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COVER:
Artist's concept of bicycle path paralleling the highway.

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Solving the Mysteries of Pavement Deflections With "Thumper"

by
T. F. McMahon and R. W. May

The responses of pavements to wheel loads have been studied for many years. Measurements of pavement deflections under controlled loadings are made with many devices. The Federal Highway Administration (FHWA) has developed improved field equipment for research on pavement deflection responses. "Thumper," a new automated, mobile system developed by FHWA, applies static or dynamic loadings to pavement and records the resulting responses. FHWA-State cooperative studies using this system should advance the methodology for pavement structural evaluation.

Introduction

Pavement deflections under controlled loading have been measured for years and have been used to evaluate inservice pavements and to design overlays. The numerous measuring methods and the disparity of the measurements have led to confusion and some reluctance in accepting deflections as a standard method of pavement evaluation.

FHWA's Office of Research is attempting to establish standard methodology for evaluation of pavements and design of overlays in a study, "Structural Rehabilitation of Pavement Systems." A dynamic deflection test system was developed and constructed. It can duplicate measurements made by nearly all existing deflection devices; in addition, it can make static loading and associated deflection measurements that other devices cannot.

After the pavement testing system was constructed, it was evaluated and analyzed; comments and recommendations were made on its performance and use.

"Thumper," the FHWA dynamic deflection test system, is the product of years of experience in pavement testing and evaluation (fig. 1). Its features promise the most usable information possible from a nondestructive testing scheme. Like most of the other current testing devices, it can perform dynamic load measurements over a wide range of frequency and load levels on any pavement surface. However, "Thumper" goes one step further. It can apply a static load to a pavement system and measure the shape of the resulting deflection basin, and more importantly, changes in the deflection basin as a function of time. This feature, coupled with a viscoelastic analysis, provides another analytical tool for understanding pavement failure and optimizing design and maintenance procedures.

"Thumper" was created to explore in detail the capabilities of various concepts of dynamics, static loading, and creep loading to extract significant data from an existing pavement for either reconstruction or estimation of remaining life expectancy. This research tool is expected to help to correctly evaluate data from existing testing devices and indicate the new capabilities that provide the most cost effecitve improvement in confidence levels.

The "Thumper" System

"Thumper" is fully automated and, after initial adjustments to the system, is operable by one person from a console adjacent to the driver's seat. The "Thumper" incorporates several separate units: (1) The vehicle, (2) the reference frame for the linear variable

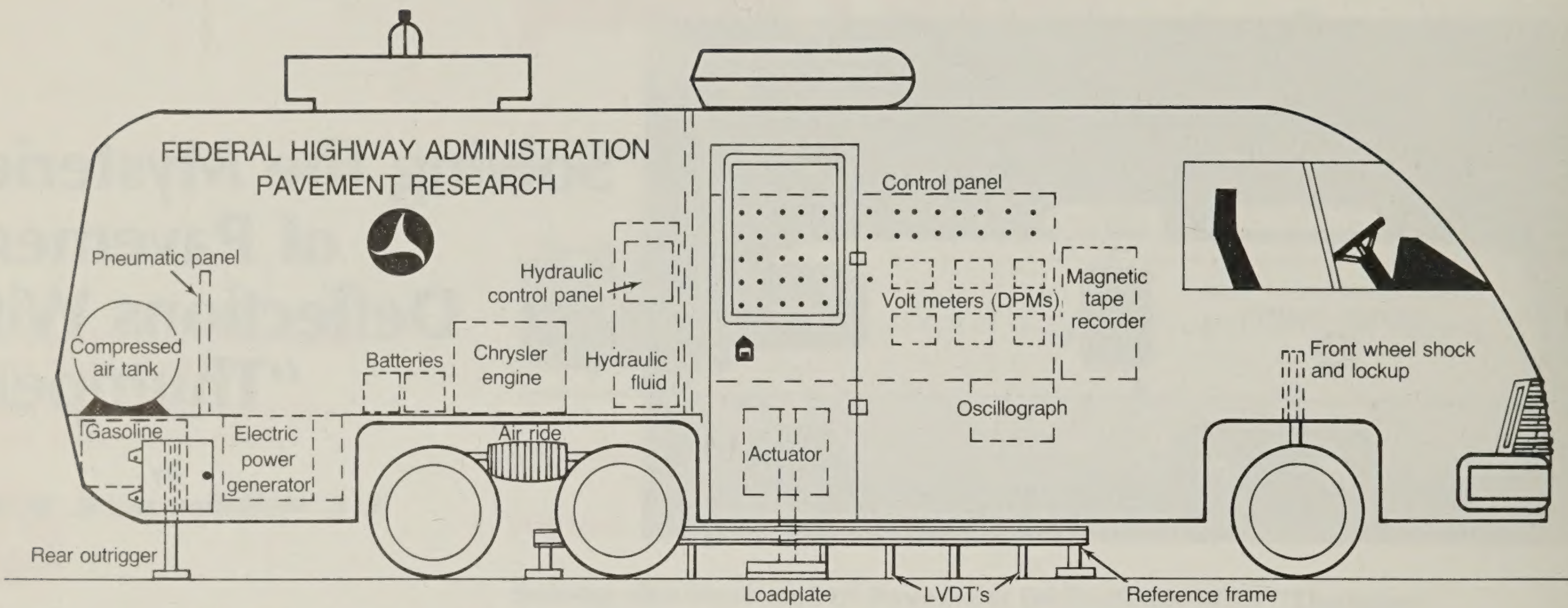


Figure 1.—FHWA pavement deflection test system.

differential transformer (LVDT) deflection measurement assemblies (fig. 2), (3) the hydraulic system, (4) the pneumatic system, and (5) the electronics system.

Vehicle

The vehicle chosen to house the mobile, self-contained pavement deflection measuring system was a 1976 transmode van with a 4 m wheelbase. Several factors made this vehicle attractive for this testing system: (1) The frontwheel drive train makes the reference frame under the chassis easy to adapt for the LVDT assemblies because there is no drive shaft or rear differential in the vehicle, (2) the vehicle can handle the gross weight and space required for the testing apparatus, (3) the extra air-conditioning required for instrumentation could be easily incorporated into the van type unit, and (4) the vehicle's level ride control was favorable for easy disabling of the rear suspension while operating in the test mode.

Reference frame

The reference frame for the six LVDT's is constructed so that when it is lowered hydraulically, it becomes a completely free-floating detached unit and is not influenced by the vehicle or load vibrations. The reference frame is also constructed so that it has a very low natural frequency. The three support feet were designed to support the frame load on asphalt surfaces without sinking into the surface.

Hydraulic system

A gasoline engine driven hydraulic pump mounted in the rear of the test van is used to supply oil under pressure to the servo-operated hydraulic actuator which creates the pavement loadings up to 40 kN in a frequency range of 0.1 to 110 Hz. The closed loop system with a load cell supplying a feedback signal to the servo-controller provides continuous monitoring of the applied load with some limitations being imposed by the vehicle resonance. To hold the vehicle resonance to a minimum, hydraulic cylinders are used to lock the spring action of the suspension. In the rear of the vehicle, hydraulic cylinders are deployed to lift the rea wheels of the vehicle while the rear air suspension bags are deflated, providing a stabilized platform from which to conduct the tests. Hydrauli power is also used to actuate two cylinders which lower the referenc frame to the test surface during the test mode.

Pneumatic system

A compressor, driven by the same gasoline engine that drives the hydraulic pump, provides air for th pneumatic system. Solenoid operated 110-volt valves, commanded by the electronics portion of the van, shuttle the air supply during the different sequences of the test mode to perform the operation needed.

The pneumatic system plays an important role in the zeroing, cleaning, and lubricating of the LVT assemblies when the reference fra e is lowered during the test mode t take deflection readings. After the reference beam becomes stable on the reference surface, pneumatic brakes lock the LVDT's in position and the positioning cylinder is retracted leaving the LVDT's to take the deflection measurements. During this sequence of operation, oil-impregnated air is circulated through the linear ball bearing in the LVDT assembly, LVDT shafts, and positioning cylinder to keep the shafts free of rust and road film.

During the tests, the rear air suspension system of the test vehicle is also controlled by the pneumatic system. The control valves first deflate the air suspension system to form the stable platform for the test and then reinflate the air suspension system to make the vehicle roadworthy again. When the system is not in the test mode condition, the rear air suspension system is switched back to the vehicle's level-ride air supply system by the control panel.

Electronics system

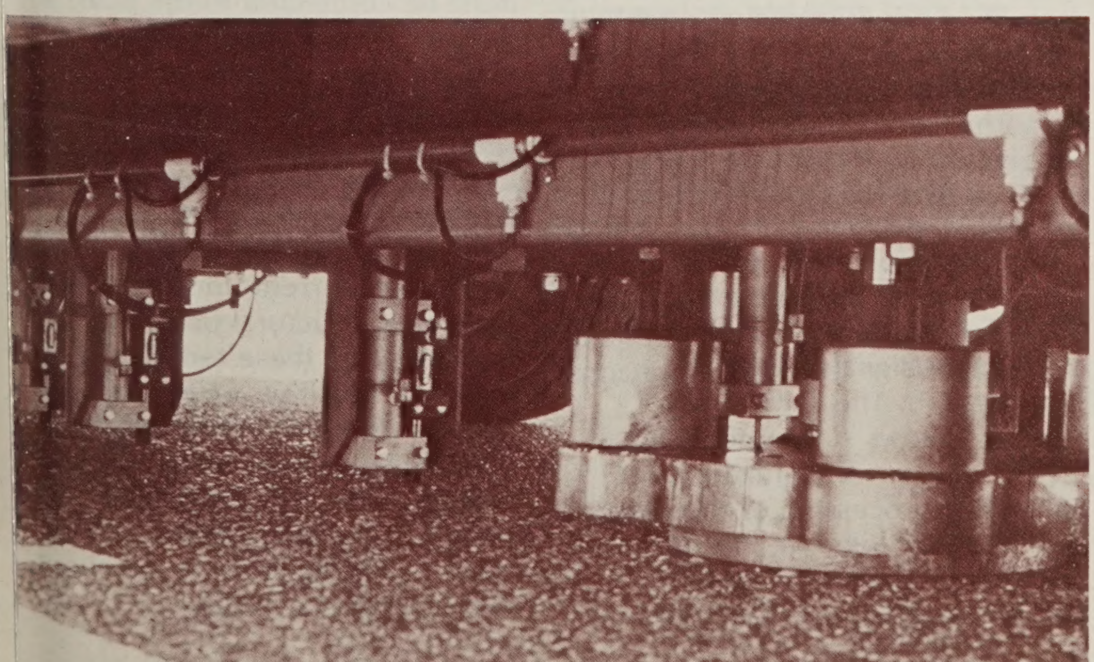
The electronics system of the "Thumper" is comprised of two interrelated subsystems which are responsible for control and measurement respectively.

Control subsystem

The function of this subsystem, operated from the master control panel (fig. 3), is to precisely control the various actuators in a proper sequence for a test. The sequence timer is the unit that sends out commands to the actuators, hydraulic or pneumatic, to extend during the pretest period. This sequence will proceed from step to step and stop just prior to executing a test function if the program switch is in the manual position. In this condition the vehicle's rear jack pads are extended, the front suspension is stiffened, the main reference beam is lowered to the test surface, and the LVDT reference pins are retracted. If the program switch were in the automatic position, the sequencer unit would command the ram controller to put preselected loads into the test surface.

Test loads are controlled by the ramp generator and function generator.

Figure 2.—Linear variable differential transformer deflection measurement assemblies.



The ramp generator gives command signals to the load ram that vary in both slope and maximum amplitude, as selected by the operator. These levels may also be stair-stepped with accurate hold times for creep type tests. These various modes of operation are selected by the program switch on the panel. The ramp generator also provides a controlled start and stop when used with the function generator. A function is superimposed on the plateau of the ramp signal after a gentle rise in ram force followed by a slow unloading at the end of a test. The function generator produces command signals for various periods and amplitudes of loading. Circuits are also incorporated in the function generator to allow a prescribed number of cycles to be skipped between loadings. This capability may be used with either a full sine wave or half sine wave, as would be generated by a vehicle wheel. To facilitate full automatic operation, a cycle counter is incorporated along with manual numeric entry capability. When the counter is equal to the preset number, the system will shift into an end-of-test procedure which will be discussed later. Another command signal sent to the load ram is a continuous preload entered via thumb wheel on the master control panel. Each of the ram commands may be inhibited by the test operator with switches on the master control panel.

The hydraulic load ram is operated in a closed loop servo-system. The input commands to the servo-amplifier are from the ramp and function generators and the load offset. The feedback command to the servo-amplifier is true ram force measured at the tip of the ram by a standard strain gage load cell. This signal is amplified prior to being applied to the servo-system and is also used as a force signal in the measurement system.

