrofitting scheme for the bridge), and potential retrofitting measures for the most common seismic deficiencies. An extensive commentary documenting the basis for the guidelines and an example problem illustrating their use are included in the report.

The report may be purchased from NTIS.



Design and Construction of Stone Columns, Volume I, Report No. FHWA/RD–83/026

by Construction, Maintenance, and Environmental Design Division

Stone columns have been used since the 1950's to stabilize cohesive soils and silty sands. This report presents the results of a comprehensive investigation of the use of stone columns to support structural foundations and to correct unstable slopes. Design and construction guidelines for using stone columns as a ground improvement technique are presented along with selected case history examples.

The report may be purchased from NTIS.

A Model of Visual Complexity of Highway Scenes, Report No. FHWA/RD–83/083

by Systems Technology Division

This report describes a study to quantify the visual complexity of night driving scenes. Five laboratory studies (using photographic slides) and one field study were conducted to investigate the visual difficulty drivers have in interpreting nighttime scenes. The data collected included measures of subjective judgment, temporal and eye-movement aspects of scene viewing, and instrumented vehicle data on vehicle dynamics and driver behavior.

A simplified mathematical model of global scene "instability" was developed and evaluated as an objective measure of scene complexity. The model analysis is based on the gray-scale values of digitized scene images. The model should be useful in identifying sections of highway where visual complexity is a problem and in predicting the relative effectiveness of various delineation treatments in providing improved guidance information.

The report may be purchased from NTIS.

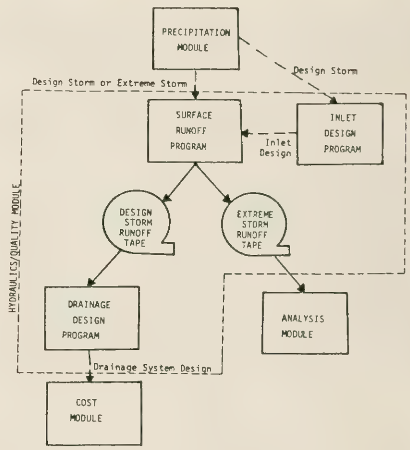
Urban Highway Storm Drainage Model, Executive Summary and Volumes 1–7, Report Nos. FHWA/RD–83/040–047

by Structures Division

These reports describe a study that developed an urban highway storm drainage model to evaluate existing drainage systems and to aid in designing new drainage systems. The model includes a package of six user-oriented computer programs that were developed and tested for the analysis and design of urban highway drainage systems and related nonpoint source pollution problems. The programs are organized into four related but independent modules.

The **Executive Summary** presents the major findings of the study and describes the computer programs that were developed and their uses.

Volume 1, **Model Development and Test Applications,** documents the development of the programs, the history of the project, an overview of the model, and the results of three test runs of the model on various proposed and developed urban highways throughout the United States.



Volume 2, **Precipitation Module,** contains the documentation and users manual for the precipitation module, which can be used to statistically analyze hourly precipitation data to generate local design storm hyetographs of selected duration.

Volume 3, **Inlet Design Program,** contains the documentation and users manual for the inlet design program of the hydraulics/quality module. User-specified design criteria are used to determine the location of fixed-sized inlets in the surface runoff conveyance system.

Volume 4, **Surface Runoff Program,** contains the documentation and users manual for the surface runoff program of the hydraulics/quality module. The program simulates the quantity and quality of runoff from the highway and its surroundings and routes the flow through the surface conveyance system to detention basins and inlets to the underground conduit system.

Volume 5, **Drainage Design Program,** contains the documentation and users manual for the drainage design program of the hydraulics/quality module. Based on the inlet hydrographs and the inlet pollutographs computed by the surface runoff program, the drainage design program sizes the storm sewers and estimates the storm water quantity and quality at the sewer outfalls.

Volume 6, **Analysis Module,** contains the documentation and users manual for the analysis module, which simulates unsteady, gradually varied flow in the drainage system using inlet hydrographs from the surface runoff program as input. Special conditions that can be simulated include surcharge, backwater, and pumping station operation.