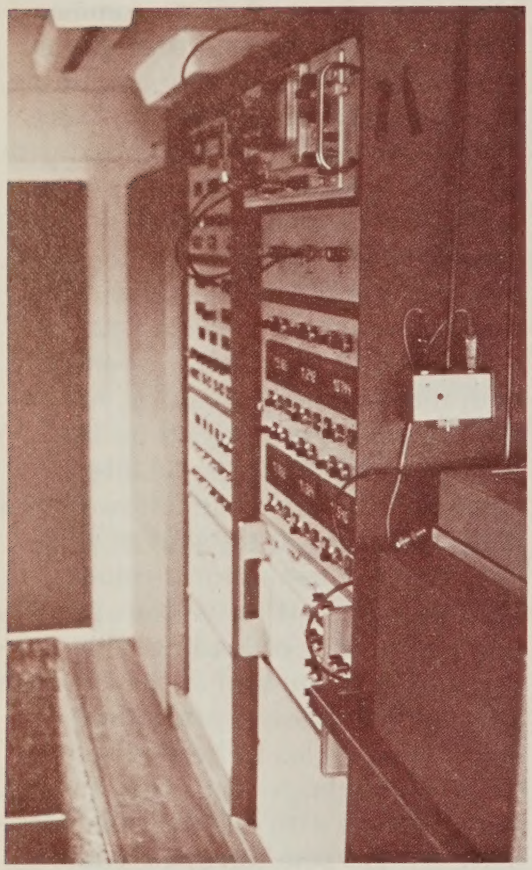


Figure 3.—Master control panel.

When all tests have been performed at a particular test site, a sequence of steps is activated either automatically or manually to return the van to a condition where it may be moved to the next location. It takes about 1 minute to retract the reference beam, lift the rear jack pads, and return the suspension to normal.

Measurement subsystem

The measurement subsystem records pertinent data signals during testing with various devices for either immediate or later evaluation. The pavement displacements are measured by six LVDT's which convert linear displacements of their shafts to proportional electrical signals with a resolution greater than 0.0025 mm; these are read out directly in inches on digital panel meters (DPM). An oscilloscope is used for system setup, maintenance, and viewing the dynamic loading characteristics from the load cell to insure the ram force wave is free of distortion at the higher repetition rates. Input signal switching for the oscilloscope is accomplished by the data switching panel where the oscilloscope may be connected to one of the LVDT's load, ramp command, or function command. The DPM's are permanently connected to the 6 LVDT's and continuously monitor the transformer's position in inches with a resolution of 1 part in 10,000. These readouts are used primarily for system setup and long term creep tests where only occasional readings are required.

To provide permanent data records, a light beam oscillograph and a 14-track analog magnetic tape recorder are used. Data signals are paralleled to both machines at all times so that either one or both may be used for a particular test. The light beam oscillograph records data from the six LVDT's and the load cell on light-sensitive paper as continuous lines, one for each channel, that vary in deflection as the signal level increases or decreases. Precise time lines are also generated on the record for time-related studies. The strip chart paper speed is variable from very slow to about 4 000 mm per second depending on the type of test. The paper drive may be automatically or manually started or stopped by a selector switch on the master control panel.

The analog FM magnetic tape recorder is a 14-track unit with the same input signals as the oscillograph. This data storage medium allows subsequent signal conditioning and digitization for computer processing of the data. As with the oscillograph, the machine may be started and stopped automatically or manually. To help locate data segments, a voice channel is used in addition to the 14 data channels on the tape.

Two additional displays constitute the measurement system. They are the load readout DPM and surface temperature readout. The load readout DPM continuously indicates the ram force and is used as a reference in setting up the static load conditions. The surface temperature digital readout monitors a noncontact, infrared thermometer aimed through a hole in the floor of the van.

The electrical power for the instrumentation is supplied by a 4 kW-120 V a.c. gasoline engine driven generator located in the rear of the van. The internal lights are powered by 12 V d.c. from the vehicle battery.

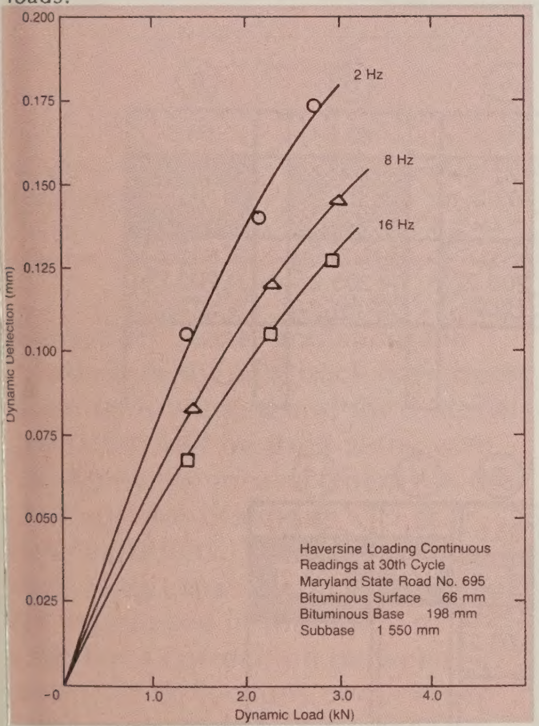
Applications

All deflection tests on pavements seek to extract information about the structural properties of the pavemen system by distortions measurable at the surface. Although a single measurement at the load point does provide useful information for overlay design, especially if a series of such measurements is made, it must be connected to the overlay design methodology by empirical experience. The well-known sensitivity of these measurements t seasonal temperatures and moistur changes requires some system of corrections even for this simple approach to overlay design. Moreover, current understanding c pavement structural behavior indicates that these temperature ar moisture sensitive pavement properties are significant in the cumulative damage functions unde repeated loading. This is the incentive behind dynamic testing fr more complete information about the pavement structure.

The versatility of "Thumper" complicates its application to the immediate problem but assures the researcher that data will be available for a solution once the loadings can be properly applied and the recorded information can be analyzed.

Since the delivery of the system to the Pavement Systems Group of the Structures and Applied Mechanics Division in August 1977, field operations have continued. A staff study has been authorized to develop methodology and computer programs for determining the moduli of the various pavement layers. One phase of the staff study, in cooperation with the Maryland Department of Transportation and the University of Maryland, is using "Thumper" to get deflection measurements at various sites in Maryland under differing seasonal climatic conditions. The objective is to use the capabilities of the device in developing a standard nondestructive method of measuring the stiffness properties of the pavement system for use in a layered system computer program for design of overlays.

Figure 4.—Change in deflection from changes in the frequency and magnitude of applied loads.



Two series of measurements, one in April 1978 and another in June 1978, have been made at three sites on Maryland pavements and the data is being analyzed. A computer program has been written which is being used to digitize the data taken on analog tape and plot the deflection basins. Preliminary analysis has been performed by visual examination and evaluation of the oscillograph recordings. Additional computer programs are being developed for more complete analysis of the data and for computation of the layer moduli. Standard test procedures have been developed. Figure 4 depicts the difference in dynamic deflection resulting from changes in the frequency and magnitude of the applied loads. It is this type of information, resulting from the versatility of "Thumper," that will eventually lead to a better understanding of pavement response.

At the request of the State of Florida, deflection responses were measured on various pavement surfaces over econocrete type bases on a newly constructed portion of US–41 just north of Fort Myers, Fla., in February 1978. Twenty-two sections of pavement were tested at this site. Several differing test series were performed on each section to examine the behavior of the various pavement systems in response to a variety of loading conditions.

Also, at Georgia's request, test deflections were obtained on experimental overlays in Georgia to help evaluate various overlay and rehabilitation strategies. The overlays were placed as an experimental project and are being evaluated in a Highway Planning and Research Program study. The project is on I–85, northeast of Atlanta, Ga. The overlays, over rigid pavement, provided an opportunity for the study of joint action after overlay with several different overlay strategies. Deflection measurements were made near the joints and at the edge, near the center of each slab. Only preliminary analysis of the data has been made, but it is easy to pick out performing and nonperforming load transfer devices.

Pennsylvania State University and the Pennsylvania Department of Transportation have been studying pavement performance for several years on a 1.6 km test track at the University (fig. 5). The performance of various overlay strategies are being measured as well as the performances of original pavement sections. "Thumper" has been used on two occasions to measure deflections of the various sections.

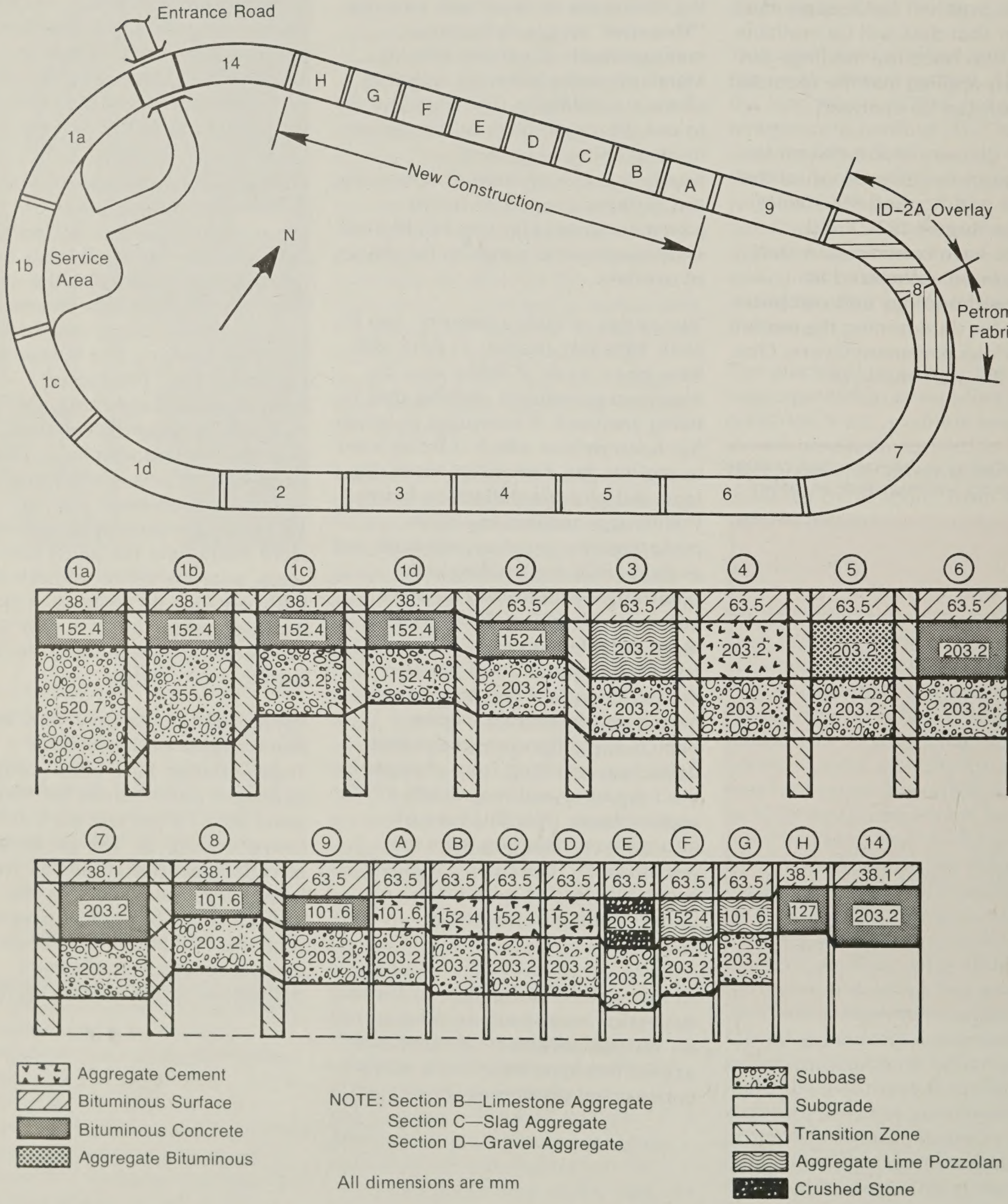


Figure 5.—Schematic of Pennsylvania State University test track.