Volume 7, **Cost Estimation Module,** contains the documentation and users manual for the cost estimation module, which is used to estimate construction, operation, and maintenance costs associated with the drainage system. The program is most useful in comparing the costs of alternative drainage system designs.

The reports may be purchased from NTIS.

SPEL

Special Product Evaluation List

SPEL: Special Product Evaluation List, Report No. FHWA/RD–83/093

by Construction, Maintenance, and Environmental Design Division

This report lists special products evaluated by State highway and transportation departments to provide information on who tested the products or accepted the material. Results are presented on 4,244 evaluations by 38 States and FHWA from December 1977 through December 1983.

The report may be purchased from NTIS.

EAROMAR Version 2, Report Nos. FHWA/RD–82/085–088

by Pavement Division

Highway administrators and engineers must select among alternative pavement investment and maintenance strategies. The selections should be based upon an economic analysis of each pavement management strategy. Economic Analysis of Roadway Occupancy for Maintenance and Rehabilitation (EAROMAR) is a computerized system to help evaluate the economic impacts of alternative pavement design and maintenance strategies. The system EAROMAR simulates structural performance and freeway operation to predict life-cycle roadway costs. These costs include highway agency expenditures and user costs.

The **Executive Summary** (RD–82/085) describes the technical factors considered in the EAROMAR system. The **Final Technical Report** (RD–82/086) gives the relationships built into EAROMAR to relate repair requirements to pavement damage. The **Users Manual** (RD–82/087) describes how to prepare data and submit runs for problem analysis. The **Program Documentation** (RD–82/088) details the structure and operation of the EAROMAR system.

The reports may be purchased from NTIS.

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

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

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

FHWA Research, Development, and Technology Implementation Catalog

by Office of Implementation

This catalog, which is revised periodically, lists selected publications, visual aids, computer programs, and training materials that are available as a part of the FHWA implementation program. Items are listed alphabetically under program areas. Subtitles and series are shown separately under the main item heading. Items are available directly from the source indicated under the "Availability" heading in each listing. Some items are available without charge to qualified individuals and agencies; others are available on a loan basis only. Indexes, at the back of the catalog, are arranged alphabetically, by program area, and by report number.

Copies of the catalog are available from the Federal Highway Administration, HRD–11, 6300 Georgetown Pike, McLean, Virginia 22101.

Federal Highway Administration

FHWA

Research, Development, and Technology

Implementation Catalog

January 1984

Field Evaluation of a Generic Thermoplastic Pavement Marking Material, Report No. FHWA–TS–83–201

by Office of Implementation

A generic formulation of a hydrocarbon-based thermoplastic was field tested in three States. Overall, the thermoplastic-primer systems performed well except for one portland cement concrete test section.

Initial nighttime visibility was influenced greatly by the concentration of surface beads. The specified density of 1 lb (0.45 kg) of beads per 100 ft (30.5 m) of line gave poor nighttime visibility. Not until the bead concentration was increased to 1.8 lb (0.82 kg) per 100 ft (30.5 m) did nighttime visibility become good. No distinctive difference exists between the glass beads with 1.65 refractive index and the beads with 1.50 refractive index.

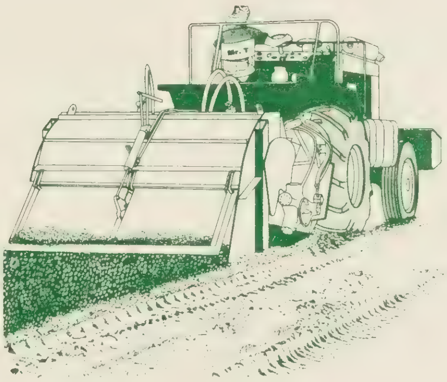
The report may be purchased from NTIS.

Soil Stabilization in Pavement Structures, Volumes 1 and 2, Report No. FHWA–IP–80–2

by Office of Implementation

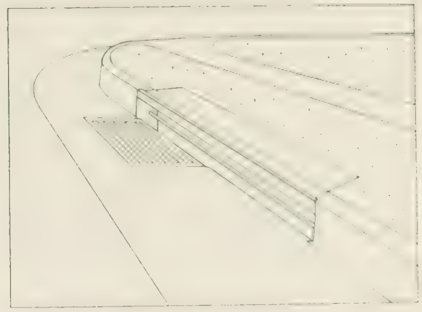
This two-volume report, which documents soil stabilization operations associated with transportation systems, has been reprinted.

Volume 1, **Pavement Design and Construction Considerations,** was prepared for pavement design and construction engineers and describes a method for selecting the kind of stabilizers as well as pavement thickness design methods and construction information. Quality control, guide specifications, and cost and energy considerations are contained in the appendixes.



Volume 2, **Mixture Design Considerations,** was prepared for materials engineers and describes methods for selecting the kind and amount of stabilizers. Methods of estimating stabilizer contents are presented as are detailed test methods, mixture design criteria, and typical mixture criteria.

The reports may be purchased from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Vol. 1, $7, Stock No. 050–001–00160–8, and Vol. 2, $6.50, Stock No. 050–001–00161–6).



Drainage of Highway Pavements, Report No. FHWA–TS–84–202

by Office of Implementation

Wider roadways, flatter slopes, and an increasing emphasis on highway safety have influenced methods of removing storm water from highway pavements. To address these changing design conditions and safety concerns, Hydraulic Engineering Circular No. 12, "Drainage of Highway Pavements," has been revised and expanded. The updated circular provides the current design criteria for determining storm water runoff, gutter flow, and inlet interception as they relate to the roadway geometry of various kinds of highway pavements. This report also addresses safety aspects associated with inlets and the water spread on pavements.

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

New Research in Progress

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

FCP Category 1—Highway Design and Operation for Safety

FCP Project 1P: Night Visibility

Title: Evaluation of Alternative Light Sources for Guide Sign Illumination. (FCP No. 41P3214)
Objective: Select, test, and evaluate a series of power-efficient luminaires for use with overhead highway guide signs. Evaluate luminaires for their production of adequate and not-excessive light, uniformity, color, reliability, ease of installation and cleaning, longevity, and power consumption.
Performing Organization: Arizona State University, Tempe, Ariz. 85287
Funding Agency: Arizona Department of Transportation
Expected Completion Date: September 1985
Estimated Cost: $44,000 (HP&R)

FCP Project 1T: Roadside Safety Hardware

Title: Bridge Rail Designs and Performance Standards. (FCP No. 31T2342)
Objective: Evaluate existing bridge railing designs through full-scale tests and static and pendulum tests of components. If necessary, redesign the bridge rails to improve their geometry, strength, and performance. Reevaluate current performance standards with regard to bridge railing geometrics. Develop a relationship between the frontal areas of the railing, the vertical opening, and the post setback that will eliminate small car snagging.
Performing Organization: Southwest Research Institute, San Antonio, Tex. 78284
Expected Completion Date: June 1985
Estimated Cost: $428,400 (FHWA Administrative Contract)

FCP Project 1W: Vehicle/Surface Interaction Problems

Title: An Investigation of Friction Life Cycles for Indiana Pavements. (FCP No. 41W1282)
Objective: Reevaluate the current method of checking the calibration of the friction testing equipment used by the Indiana Department of Highways. Develop procedures for adjusting data according to daily variations in the calibration check results. Collect additional data for use in further developing the equations for predicting pavements' seasonal variation in friction levels and their overall friction life.
Performing Organization: Indiana Department of Highways, Indianapolis, Ind. 46204
Expected Completion Date: June 1988
Estimated Cost: $13,100 (HP&R)

FCP Category 2—Traffic Control and Management

FCP Project 2L: Electronic Devices for Traffic Control

Title: Improved Techniques for Sensing Vehicle Axles. (FCP No. 22L1122)
Objective: Conduct a study to improve current methods for sensing the passage of vehicle axles over a buried sensor. Conduct a physics-oriented analysis to determine basic vehicle properties and which kinds of sensors are suitable for permanent installation. Develop a breadboard model of the best sensor and conduct limited field tests.
Performing Organization: Federal Highway Administration, Washington, D.C. 20590
Expected Completion Date: March 1985
Estimated Cost: $20,000 (FHWA Staff Research)