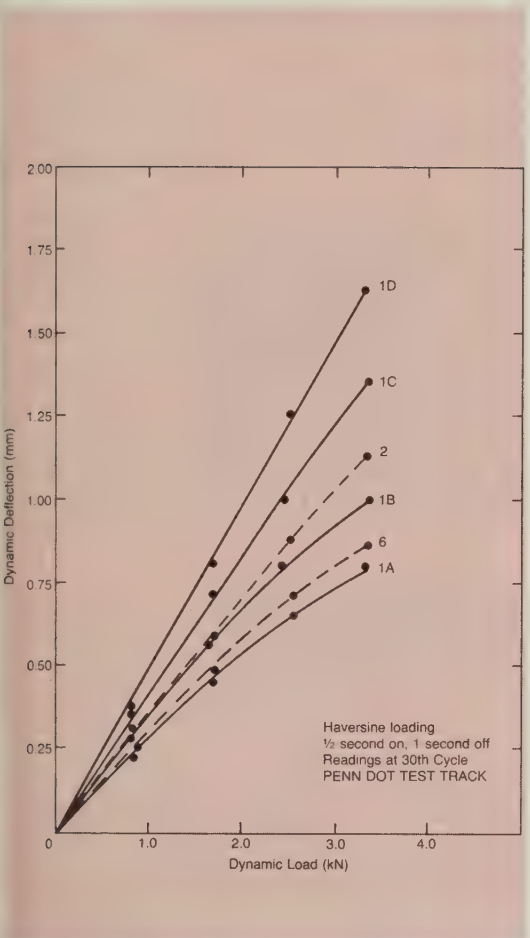


Figure 6.—Results from the Pennsylvania State University testing.

The surface, base, and subgrade materials in the test track have been thoroughly studied and the material properties will be used along with the deflection measurements in the development of moduli determinations. The creep deflection measuring capabilities of "Thumper" are being used in this study to establish a correlation between pavement deflections under creep loads and rutting performance as predicted by the VESYS analysis system. Figure 6 is an example of the results being obtained at the University.

Future Plans

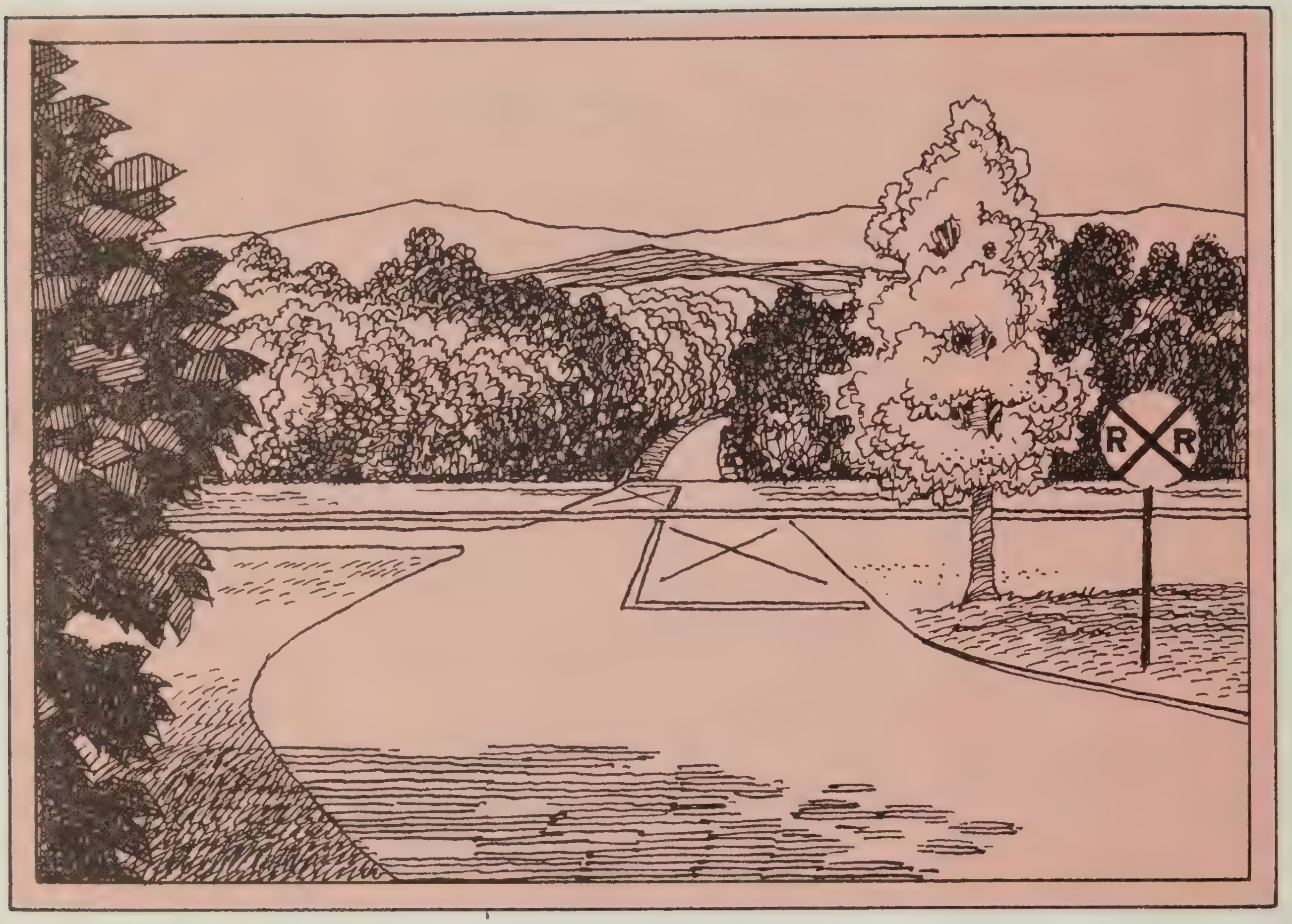
The Pavement Systems Group will continue to use "Thumper" in the development of new approaches to the following: Moduli determination, the standardization of pavement evaluation methodology, and improved overlay design. Although it is a rather costly and highly sensitive piece of equipment for routine deflection measurement use, it is a superb research tool.

Research is being initiated to use the unique capabilities of "Thumper." Pavement systems are complicated structures consisting of two or more layers, each of which has a number of parameters which characterize their response to load. "Thumper" offers a possible means of isolating and evaluating these responses. It can measure not only the dynamic deflection over a wide range of frequencies but also the accumulative deflection of the entire basin, the time rate of this accumulation, the rebound rate and amount, and the resulting permanent displacement. It is hoped that new equations for determining the layer parameters may be evolved through these capabilities. It may be possible to establish creep sensitive and frequency sensitive correlations which will allow direct measurement of these parameters in types of load response experiments in which the remaining elastic responses do not appear.

Work will also continue on analysis of data now stored on tapes. Additional measurements are planned both in Maryland and at the Pennsylvania State University test track. In addition, the effects of freeze-thaw and frost heave on the response of pavements will be studied.

T. F. McMahon, Chief of the Pavement Systems Group, Structures and Applied Mechanics Division, Federal Highway Administration, is responsible for research in the design and construction of highway pavements. Dr. McMahon has been involved in research programs in quality assurance of highway construction and compaction of highway materials and has authored many papers on quality assurance and pavement design. Prior to his employment with the Federal Highway Administration, he was a member of the staff of civil engineering at the University of Kansas.

R. W. May is a civil engineer in the Structures and Applied Mechanics Division of the Office of Research, Federal Highway Administration. He joined FHWA in 1975 and has been working in the Pavement Systems Group in the areas of pavement evaluation and rehabilitation.



Railroad Grade Crossing Passive Signing Study—Phase 2

by Janet Coleman, Joseph S. Koziol, Jr., and Peter H. Mengert

This article presents the results of a study to determine the effectiveness of new passive signing systems in warning drivers of the potential hazards at railroad grade crossings and is a condensation of the study final report. (*1*)[1] Experiments were conducted over a 2-year period. The results of Phase 1 were previously reported and indicated improved effectiveness for the new signs tested. (2) This led to a decision by the study advisory committee to continue testing into Phase 2. The results of Phase 2 confirmed the findings of Phase 1. With the new signs, drivers displayed more awareness (that is, increased percentage of head movements or looking for trains) at the crossings tested.

[1] Italic numbers in parentheses identify the references on page 134.

Introduction

In the interest of greater motorist safety, and because th existing signing system (at-crossing sign and advance warning sign combination) has not been changed for many years (other than the angle of the crossbuck, whic was changed in the 1971 Manual on Uniform Traffic Control Devices), a two-phase study was initiated to evaluate the effectiveness of new passive signing systen in warning drivers of the hazards at railroad grade crossings. Each new signing system was composed of a at-crossing crossbuck and an advance warning sign.

Phase 1 of the study evaluated seven new passive signir systems at five sites in Ohio and one site in Maine. Th purpose of Phase 1 was to determine, on a limited sca if any of the new signs were more effective than the existing signs and to determine the important variable for the study. Seven signing systems were selected by study advisory committee formed in the initial stages this study. The committee consisted of representatives of 25 participating States, the Federal Railroad Administration (FRA), the Federal Highway Administration (FHWA), and the Association of American Railroads (AAR).

Phase 1 data collection was completed in October 1975 and the results indicated that the new signs increased driver awareness at railroad grade crossings. Phase 2 was undertaken to test and verify on a nationwide basis the most effective signs as determined in Phase 1, to quantify the expected improvement from the new signs, and to recommend what sign or signs should be adopted for driver warning at railroad grade crossings. This article presents the results from Phase 2 of the study.

Signs

The three new signing systems evaluated in Phase 2 of this study are shown in figure 1 together with the existing (base) system.

System 1 (the Texas system) consisted of a red and yellow ıdvance warning sign and a white crossbuck with the 'Railroad Crossing" legend in black lettering ;uperimposed over a circular red and yellow background vith black border. This was the most effective signing ;ystem tested in Phase 1 of this study.

System 2 consisted of a red and yellow advance warning ;ign and a yellow crossbuck with black border. This ystem was not tested in Phase 1 but was selected for 'hase 2 for three reasons: (1) The yellow crossbuck was :urrently being tested by a number of States, (2) the 'ellow crossbuck was also in wide-scale use in Canada, nd (3) the yellow crossbuck was part of the Phase 1 ystem which was second best in terms of increased lriver awareness.

ystem 3 consisted of a red and yellow advance warning ign and a standard white crossbuck with the "Railroad :rossing" legend in black lettering. This system also was ot tested in Phase 1 of this study but was selected for hase 2 testing to determine if the colorful advance 'arning sign alone could explain the expected system nprovement. Most of the new systems tested in Phase 1 ıvolved two new signs—a crossbuck and an advance arning sign; none involved just a standard crossbuck ith a new advance warning sign. If system 3 were 'roven to be just as effective as system 1 in the Phase 2 'sts, only half as many signs (the advance warning sign ɹt not the crossbuck) would be involved in any 'commended replacement program.

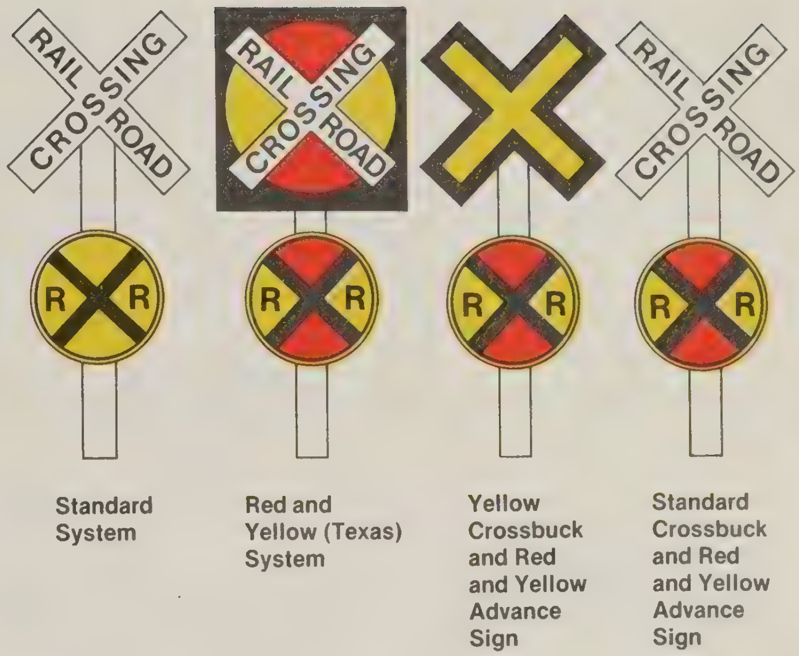


Figure 1.—Crossbucks and advance warning signs tested in Phase 2.

Although the "look-for-train" advance warning sign was included in the system which was second best in Phase 1, it was not included in Phase 2 because it was viewed as a supplementary sign which could be used at the discretion of the local authorities.

Site Characteristics and Test Conditions

Experiments were conducted at 18 sites in 14 States (fig. 2). The sites were fairly well distributed across the country, with a good representation of the 25 States participating in this project. There were two sites each in California, New Mexico, Texas, and Louisiana, and one site each in Maine, Maryland, Minnesota, North Dakota, Montana, Washington, Colorado, Kansas, Nebraska, and Georgia. As in Phase 1, each site was selected based on the following general requirements:

- Two-lane, two-way rural road with a speed limit greater than 72 km/h preceding the crossing.
- Average daily traffic (ADT) between 1,000 and 4,000.
- An average of two to four trains per day.
- Sight distance restrictions in at least one of the two quadrants for the direction in which driver behavior was observed.

All tests were conducted in only one direction and only during good weather conditions (no rain, snow, fog, or wet roadway). All vehicles crossing the tracks were

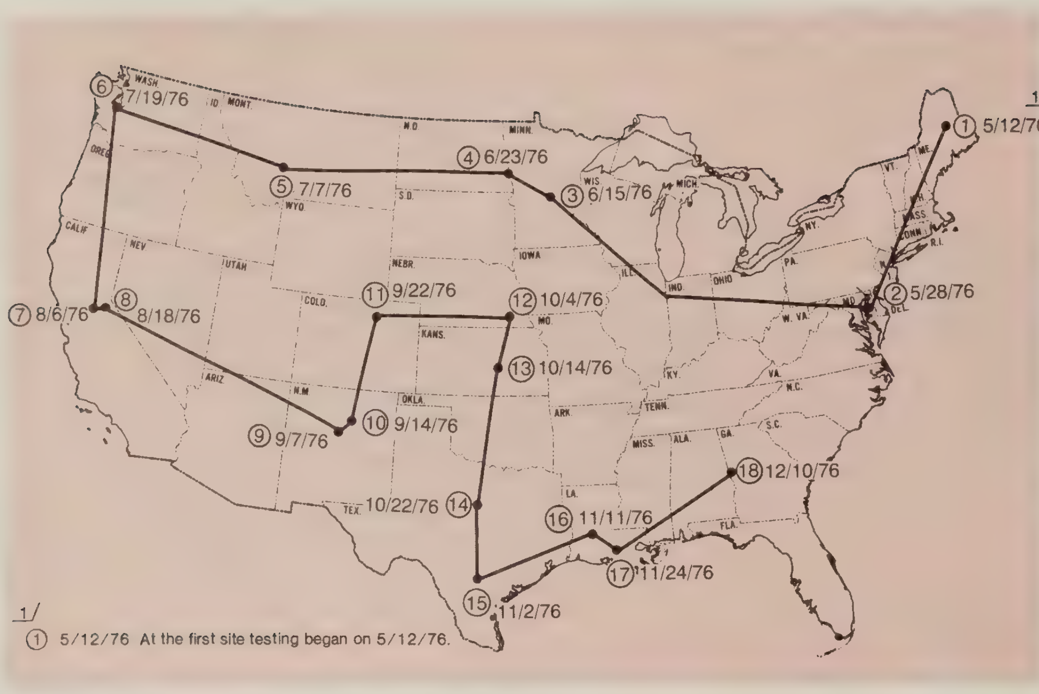


Figure 2.—Site locations and test schedule.

included in the analysis except the following: (1) Vehicles required by law to stop, (2) vehicles approaching within 5 minutes of a train arrival, (3) short headway vehicles (less than 6 seconds between two vehicles approaching the crossing in the same direction; only the following vehicles were excluded in these cases), and (4) vehicles turning off the road in the data collection zone.

Experiment Variables and Measures of Effectiveness

During the course of the experiments, both dependent and independent variables were measured or recorded manually for each sampled vehicle as it traversed the test area.

Independent variables

The *independent variables* controlled for in the study were as follows:

- Site (18 in 14 States).
- Time of day (day/night).
- Observer collecting manual data (two at each site).
- Observer location (van, car).

In Phase 2 of the study, two observers were used to collect the manual data. Both a car and the van which housed the electronic equipment served as observer locations. The observers alternated in collecting the manual data from within each vehicle.

Dependent variables

The *dependent variables* were head movement (driver looked for train) and speed profile. Head movement da were collected visually and recorded manually, while speed profile data were collected electronically.

Measures of effectiveness

The following measures of effectiveness were formulat from the dependent variables and were used for evaluating the effectiveness of the new passive signs.

Head movement (defined for observer No. 1, observe No. 2, car location, and van location)

Each observer was instructed to gather head moveme data on the driver of each vehicle within the data collection zone according to one of three conditions Definite head movement (driver looked for train), definitely no head movement (driver did not look fo train), and not sure.

The data collection zone was the area 61 to 244 m before the crossing. The head movement measure (percent of head movement) was defined as follows:

$$\text{Percent of Head Movement} = \frac{\text{Number of Head Movements}}{\text{Number of Head Movements} + \text{Number of Definitely No Head Movements}}$$

Because there were two observer locations (car and van), the observers were instructed to distribute their time as equally as possible between the two locations. The head movement measured by each observer was averaged over the two observer locations.

Head movement was considered the primary measure in this study; it provided not only an indication of the driver's attentiveness and safety orientation, but also a direct and positive indication of the driver's sight of and reaction to a particular signing system.

Agreed head movement

This measure considered only those drivers for whom both observers agreed on either "definite head movement" or "definitely no head movement." More consideration was given to this measure in the analysis because it reflected the results of both observers.

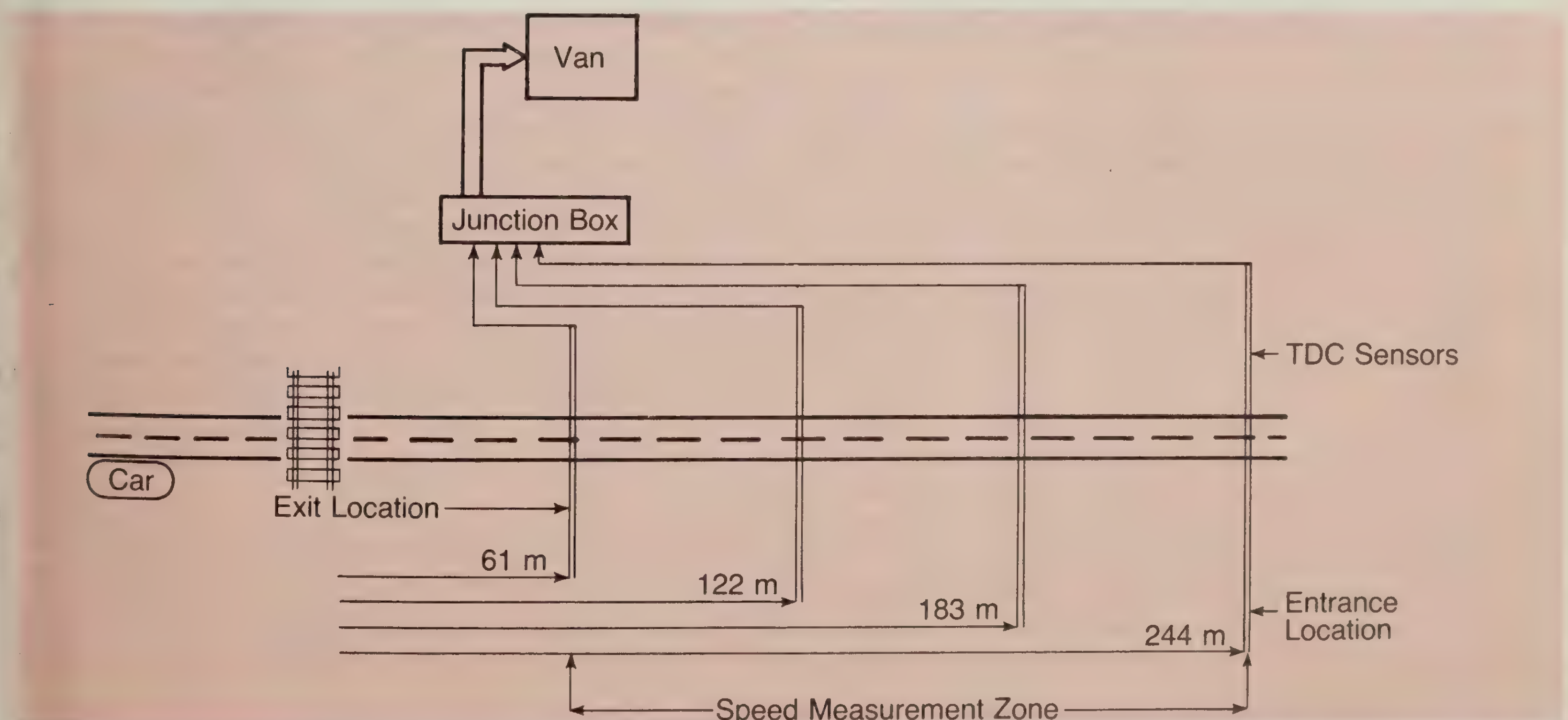
Speed reduction

Speed reduction was defined as entrance speed 244 m from the crossing on the approach side minus exit speed 61 m from the crossing on the approach side (fig. 3). This measure indicated whether or not the driver reacted to the signing system by slowing down in the approach to the crossing. In general, large values of speed reduction implied more effectiveness.

Speed near crossing

This was the speed 61 m from the crossing on the approach side. This measure provided information on the relative safety aspects between signs and sites. When coupled with speed reduction, it also provided information on the vehicle's speed profile near the crossing. Because advance warning signs were located approximately 91 to 183 m from the crossing, reaction to the advance warning signs was expected to occur before the driver was "near the crossing." In general, smaller values of this measure implied more effectiveness. However, in the analysis, a "speed near crossing" effect was not considered important unless accompanied by a "speed reduction" effect.

Figure 3.—A crossing approach field equipment setup and observer locations.



Data Collection and Test Schedule

Electronic and manual data were collected on only one side of the crossing at each site. The signs were installed on both sides of the crossing. The electronic data provided speed profile information and were obtained using a data acquisition system housed in a mobile van. The mobile van, the same one as used in Phase I, was parked off the side of the road about 61 m from the crossing on the approach side. A second vehicle (car) was also used in Phase 2, primarily as transportation for the data collectors in the vicinity of each site once the data acquisition system was set up. During the experiment, the car was parked on the opposite side of the road and on the opposite side of the crossing from the van, within 30 m of the crossing. One data collector was stationed in each vehicle when collecting the manual data (fig. 3).

Although motorists could see the van and car, the vehicles were parked as unobtrusively as possible and did not seem to affect the drivers' reactions to the various systems. It should be emphasized that the vehicles were present for all systems including the standard base system.

Sensors laid across the lane of the road on the approach side of the crossing measured speed profile data. Each axle of each vehicle activated the sensors.

Manual data were collected on a clipboard by each observer and at each location (van and car). The manual data consisted of vehicle crossing time, head movement state, required stop vehicles, turning vehicles, and train crossing time (whenever it occurred during the data collection period).

All data were collected for the three new systems and the base system during one visit to each site. After the data acquisition system was set up and checked out, the data collectors obtained data for the base system, then for each of the new systems, and finally for the existing base system once again. The order in which the new systems were installed was experimentally balanced across the 18 test sites.

The minimum sample size per system per site was 200 vehicles. At most sites, this resulted in 1 day of data collection per sign system. For those sites with lower traffic volumes, a 2-day limit was set on data collection. This was done to keep the total experiment within reasonable length and cost and to allow a reasonable schedule to be set up for coordinating the data collection progress with the local authorities in whose States the tests were being conducted.

Each system (except the base "before") was installed on the morning of data collection. No data were collected over weekends, holidays, or during bad weather. Over weekends and holidays, the base system was reinstalled (if not already set up) until testing was resumed the following week. If weather did not permit testing on a particular day, the last installed system was kept up until weather permitted the next system to be tested. Testing during the day was equally distributed between morning and afternoon periods.

Speed profile data were collected at night at six sites. All 18 sites were not used because no night effects were found during Phase 1. Testing at night was restricted to a 4-hour period starting about 1 hour after sunset.

Results

Four different types of analyses were performed on the data. Only one type of analysis, the two-way analysis of variance (ANOVA), is discussed in this article. In addition to the two-way analysis of variance, the following analyses were performed and are discussed in detail in the study final report: A three-way analysis of variance, an analysis of the individual means and standard deviation of the measures for each system at each site, and an analysis of the subjectivity of the head movement data. (*1*)

A two-way analysis of variance was performed for five sign conditions—the three new systems, the base "before," and the base "after." The two variables were site and sign condition. Table 1 summarizes the levels of significance for each variable and for each measure of effectiveness. The results show that there were significant differences from site to site for all the measures. This means that the nominal behavior (head movement and speed) of drivers approaching crossings differed from site to site. The more interesting finding was that there were sign condition effects[2] for all the head movement measures and the speed near crossing measure but not for the speed reduction measure. The table 1 results show that there was a difference in significance level for the head movement data from observer 1 and observer 2. One possible explanation for this is that observer 2 may have been more stringent in his requirements for recording head movement.

[2] In the analysis, 0.05 was used as the level of significance for declaring an effect.