Title: Wide Area Detection System (WADS)—Applications. (FCP No. 32L1182)
Objective: Identify, with user inputs, the priority of WADS applications (for example, incident detection, traffic control, and various traffic studies). Develop conceptual design specifications for priority WADS applications, determine common elements, and develop a plan for future preparation of software packages at minimum cost and duplication of effort.
Performing Organization: Sperry Corporation, Great Neck, N.Y. 11020
Expected Completion Date: June 1986
Estimated Cost: $245,560 (FHWA Administrative Contract)

FCP Category 3—Highway Operations

FCP Project 3A: Maintenance Management

Title: Nonmetric Cameras in Highway Maintenance and Safety. (FCP No. 43A1803)
Objective: Develop the necessary equipment, computer programs, and procedures for using nonmetric cameras in highway mapping applications to provide an economical and rapid method of obtaining cross sections, slope mapping, intersection mapping, deformation measurements, and stockpile volumes.
Performing Organization: West Virginia University, Morgantown, W. Va. 26506
Funding Agency: West Virginia Department of Highways
Expected Completion Date: May 1985
Estimated Cost: $60,390 (HP&R)

Title: Global Positioning System (GPS) Exploitation by Ohio Department of Transportation. (FCP No. 43A1813)
Objective: Develop and demonstrate global position system to control survey tasks associated with planning, designing, constructing, and maintaining transportation systems.
Performing Organization: Ohio State University, Columbus, Ohio 43210
Funding Agency: Ohio Department of Transportation
Expected Completion Date: February 1986
Estimated Cost: $56,080 (HP&R)

FCP Category 4—Pavement Design, Construction, and Management

FCP Project 4A: Pavement Management Strategies

Title: The Influence of Heavy Truck Design and Operation on the Dynamic Loading of Pavement. (FCP No. 34A1011)
Objective: Develop improved single axle load equivalencies that are compatible with current truck axle configurations, kinds of suspension, kinds of tires, and pavement conditions.
Performing Organization: Research and Special Programs Administration, Washington, D.C. 20590
Expected Completion Date: November 1986
Estimated Cost: $450,000 (FHWA Administrative Contract)

Title: Pavement Selection Based on Life Cycle Cost. (FCP No. 44A2092)
Objective: Collect, document, and summarize data pertaining to the performance of four kinds of pavement (full-depth asphalt mix, asphalt mix on an aggregate base, continuously reinforced concrete, and jointed reinforced concrete) in Mississippi to determine life cycle pavement costs.
Performing Organization: Mississippi State Highway Department, Jackson, Miss. 39205
Expected Completion Date: December 1984
Estimated Cost: $16,000 (HP&R)

FCP Project 4B: Design and Rehabilitation of Rigid Pavements

Title: Condition Monitoring of Continuously Reinforced Concrete Pavements. (FCP No. 44B2124)
Objective: Measure and analyze the kinds and rates of deterioration occurring in continuously reinforced concrete pavements in Oregon. Determine the probable causes of deterioration and recommend design, construction, and maintenance improvements.
Performing Organization: Oregon Department of Transportation, Salem, Oreg. 97310
Expected Completion Date: January 1987
Estimated Cost: $13,000 (HP&R)

FCP Project 4C: Design and Rehabilitation of Flexible Pavements

Title: Material Characterization of Hot-Mix Recycled Bituminous Pavements. (FCP No. 44C3104)
Objective: Evaluate recycled materials to determine the effect of weathering, the effectiveness of mixing on the rate of dispersion and distribution of recycling agent, and physical properties such as Hveem, Marshall, resilient modulus, and fatigue.
Performing Organization: Indiana Department of Highways, Indianapolis, Ind. 46204
Expected Completion Date: December 1986
Estimated Cost: $32,100 (HP&R)