Table 1.—Two-way ANOVA summary table

Conditions	Effectiveness measures						
	Observer 1 head movement	Observer 2 head movement	Agreed head movement	Van head movement	Car head movement	Speed reduction	Speed near crossing
Site (18)[1]	0.01[2]	0.01	0.01	0.01	0.01	0.01	0.01
Sign condition (5)[3]	0.01	0.05	0.01	0.01	0.01	—	0.01

[1] Eighteen sites were used in Phase 2.
[2] Significance level.
[3] Five sign conditions were used in the ANOVA.

Table 2 shows the mean sign condition effects for the five sign conditions, including the base "before" (B1) and the base "after" (B2). The 95 percent confidence ranges for multiple comparisons (Bonferroni Method) are also shown. An interesting finding was that system 1 was superior not only to both the B1 and the B2 sign conditions for all head movement measures, but also to the average of the five head movement measures. In terms of the average head movement measure and most of the other head movement measures, system 1 was significantly better than both base conditions. System 1 was superior to, but not significantly better than, systems 2 and 3. Finally, systems 2 and 3 were found to be equally effective and superior to but not significantly better than the B1 and B2 conditions. The fact that the B2 sign condition showed more head movement than the B1 sign condition was probably due to a number of factors including the following: (1) The sign testing effect itself (that is, people became aware of the sign testing), and (2) the observers became less stringent in their requirements for what constituted a head movement.

The only major finding for the speed measures was for the "speed near crossing" measure where system 1 was significantly better than the B1 condition and almost significantly better than the B2 condition. However, the change in speed was small, about 1.6 km/h, and was not

Table 2.—Mean sign condition effects—five signing systems including base before (B1) and base after (B2)

Sign condition	Effectiveness measures							
	Observer 1 head movement	Observer 2 head movement	Agreed head movement[1]	Van head movement	Car head movement	Average head movement	Speed reduction	Speed near crossing
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*km/h*	*km/h*
B1	30.6	35.7	28.2	34.2	32.1	32.2	8.85	63.57
1	40.5	43.1	39.5	43.6	40.3	41.4	8.98	61.49
2	37.7	38.9	34.3	38.7	38.1	37.5	9.01	62.15
3	36.6	39.4	35.0	39.3	36.8	37.4	8.87	62.70
B2	34.4	37.9	29.4	34.8	37.2	34.7	8.77	62.97
95 percent confidence range 3.93	4.5	6.5	5.5	5.7	5.1	5.5[2]	0.95	1.61

[1] In the subjectivity analysis it was found that the observers agreed 76 percent of the time on the driver head movement state. Complete details are covered in the study final report. (*1*)

[2] For determining significance, the difference between any two sign configurations was compared to the 95 percent confidence range. In order to have a significant improvement in average head movement, at least a 5.5-percent increase in head movement is necessary.

accompanied by a speed reduction effect. Hence, no significant speed profile effect was declared for any of the new systems. Only the day-data results are presented because no speed change effects were found for the new systems at night.

In retrospect, the following comments are offered with regard to the measures of effectiveness used in the study. Accidents at grade crossings are rare; if a study were planned to detect safety benefits (accident reduction) due to new signing, at least 3 to 5 years of data collection at large numbers of crossings would be needed. To both shorten the study period and reduce the number of test sites needed, alternative measures of effectiveness were used in this study. Studies have shown that drivers do reduce their speeds somewhat on the approach to a grade crossing in anticipation of a rough crossing. During the passive signing study, speed reduction was used to see if there was any additional reduction in speeds attributable to the new signs. The speed data showed that the changes were not significant. During the study, the sites that were selected had sight restrictions in at least one of the approach quadrants. Drivers were required to make an obvious "head movement" to look for the train. A reduction in speed was the desired objective, but normal drivers do not reduce their speed until after they have perceived a hazard. Perceiving a hazard comes after looking for a hazard. More drivers did look for trains when the new signs were present; therefore, more drivers would have been in a better position to reduce their speeds if a hazard (an approaching train) had been perceived.

During the course of the study, no extensive cost analysis was made of the total costs involved in changing to a new advance warning sign nationwide. In preparation for the final meeting of the study advisory committee, cost estimates for grade crossing signs were obtained from five States. The average costs were about $100 per sign or $200 per crossing. The average life of a grade crossing sign is approximately 7 to 8 years. Two estimates on the cost of the new red and yellow advance warning sign showed a cost increase of approximately $0.50 per sign or $1 per crossing. The average cost of a fatal accident is now estimated at $331,465.[3] If only one fatal accident were averted in a 7- to 8-year period, this would cover the cost of installing the new advance warning signs (with 2 signs per crossing at a cost of $201) at about 1,649 crossings.

The FHWA's Office of Research plans additional work with the new advance warning sign to determine the following: (1) The safety benefits to be expected through the use of the new sign, and (2) the total costs involved in changing to the red and yellow advance warning sign at passive grade crossings nationwide.

Conclusions and Recommendations

The major findings[4] of Phase 2 of the Railroad Crossing Passive Signing Study were as follows:

- System 1 showed significant improvement over the base system in terms of head movement, averaging (over all sites and all measures) an increment of 6.7 percent more head movement (from 34.7 percent to 41.4 percent).[5]
- The advance warning sign of system 1 (also system 3) accounted for almost half (from 34.7 percent to 37.4 percent) of this improvement.
- Systems 2 and 3 were equally effective in terms of head movement and respectively showed increments of 2.7 and 2.8 percent more head movement (from 34.7 percent to 37.4 and 37.5 percent) than the base sign configuration.
- The base signs showed no significant differences in speed profiles compared to the base sign configuration under day or night conditions.
- There were significant differences in head movement improvement from site to site.

[3] Based on the 1977 National Highway Traffic Safety Administration figures which include $244,480 for loss of future earnings and $86,98 for other expenses.

[4] Refer to table 2, the average head movement (percent) figure.

[5] The estimates of improvement are conservative because the base "after" percents were used in the comparisons.

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Debris Problems in the River Environment

by
Stephen A. Gilje

The Federal Highway Administration (FHWA), Office of Research, is sponsoring the research study "Debris Problems in the River Environment." The objectives of the study are to determine the magnitude of the debris problem and examine the significance of the factors which determine the potential for debris accumulation at highway bridges. Debris is defined as any material not generally transported during normal stream flow and which causes hydraulic problems when accumulated against a bridge or at a highway crossing. The study will also investigate the effectiveness of countermeasures presently used for debris problems and recommend those which may be useful. The study is scheduled for completion in 1979.

This article[1] presents the research progress to date and focuses on the magnitude of the debris problem, debris control practices, and countermeasures which can be used for debris accumulation. Debris accumulation is shown to be a significant problem which has not received proper recognition in the past.

Introduction

In 1969, Hurricane Camille swept across the Eastern United States leaving a path of destruction in its wake. The communities of Nelson County, Va., felt the full impact of the storm. (*1*)[2] In less than 5 hours, 762 mm of rain fell on this rural mountain county, causing streams to flood. However, the damage was much worse than that caused by normal flooding. The rain fell with such intensity that the soil on steep slopes gave way under the added weight and lubricating forces. Streams cut deeply into the bases of hillsides and added to the instability. Under the constant and prolonged deluge, forests were unable to hold the earth against the flooding forces. Massive landslides resulted sweeping tons of mud, rock, trees, shrubs, and miscellaneous debris into the streams. For the most part, the mud and rock passed through the streams with little damage, but the trees and other debris did not. Debris jammed culverts and became braced on bridges. Streams were dammed and flood levels raised. When the rain finally stopped and the flood passed, 91 bridges in Nelson County were damaged or gone. Because this same destruction could happen again if the remaining debris was not removed immediately, the Governor's office issued a report stating that debris clearance was among the more urgent items to be carried out after the flood: "Definite hazards exist if the collection of debris is not removed and remains until high water refloats the debris and sends it again downstream, resubjecting transportation routes and other properties within the flood plain to further damage or destruction."(*1*)

Although the events which occurred in Nelson County are not uncommon, they usually do not occur with such intensity. Landslides may not be the source of debris; large amounts of debris may come from clearcutting or lumbering

[1] Paper presented to the 1978 Transportation Research Board Committee A2A03, Hydrology, Hydraulics, and Water Quality, in Washington, D.C.

[2] Italic numbers in parentheses identify references on page 141.

operations and trailer parks or garbage dumps. Sometimes even healthy forests in the flood plain may provide enough debris during flooding to create problems. Although the Nelson County situation has occurred in hundreds of localities thoughout the country, debris generally has not been recognized as a major hydraulic problem, has not received much attention in the highway community, and has not been studied in much detail. Damaging debris accumulations occur during major flooding and have been accepted as an inevitable part of the natural catastrophic event. As a result, little research has been devoted to the problem, and each community or State agency is left to its own devices to clean up and take emergency actions to save as many stream crossings as possible.

An FHWA administrative contract is underway to evaluate the magnitude of the debris problem and to investigate possible countermeasures.

Extent of the Debris Problem

A study completed for the FHWA in 1972 found that about 20 percent of all bridge failures were partially attributable to debris. (*2*) The data were derived from examination of bridge damage reports compiled by FHWA's Office of Engineering. Data collected for bridge damage reports primarily document the destruction of highway facilities and specify the extent and cost of the damage.

The cause of the damage is not always specified in bridge damage reports; in many instances, debris is not identified as the problem even though it may be a major factor. In the 1977 FHWA study, photographs of the damage, news articles, and descriptions of the extent of the damage were sufficient to determine the cause of the damage. Debris was visible in the majority of the photographs of the damaged structures, although damage was attributed solely to debris only when so stated in the reports or when positive evidence was available to determine that debris was the cause. If more accurate and revealing data were available, the proportion of damage caused by debris probably would be higher than 20 percent. In addition to the large amount of damage exclusively due to debris accumulation, debris increases the severity of other hydraulic forces.

The research study completed in 1972 shows that debris is a significant factor in bridge damage, but because bridge damage reports are filed only for catastrophic floods, the number of cases is limited and information from this type of flooding is expected to be biased. It is still difficult to be certain of the magnitude of the role of debris in all stream crossing damage, but ongoing research is attempting to get a more complete picture of the problem.

In 1977, 44 States were surveyed in order to place the debris problem in better perspective (fig. 1). Figure 1 indicates that debris poses problems of varying degree. Only two States reported that debris caused no problem. In approximately 20 States, debris often blocks waterways, increases local scour, or otherwise endangers bridges. Only two States have done research on debris, and that research was on a case-by-case basis. None of the States that were surveyed had specific maintenance guidelines for debris problems.

A puzzling inconsistency is apparent when reviewing the statistics presented in figure 1. There is a sharp contrast between the large number of States which recognize debris as a problem and the very small number of States which have either investigated debris or have specific maintenance guidelines. This suggests that many States have identified hazards associated with debris, but the extent and the importance of the problem is not understood.

A review of State agency drainage manuals reveals that only 17 States discuss debris problems at all and then usually only in regard to culvert design; the manuals indicate the possible presence of debris should be taken into consideration in the design of a new bridge and remedial measures should be taken if necessary.

Figure 1.—Summary of States' attitudes toward debris problems at bridges.

QUESTION	NUMBER OF STATES YES	NUMBER OF STATES NO	REMARKS
1. DOES FLOATING DEBRIS CAUSE BRIDGE OR STREAM CROSSING MAINTENANCE PROBLEMS?	42	2	MAJOR PROBLEM: 4 STATES MODERATE PROBLEM: 25 STATES MINOR PROBLEM: 13 STATES NO PROBLEM: 2 STATES
2. A. HAVE ANY MEASURES BEEN UTILIZED TO ALLEVIATE DEBRIS PROBLEMS OR	39	5	GENERAL MEASURES (FREEBOARD, LONGER SPANS, SOLID PIERS, CLEARING)
B. HAVE ANY STUDIES BEEN MADE ON DEBRIS ACCUMULATION?	2	42	RESEARCH (CASE-BY-CASE
3. HAVE THERE BEEN ANY DESIGN OR MAINTENANCE GUIDELINES DEVELOPED TO COUNTER DEBRIS ACCUMULATION PROBLEMS?	0	44	STATES DO NOT HAVE SPECIFIC GUIDELINES

Types of Debris

Vegetation is the most common and troublesome debris. Trees cause the biggest problem because of their size, shape, and facility for entrapment at bridge piers (fig. 2). The degree of hazard posed by trees is dependent on their abundance in the flood plain, stream channel stability, and land use practices in the flood plain. Debris is particularly troublesome in actively meandering streams because of the continuing lateral erosion of the stream banks. Clearcutting with inadequate cleanup and standing dead and burned trees also contribute to the debris problem. Smaller vegetation such as tree branches, leaves, and shrubs add to river debris and may cause local flooding by clogging culverts.

Figure 2.—Massive vegetation debris accumulation on a bridge pier.