Title: A Field and Laboratory Study of Cold-Mix Recycling in Ohio. (FCP No. 44C3114)
Objective: Select and test cold-mix recycling construction projects to obtain material and structural pavement condition ratings before construction, develop mix design recommendations, monitor the cold-mix recycled pavements, and develop an implementation guideline for cold-mix recycling.
Performing Organization: Resource International, Inc., Worthington, Ohio 43085
Funding Agency: Ohio Department of Transportation
Expected Completion Date: December 1985
Estimated Cost: $34,000 (HP&R)

Title: Introduction of Lime Into Asphalt Concrete Mixtures. (FCP No. 34C4043)
Objective: Conduct a survey of current methods used by the construction industry for adding lime to an asphalt mixture. Determine the best methods for adding both dry lime and lime slurry to batch and drum-mix plants. Test aggregates from four sources in the laboratory and field. Perform immersion-compression and Lottman tests on the laboratory specimens and nine field cores taken from test sites.
Performing Organization: Chicago Testing Laboratory, Inc., Northbrook, Ill. 60062
Expected Completion Date: November 1985
Estimated Cost: $261,600 (FHWA Administrative Contract)

Title: Improved Construction Joints for Flexible Pavements. (FCP No. 44C6134)
Objective: Construct two experimental field projects—one with a bottom and top course and one a resurfacing—with different variables related to a longitudinal construction joint. Compare a beveled edge with a vertical edge. Measure density and roughness.
Performing Organization: New Jersey Department of Transportation, Trenton, N.J. 08625
Expected Completion Date: October 1987
Estimated Cost: $17,450 (HP&R)

Title: Development of Guidelines for Rehabilitating Flexible Pavements. (FCP No. 44C6144)
Objective: Match rehabilitation strategies with various kinds and levels of distress. Obtain estimates of the extended service life provided by various strategies. Consider costs to provide input into a pavement management system to select the optimum rehabilitation technique or resurfacing option to be used.
Performing Organization: Virginia Highway and Transportation Research Council, Charlottesville, Va. 22903
Funding Agency: Virginia Department of Highways and Transportation
Expected Completion Date: April 1985
Estimated Cost: $25,500 (HP&R)

FCP Category 5—Structural Design and Hydraulics

FCP Project 5A: Bridge Loading and Design Criteria

Title: Development of a Cost-Efficient Computer Analysis and Program for Horizontally Curved Box Girder Bridges. (FCP No. 45A3542)
Objective: Simplify the analysis and develop a cost-efficient computer program for horizontally curved box girder bridges.
Performing Organization: Mississippi State University, State College, Miss. 39762
Funding Agency: Mississippi State Highway Department
Expected Completion Date: December 1984
Estimated Cost: $42,000 (HP&R)

FCP Project 5K: Bridge Rehabilitation Technology

Title: Fatigue Strength of Weathered and Deteriorated Riveted Members. (FCP No. 35K2122)
Objective: Conduct pilot studies on the fatigue behavior and resistance of weathered and deteriorated full-scale riveted built-up members to determine if there is a difference in fatigue resistance for different riveted details. Use stringers from actual bridges.
Performing Organization: Lehigh University, Bethlehem, Pa. 18015
Expected Completion Date: January 1985
Estimated Cost: $150,000 (FHWA Administrative Contract)

FCP Project 5Q: Bridge Maintenance and Corrosion Protection

Title:Cathodic Protection of Bridge Decks Using Sprayed Metal and Conductive Paint Anodes. (FCP No. 45Q2502)
Objective: Evaluate conductive paint as an anode, develop a procedure for spraying zinc metal to a concrete surface, apply cathodic protection on bridge decks, and determine current density for protecting reinforcing steel in concrete.
Performing Organization: California Department of Transportation, Sacramento, Calif. 95807
Expected Completion Date: July 1986
Estimated Cost: $59,000 (HP&R)

New Publication

FHWA's Offices of Research, Development, and Technology (RD&T) have released their fiscal year **1983 Annual Report on the Federally Coordinated Program (FCP) of Highway Research, Development, and Technology.**

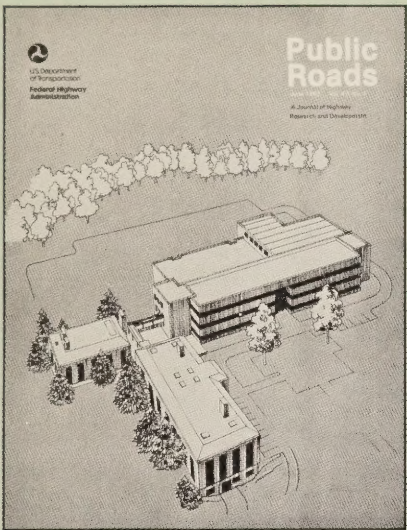
This report briefly describes the goals and programs of the FCP and FCP accomplishments in highway RD&T during FY 1983.

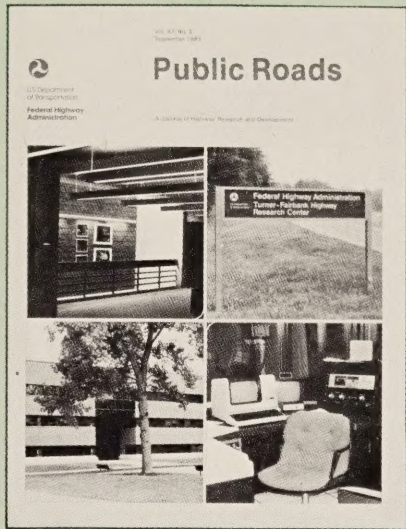
Also cited are specific accomplishments in the recently reorganized FCP categories of Highway Design and Operation for Safety; Traffic Control and Management; Highway Operations; Pavement Design, Construction, and Management; and Structural Design and Hydraulics. The new laboratories and the renovated laboratories at the Turner-Fairbank Highway Research Center in McLean, Virginia, are briefly described. The report is prefaced by a message from Federal Highway Administrator R. A. Barnhart.

While supplies last, individual copies of the report are available without charge from the Federal Highway Administration, Offices of RD&T, HRD–10, 6300 Georgetown Pike, McLean, Virginia 22101.

☆U.S. Government Printing Office: 1984—421–783/303

TITLE SHEET, VOLUME 47





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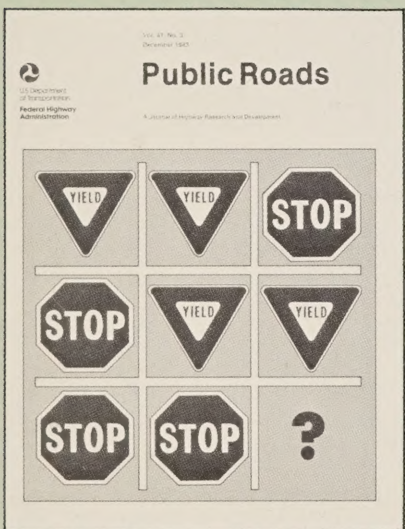

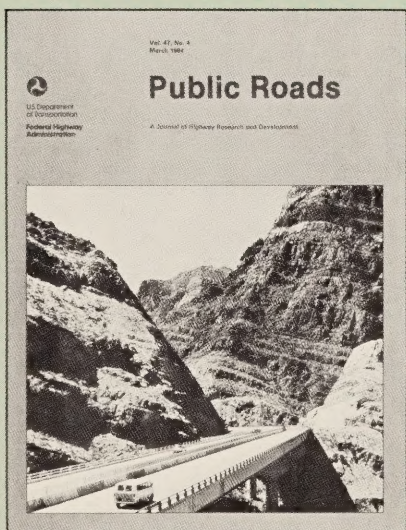

VOLUME 47

U.S. Department of Transportation
Federal Highway Administration

June 1983–March 1984

The title sheet for volume 47, June 1983–March 1984, of *Public Roads, A Journal of Highway Research and Development,* is now available. This sheet contains a chronological list of article titles and an alphabetical list of authors' names. Copies of this title sheet can be obtained by sending a request to *Public Roads,* Federal Highway Administration, HRD–10, 6300 Georgetown Pike, McLean, Virginia 22101.

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