Manmade debris occurs in a variety of shapes and sizes and, unfortunately, is plentiful. Anyone who has stood beside an urban stream has observed the problem. In addition to esthetic pollution, this debris causes drainage problems when trapped in culverts or on bridges. In one neighborhood where flooding had suddenly and unexpectedly become a problem, a refrigerator, two air conditioners, an assortment of old tires, and a tree stump were found in a large culvert beneath the street. In a large flood, almost anything may become debris if it is dislodged, transported, and finally comes to rest against a highway facility. This may include cars, trailers, and even houses or barns (fig. 3).

Ice, too, may act as debris during the spring breakup. It can build up tremendous stresses on bridge piers and can block culverts and other drainage fixtures. Ice flow mechanics and accumulation have been studied extensively by many government agencies; however, this area of research falls outside the scope of the present study and is not discussed in this article.

River sediment may become a debris problem if it occurs in extremely high concentrations or in steep mountain streams when large rocks are picked up by the flow and carried at high velocities. Small bridges may literally be beaten to destruction by large boulders carried in rapid flow.

A debris classification system has been proposed by the California Division of Highways and adopted for use in evaluating debris at culverts. (*3, 4*) The classification system is generally limited to natural debris which is more accurately estimated when planning a highway facility. The classification scheme distinguishes between types of floating debris, flowing debris, and stream sediment. The emphasis is on objects that may collect or deposit in tranquil stream locations or get trapped in culverts.

A similar classification for bridges would place emphasis on entire trees, parts of trees, other vegetation, large manmade objects, large sediment concentrations, and boulders.

Hydraulic Hazards Caused by Debris

Debris may block the waterway and increase bed, bank, and abutment scour. On rare occasions debris may pose a fire hazard when not cleared following a flood. (*5*)

The most destructive damage occurs when the waterway is completely blocked and the bridge facility becomes a dam. As the water stage rises rapidly, water and additional debris place extreme loads on the crossing facility. If the bridge superstructure withstands the tremendous lateral pressures and the flood stage rises further, the highway will be flooded. Erosion of the fill, roadway, and abutment may progres[s] until the facility fails. Because culverts offer a limited opening, complete blockage is more commo[n] than blockage of larger structures and highway flooding usually results.

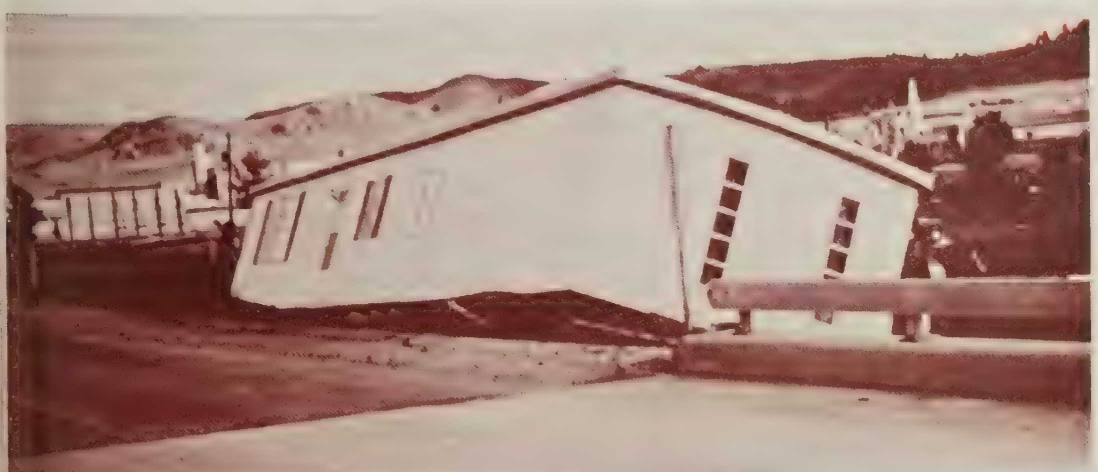
Partial blockage is more common than complete blockage. When the size of the waterway opening is decreased, the water stage upstrea[m] rises. Flow velocities through the opening increase. Increased flow velocities cause excessive scour, deeper than predicted for an unobstructed opening. When floating debris lodges against a single bridge pier, the effective size of the pier is increased. This creates an intensive diving flow which increases local scour at the pier footing. In the worst case, the increased general scour combines with local scour at the pier footing to produce scour far worse than predicted in the design of the structure. Approach section washouts, road damage, and other erosion problems are common in many cases of heavy debris accumulation (fig. 4). Yet, even small debris collections can significantly alter the hydraulics of bridge waterways. On State Highway 172 near Parker, Ariz., velocity measurements were made during a flood on Oshorne Wash. The bridge was designed for a 50-year flood, and velocities through the bridge opening were estimated to be 1.9 m/sec for a flood of this high magnitude. When debris occupied 11 percent of the waterway opening, velocities for a much smaller flood (return interval 35 years) were in excess of 2.4 m/sec. Obviously, the debris had a significant impact on the flow hydraulics. If subjected to similar debris accumulation problems during the design flood, the structure might not withstand the flood forces. Debris accumulation on piers also subjects them to lateral stresses which can cause severe structural damage (fig. 5).

The importance of debris impact force, other than that created by ice, has been found to be of minor significance in damaging solid bridge piers. Where rocks in mountain streams are a threat, steel plates have been used to prevent abrasion of the upstream bridge surfaces. Debris impact has caused some minor damage to structures on open pile bents and to culverts.

Common Practices for Handling Debris

Emphasis has been placed on protective measures for culverts because they offer limited waterway openings and thus provide a high potential for clogging. A hydraulically smooth culvert inlet is extremely important to the efficiency of a structure. Even a small amount of debris at the inlet will significantly affect its efficiency and reduce drainage capacity. Also, the cost of installing and maintaining protective devices is low in comparison to the cost of enlarging the culvert barrel to pass anticipated debris. Protective devices include deflectors, racks, risers, cribs, fins, dams, and basins. (4)

Figure 3.—Manmade structures can become a debris problem in severe floods.



Few countermeasures for debris have been used for bridges; however, certain design features have resulted in structures which are less debris prone than others. These features include height of the bridge superstructure, pier type, and pier location in relation to span length. In most States, new bridges are constructed 0.6 m above the design flood stage, or for critical stream crossings, 0.6 m above the maximum recorded flood stage. In some States this elevation is raised to 0.9 or 1.2 m for rivers known to have debris, depending upon the nature of the problem and the properties of the anticipated debris. Piers which offer minimum protrusions, indentations, and holes are safest from debris. Some States continue to use open pile bents for economic reasons, but it is generally conceded that solid piers with smooth edges should be used in a stream with an appreciable amount of debris. Column piers with solid web walls trap less debris than column bent or open pier structures which are not protected. A cantilever or hammered pier, like a solid pier, offers minimal obstruction to passage of debris. (5)

A logical way to eliminate debris accumulation at bridge piers is to locate piers out of the main flow. In straight stream reaches debris tends to flow in the center of the river on a rising stage and nearer the banks on the falling stages. Around river bends the debris will move toward the outside of the bend. Therefore, if possible, piers on straight reaches should not be placed in the mainstream or on the outside of meander bends where debris will flow during the main flood event. In a stream known for debris potential, a longer span should be considered. A cost-risk analysis becomes very important for determining the proper span length for a debris-prone river.

Figure 4.—Massive debris pile-ups may constrict flood flow and result in scour at the bridge pier. Note damaged deck at the far end of the bridge.

Figure 5.—The lateral forces of water against a large debris accumulation caused the destruction of this bridge pier.

During a flood, maintenance can be critical to the survival of a bridge laden with debris. Most States provide crews to clean hazardous debris from bridges (fig. 6). Cranes are used to lift debris to trucks for disposal or, in emergency situations, maintenance crews may simply drop the debris on the downstream side of the bridge with the hope that it will not cause problems at another bridge. One State agency reported success in emergency situations by pushing debris away from the pier and into the mainflow where it was carried under the bridge and safely away.

Although countermeasures at bridges are not widely used, experience gained from the few existing installations is useful. There are three basic types of countermeasures that can be used: Deflectors, fins or cribs, and collectors.

Debris deflectors guide debris away from piers or other crossing encroachments. They differ from culvert debris deflectors in that they guide debris through the waterway, whereas culvert deflectors force the debris away from the waterway opening (inlet) causing debris deposition on the upstream side of the structure. The most common type of debris deflector is pile groups upstream and in front of the piers. Dolphins and fenders, constructed to protect the bridge against collision by vessels, may serve a dual purpose by acting also as debris deflectors. However, if they are not designed with the potential of debris accumulation in mind, they may actually increase the hazard.

A debris deflector was installed by the Army Corps of Engineers in the Chena River in Alaska. As a result of a study conducted to determine debris hazards to the Chena River Flood Control Dam at Moose Creek, deflectors were recommended to prevent clogging of the dam's inlet. (6) Three groups of pilings were designed to aline logs coming downstream to parallel stream flow, thus passing the debris through the structure. A model test was successfully run and the mechanism functioned properly. When tested for the natural environment, however, the deflectors did not function as well. The importance of this finding is that deflectors require flow control structures to insure stable flow conditions. The river's geometry had changed and flow no longer paralleled the orientation of debris moving off the deflectors. Similar deflectors were installed at seven highway bridges in Wisconsin. The bridges were built in densely wooded, debris-prone watersheds. Although these bridges were built in the 1930's and tested by numerous floods, to date only one bridge has failed.

Debris fins and cribs are similar to deflectors but are attached to the leading pier and guide debris to either side of the pier (fig. 7). A debris crib is any type of shielding that protects otherwise debris-prone parts of the structure. Cribs are commonly provided for open pile bents in streams known for their debris potential. The spacing between crib strips should be chosen carefully or the problem the strips were meant to solve could become worse. The space between the strips should be small enough to preclude branch entrapment and resulting debris accumulation. The crib should

Figure 6.—Crane being used to remove hazardous debris.

Figure 7.—A debris fin (foreground) and a debris crib (last pier).

also be strong enough to insure that it will not be torn loose and obstruct other debris.

Debris traps or collectors have not been used upstream from bridges, but through evaluation of other structures (not built as debris collectors but inadvertently serving this purpose) it is speculated that they would work in the right locations. The trap should be located to facilitate maintenance and disposal of collected debris. Care should be taken to be sure that after collecting debris the trap does not release a large accumulation which would float downstream and cause a bigger problem than if no countermeasure were used. This situation can be avoided by proper maintenance.

Summary

Environmental considerations, in addition to hydrology, hydraulics, and river geomorphology, play an active part when dealing with flood hazards. Debris accumulation at stream crossings is a stream hazard of considerable importance. Countermeasure guidelines are available for debris at culverts but more guidance is needed for controlling debris at bridges. With costs of bridge construction escalating, the need for improved designs and countermeasures against stream hazards are important in insuring that new structures, and those in existence, serve for as long as possible. Proper recognition of debris problems coupled with the tools needed to deal with debris will do this effectively.

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Gravity escape ramp.

Emergency Escape Ramps for Runaway Heavy Vehicles

by
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Harry B. Skinner, and Jonathan N. Young

This article summarizes the findings of the State-of-the-Practice Report on truck escape lanes which was developed by the Tennessee Department of Transportation for the Federal Highway Administration's Implementation Division. It also includes an example of the development of a truck escape ramp in the Siskiyou Mountains of southern Oregon.

Introduction

When heavy vehicles operate on steep grades, the possibility exists that a truck will lose its braking capability, go out of control at a high speed, and crash. Most highway safety engineers can relate the details of such an occurrence because it results in a rather spectacular accident.

The list of disastrous occurrences is infinite:

- An out-of-control truck fails to negotiate a curve on a steep downgrade, goes through a guardrail, over the edge of the road, and down a 90 m rock slope into a stream bed.
- A truck driver steers to the left edge of a one-way roadway in an effort to stop his vehicle against a vertical rock cut slope. The truck catches fire from the friction and explodes.
- A truck without brakes runs over a small sports vehicle that is going the speed limit.
- An out-of-control truck overturns onto a passenger vehicle it is trying to pass.

The truck escape lane is a safety design concept for coping with the problems of a runaway heavy vehicle on a grade. The first truck escape lane was designed and constructed in 1956. Since then, escape lanes have become more effective in design and operation. Improvements have come from observations of operations and their implementation in later designs, rather than from basic research.

State-of-the-Practice Report

Recently, the Tennessee Department of Transportation developed a State-of-the-Practice Report[1] for the Federal Highway Administration's Implementation Division. This contractual effort also included the development of a 26-minute slide-tape presentation[2] on the problems of truck escape lane design, operation, and maintenance; examples of successful and unsuccessful escape lanes are shown.

In the report, all emergency escape ramps were divided into three general categories: Gravity type, arrester beds, and a combination gravity type/arrester bed. Fifty-nine ramps were investigated and are described.

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[1]The report is available from the Federal Highway Administration, HDV–21, Washington, D.C. 20590.

[2]The slide-tape presentation is available on a loan basis from the Federal Highway Administration, HHI–2, Washington, D.C. 20590, from FHWA regional offices (see page 135).

Gravity type are those ramps which rely solely on an ascending grade to reduce the speed of an out-of-control vehicle. These ramps are normally hard-surfaced, 90 to 915 m long, 3.7 to 13.7 m wide, 150 to 910 mm deep, and have a +5 to +43 percent grade. The report discusses 24 ramps of this type.

Twenty-two arrester beds are described in the report (figs. 1 and 2). The sand arrester beds were 26 to 61 m long, 3 m high, and 6 m wide at the top. The gravel arrester beds were 91 to 215 m long, 5.5 to 6.1 m wide at the top, and 460 mm deep. Arrester beds required less length and, therefore, less surface area than gravity type ramps and were constructed either on an ascending or slightly descending grade.

The 13 combination ramps (fig. 3) evaluated were 618 to 670 m long, 5.5 to 15.2 m wide at the top, 305 mm deep, and had −1.5 percent to +6.7 percent grades.

Following the observations and evaluations are 13 conditions which should be met when designing and constructing an emergency escape ramp:

1. Sufficient available space. Limited space would probably dictate the use of an arrester bed. Unlimited space in the appropriate place and on the proper grade may lead to the selection of a gravity type ramp.

2. The ramp length must be sufficient to stop a design vehicle under design conditions. In a few instances, a standard impact attenuator has been used at the end of short ramps (fig. 4).

3. If an arrester bed is used, the possibility of a second vehicle using the ramp before the first vehicle has been removed should be considered. This usually requires extra width.

Figure 1.—Arrester bed ramp with embedded truck.

Figure 2.—Arrester bed ramp being constructed.

Figure 3.—Combination arrester bed and gravity ramp.

4. Access to the ramp must be adequately marked. Also, the geometrics of the ramp must be delineated in areas where snow or dust may obliterate the surface definition of the ramp.

5. Arrester beds must be very well drained to avoid frosting and freezing in cold weather. The material should not compact easily, and it should be easy to restore the original surface condition after use. If fine particles infiltrate the material, they should be removed or the entire bed replaced. Otherwise, the arrester bed will not function as a speed attenuator.

Figure 4.—Arrester bed ramp with attenuator.

6. Arrester beds should be designed in such a way that stopped vehicles will not roll back onto the main roadway.

7. Surfaced service roads should be constructed adjacent and parallel to arrester beds to allow wrecker access to vehicles embedded in the ramp. Wrecker tiedowns should be provided approximately every 90 m.

8. Advance signing should be provided for long, dangerous grades so heavy vehicle operators can check their braking system. Also, advance notice of the escape ramp is imperative.

9. The motoring public should be made aware of the purposes of escape ramps through proper signing and information displays in welcome centers, rest areas, and roadside parks.

10. The ramp design must respond to the needs of traffic and to the terrain.

11. Specific delineation is needed for ramps to discourage use by vehicles not intending to use the facility.

12. A squared-off apron should be provided for vehicles entering an arrester bed. If one front wheel drops into the sand or gravel while the other wheel remains on the hard surface of the roadway or shoulder, vehicle control will be difficult.

13. If a lighter vehicle, such as a pickup truck, uses an arrester bed and if the pea gravel is uniformly 460 mm deep, the vehicle will probably come to an abrupt halt. To overcome this situation, the depth of pea gravel should be increased from 150 to 460 mm over the first 60 to 90 m of the ramp.

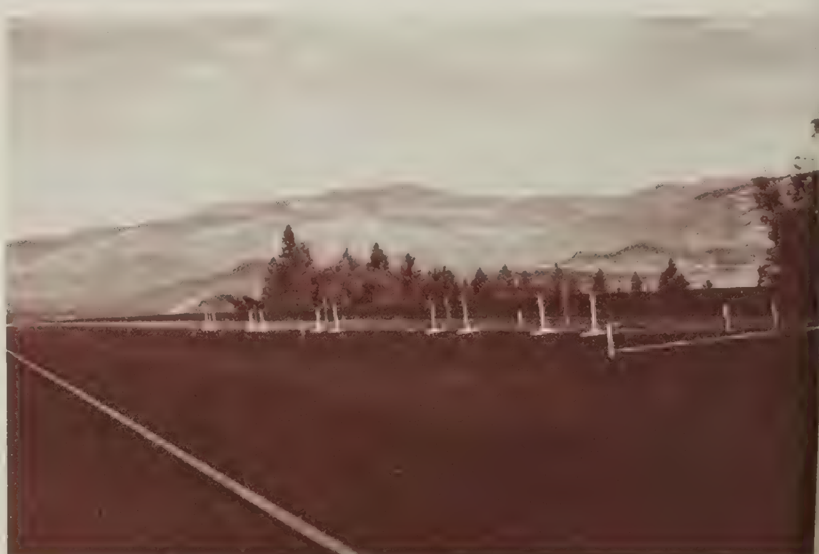
Development of the Siskiyou Truck Escape Ramp

The Siskiyou Truck Escape Ramp, on the northbound roadway of I–5 near the Siskiyou Summit in the Siskiyou Mountains of southern Oregon, reflects many of the conditions presented above. It is a descending escape ramp constructed adjacent to and on the same grade (−5.5 percent) as the mainline. The ramp incorporates a 366 m long gravel arrester bed (fig. 5).

The Oregon State Highway Division (OSHD) developed the design by extensively researching ramps in Oregon and other States, testing arrester bed materials, field testing mound configurations, and studying the roadside terrain. The ramp was constructed as part of a project on I–5 which includes many safety improvements; the ramp was opened for use in February 1978. As part of a separate research project, the truck escape ramp was tested in December 1977 in an effort to correlate escape vehicles' speeds, weights, and configurations (numbers of axles or wheels) to stopping distance and subsequent rolling friction.

The history of runaway vehicle accidents on the descending grade of I–5, north of the summit, demonstrated the need for the Siskiyou Truck Escape Ramp. The roadside was studied to find an ascending grade on which to locate an escape ramp. This section of

Figure 5.—Overall view of the Siskiyou Truck Escape Ramp with smooth arrester bed.



I-5 rests on the east slope of a mountain range running predominantly north and south. Hills rise on the left and slopes drop on the right as one proceeds downhill in a northerly direction.

The terrain negates the possibility of an ascending truck escape ramp to the right of the road. Many designs were devised, reviewed, and finally rejected. One design called for building fill to establish an uphill grade for a ramp to the right of the road. Because of the tremendous fill requirements, this design was rejected. One design called for constructing a right offramp which would lead up and over the roadway and into an uphill ramp built on the hills to the left of the road. The cost of constructing the structure over the roadway doomed this idea. Still another design would have switched the traffic flow so downhill lanes (northbound) would be on the left and uphill lanes (southbound) would be on the right. This way runaway vehicles could take a left offramp onto an uphill escape ramp built on the left side of the road. This idea, too, was discarded because of the cost of building structures to switch the traffic flow and because of traffic operations considerations.

With no feasible ascending ramp design available, a location was sought for a descending truck escape ramp incorporating a gravel arrester bed. An adequate length of roadside area to the right of the shoulder was available approximately 7.6 km north of the summit because of slide stabilization and mainline realinement work in the area. The task of designing the general ramp and the arrester bed configuration and the construction of the ramp remained.

Laboratory tests were run on two gradations of aggregate (fine material, 9.5 mm to #10; and coarse material, 19 mm to 12.7 mm) to determine the better material for an arrester bed. These tests included the Kelly Ball Test and a compression test on cylinders of aggregate saturated with water, drained, and then subjected to freezing temperature. The tests indicated that a coarse, free-draining material is preferred to a finer material in an area prone to freezing; the coarse material, when subjected to water and freezing, provides less support (compressive strength) than the finer material under similar conditions. Therefore, during freezing weather, an arrester bed of coarse material would be more likely to maintain its attenuating function than a bed of finer material which would tend to support a load better and subsequently provide less resistance and deceleration to a runaway vehicle.

Field tests of gravel mounds were then conducted on a hard pavement to study the suitability of mounds on a flat arrester bed. Mounds of various shapes, sizes, spacing, and materials were actually impacted by test trucks. The results showed that mounds are potentially dangerous and can cause vehicular damage. But because the mounds were tested on a paved surface and could possibly function differently when on a gravel arrester bed, the plan to test the mounds on the actual arrester bed was retained with added precaution to be taken during these tests. In addition, the preliminary mound tests indicated that mounds of finer material were as effective as mounds of coarse material and were less damaging to vehicles.

Based on research, testing, and field review of the location, the Siskiyou Truck Escape Ramp incorporates the following:

1. The arrester bed is 366 m long, 7.9 m wide, and 457 mm deep, composed of clean, uncrushed natural gravel of a relatively coarse gradation—12.7 mm to 19 mm.

2. Insuring quick, reliable drainage of the arrester bed was a major concern because of the ramp's location in a high elevation subject to much rain and to frost and freezing during the winter. Approximately 60 percent of the arrester bed was build on AC pavement of the old roadway (the road was realined in some areas), and the remaining 40 percent was built on new AC pavement to help drain water from the arrester bed into drainage pipes. The cutoff trenches containing the corrugated drain pipes were backfilled with sand filter material. Ponding water has indicated that the sand is too fine to transport water from the bed to the pipes quickly

Figure 6.—Smooth bed with 0.3 m high transverse mounds, 3.7 m service lane, and oversize jersey-type barrier.

enough. Possibly a larger size aggregate or a different filter material should be used. Sufficient amounts of ponded water, frost, snow, or ice could render the arrester bed ineffective.

3. A 3.7 m paved service lane separates the bed from the roadway shoulder (fig. 6). Anchors are located at 91.4 m intervals in the service lane to aid tow trucks removing runaway vehicles which become embedded in the arrester bed.

4. A paved, squared-off entry to the arrester bed is provided as is a paved return at the end of the bed in case a vehicle runs over the end of the bed.

5. To retain vehicles, the outside edge of the ramp is lined with jersey type concrete barrier, approximately 3 times normal size (fig. 6).

The tests on the escape ramp called for varying vehicle types, speeds, methods of entry, and bed surface configurations to correlate stopping distances to vehicle characteristics and to find an optimal bed configuration for safe and positive vehicular deceleration. Many interesting results were obtained. Most importantly, the tests indicated that an empty 2-axle truck (5 761 kg) entering the bed at 129 km/h should stop approximately 335 m into the bed, and a fully loaded 2-axle truck (11 975 kg) should stop in approximately 274 m. (Tests were run at speeds up to 88.5 km/h and results were extrapolated to 129 km/h.) Similar results were obtained for an empty 5-axle truck (13 064 kg) and a loaded 5-axle truck (22 317 kg) entering the ramp at 129 km/h. All cases indicated that the 366 m long arrester bed is long enough to stop runaway vehicles of the type and loading tested at speeds up to 129 km/h, provided that the bed is not frozen to a significant depth.

Figure 7.—Single-unit truck test at 40.2 km with right wheel entry.

The tests confirmed that the smooth arrester bed can decelerate vehicles safely and without damage. Mounds were tested but proved unsafe, causing bouncing, some loss of vehicle control, and potential damage to the vehicle.

A uniform entry with both front wheels entering the bed concurrently is recommended. A test of a right wheel entry with the left wheel on the paved service lane (fig. 7) resulted in the truck's right wheels immediately grabbing and pulling an entire vehicle into the bed. Some loss of driver control was experienced. However, once completely into the bed, the truck tended to straighten out and some steering was possible.

The truck escape ramp was opened for use in February 1978, although the rest of the project was not yet complete. Temporary signing advised of the ramp's availability. By the end of April, 32 vehicles had used the ramp. The reports indicated that vehicles were stopping safely although in distances somewhat less than predicted by the OSHD's tests. The OSHD is continuing to study the test data to better correlate vehicle characteristics and stopping distance. The fastest runaway truck weighed approximately 30 845 kg and entered the bed at approximately 129 km/h. It stopped 155 m into the bed.

There were a few cases of ramp misuse. The problem could be the result of inadequate warning that the ramp is for runaway vehicles only. Drivers may not fully understand what the ramp provides. Also, there might be some confusion because the escape ramp is located just ahead of a rest area. Two vehicles which became embedded in the ramp had stopped for a driver change and a brake check, respectively. Signing at this time was temporary and was to be reviewed for improvements to be incorporated during the construction of permanent signing.

Conclusion

The Siskiyou Truck Escape Ramp has proven able to safely stop runaway vehicles in the arrester bed built on a descending grade. Test results and current reports of actual use have been positive. The descending ramp was, however, considered only after the terrain and alternatives using an ascending ramp were studied and found unfeasible. The OSHD suggests that the ascending ramp be the design first considered for application once the need for an escape ramp is established. It further emphasizes that wherever freezing temperatures exist or the potential of arrester bed contamination by fine soils exists, the ascending escape ramp should be retained if at all possible.

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The CMD: A Device for Continuous Monitoring of the Consolidation of Plastic Concrete

by Terry M. Mitchell, Phillip L. Lee, and Glenn J. Eggert

Introduction

Obtaining proper consolidation of portland cement concrete during construction is a prerequisite for good quality pavements. According to a recent National Cooperative Highway Research Program Report, proper consolidation improves all of the important properties of concrete. *(1)*[1] It increases strength, durability, and resistance to freezing and thawing; improves the bond to reinforcing steel; and reduces considerably the permeability of concrete to deicing salts.

[1] Italic numbers in parentheses identify references on page 155.

Consequently, vibration to drive off the excess air and achieve optimum densification has become an integral part of paving operations. However, in order to control consolidation, highway agencies have been forced to rely on "recipe" procedural specifications which prescribe the types of vibrators, their operating frequency, and their spacing on the paver. Because these "recipes" vary considerably from State to State, an end result specification for consolidation would be desirable. However, to implement such a specification, a method for measuring the density of fresh or plastic concrete is obviously needed.

To accomplish this task, the Federal Highway Administration (FHWA), Office of Research, contracted with a company to develop a device that could be mounted on a slipform paver to continuously and automatically monitor the degree of consolidation of newly placed concrete. A prototype Consolidation Monitoring Device (CMD) was constructed and evaluated in both the laboratory and the field. This article summarizes the developmental study. *(2)*

Instrument Design and Operation

Early in the study a nuclear technique, gamma ray backscatter, was chosen for the CMD. Gamma ray techniques are already widely used by transportation agencies in commercially available, portable nuclear gages to control compaction of soil and soil aggregate in pavement structures, of bituminous concrete pavements, and, most recently, of portland cement concrete pavements and bridge decks. (*3, 4*) However, most of the commercially available density gages are used only for discrete stationary measurements and are not readily adaptable to continuous density monitoring.

Figure 1, a schematic of the source/sensor unit, explains the operation of the CMD. Gamma rays are emitted from a 500 millicurie Cesium–137 (500 mCi ^{137}Cs) source into the concrete. There they are attenuated by the concrete (by Compton scattering), and a number, in proportion to the density of the pavement, are returned to the detector.

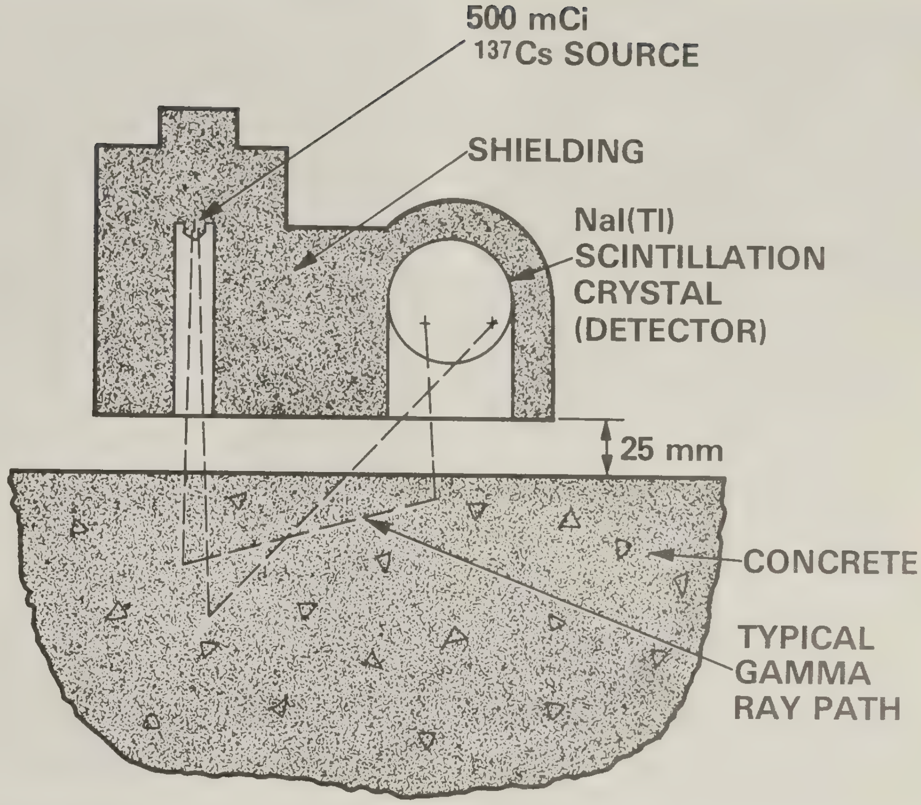


Figure 1.—Backscatter measurement with CMD source/sensor unit.

The detector consists of a thallium-activated sodium iodide, NaI(Tl), scintillation crystal 45 mm in diameter by 100 mm and a photomultiplier tube. The crystal produces a momentary light pulse for each gamma ray detected; the tube, and some associated circuitry, convert the light pulses into a d.c. voltage proportional to the rate of incoming light pulses. The d.c. voltage is used to drive a meter for continuous display of density and a strip chart recorder for a permanent density record.

The d.c. voltage that drives the density meter and strip chart recorder changes as the density of the concrete varies, but it also changes as a result of the random gamma ray emission from the ^{137}Cs source. To average some of the randomness of the source, the circuitry includes a first order time constant (resistance-capacitance). This time constant can be varied but was typically set at 3 seconds during the field trials. An important second effect of this time constant is seen when the CMD dynamically traverses the pavement surface. Because of the time constant, the concrete currently being measured as well as that previously sampled is averaged into the density readout. The longer the time constant and the faster the source/sensor traverses the concrete, the larger is the concrete volume averaged into the density reading.

The source/sensor unit is mounted and positioned on the slipform paver by a traversing mechanism (fig. 2). A carriage holds the source/sensor unit and supports it on wheels which roll on a horizontal guide beam. The guide beam is attached to the paver by three or more adjustable mounting mechanisms which provide for vertical and angular positioning of the beam. The source/sensor unit must be maintained at 25 mm (±2 mm) above the concrete surface to measure the density correctly. The carriage is moved by a 12-volt d.c. motor and gear reducer and a drive chain. The motor may be powered either by an auxiliary automotive-type battery or by the paver's 12-volt system. Once the traversing mechanism has been mounted on a paver, all of the other CMD components can be conveniently installed at the beginning of a paving day and removed at the end of the day.

All of the other electronic circuitry is included in the second major component of the CMD, the control and readout unit (fig. 3). This unit contains the signal processing

Figure 2.—Source/sensor unit and traversing mechanism.

circuitry, the meter and recorder, all of the adjustments required for calibration, high and low density alarms, and event marker pens to indicate on the strip chart recorder the transverse and longitudinal positions of the source/sensor unit over the pavement surface.

Some of the other details of the prototype's construction and operation are as follows:

- Approximate weights:
Control and readout unit—30 kg.
Source/sensor unit—85 kg.
Traversing mechanism—230 kg.
- Control and readout unit operation—30 hours from rechargeable, gelled electrolyte, lead-acid battery.
- Full battery recharge time—26 hours.
- Radiation level—maximum 40 millirem/hr on top and side surfaces of source/sensor unit.

Reference 5 gives a more detailed description of the prototype CMD's design and operation.

Laboratory Evaluation

Before the prototype CMD could be evaluated in the field, a series of laboratory tests was conducted. Test objectives included establishing the following: (1) The accuracy of the device's density readout; (2) the effect of environmental variations on the density measurements; (3) the effect of variations in geometry—for example, air gap and angle relative to the pavement—on the density measurements; and (4) the best means of calibration in the field.

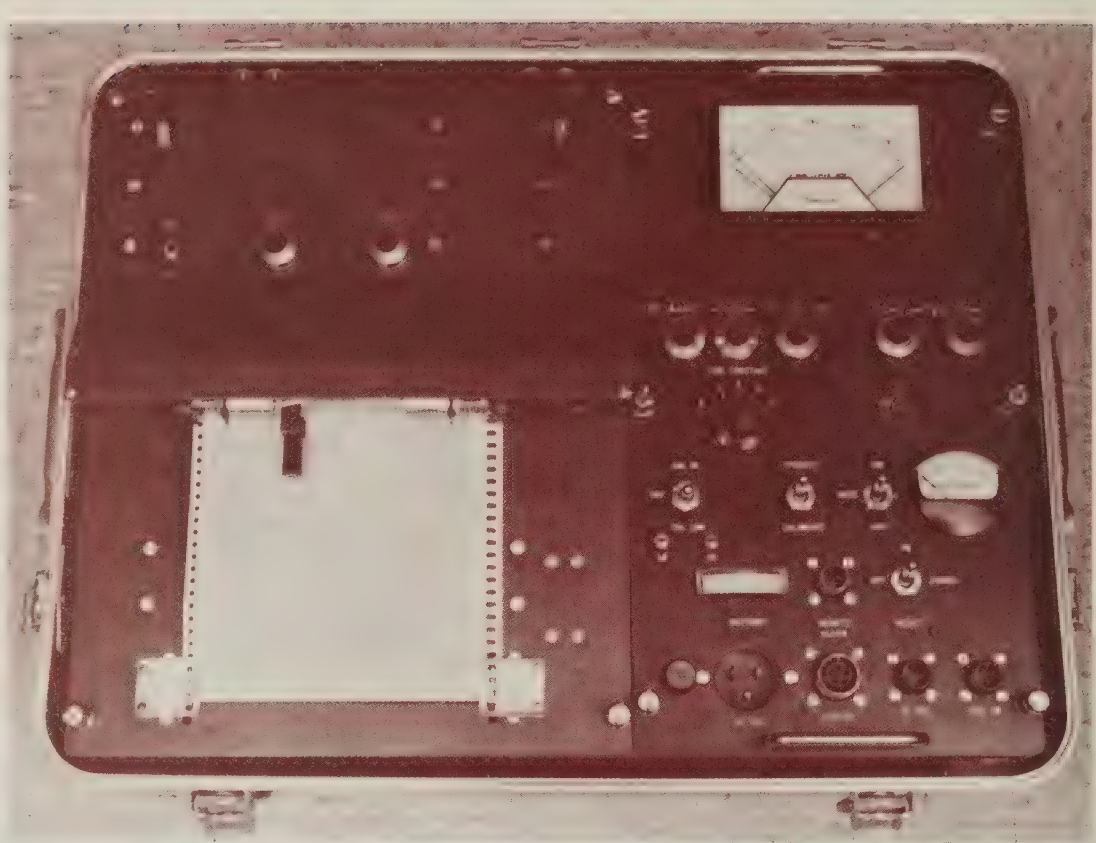


Figure 3.—Control and readout unit.

Accuracy

The CMD's "accuracy" is defined as how well the device duplicates density values obtained by conventional methods. The methods chosen for comparison were as follows: Theoretical, rodded, and vibrated unit weight; weighed density (weighing a large, known volume of concrete); core density; and commercial (static) nuclear gage density. All of the conventional methods have limitations as standards of comparison; for example, the density obtained in a unit weight container may be quite different from the bulk density obtained in a large mold during the CMD tests.

The CMD was evaluated in the laboratory in two modes: Statically, positioned above a mold containing fresh concrete; and dynamically, moving at 0.030 to 0.074 m/s across the mold to simulate a paving operation. The static test concretes included weighed densities from 2 270 to 2 490 kg/m³ and both siliceous and calcareous coarse aggregates. The dynamic test data was obtained over a 3.0 m by 1.2 m mold that was divided into three compartments at different densities—low (2 170 kg/m³), medium (2 350 kg/m³), and high (2 450 kg/m³).

Table 1 shows the mean and standard deviation values computed when the densities established by the CMD were compared with those established by several of the conventional test procedures used in the laboratory evaluations. The last conventional method listed, the combination core density, was introduced in an attempt to weight the standard core density determination to the CMD's sensitivity; CMD response depends more on the density of the concrete near the surface than on the density farther down. Although the standard deviation is the more meaningful of the two statistical parameters, both show that the results from the CMD correlate very well with the results from other tests. Statistical results also show that the standard deviations are larger in the dynamic tests than in the static tests.

The mean difference results were supplemented by regression analyses. Figure 4 shows the CMD density data plotted as a function of the core density for all of the laboratory tests (static plus dynamic). The regression line, shown with the 95 percent confidence limits for its location, can be used to predict core density values for a particular CMD value. Detailed examination of the regression analysis data led to the conclusion that the CMD was capable of duplicating core density measurements within a ±35 kg/m³ confidence range (95 percent confidence limit).

Other laboratory results

Additional conclusions from the laboratory study included the following:

- When a siliceous coarse aggregate was substituted for calcareous (in a limited number of tests), the chemical change did not noticeably influence the density readout. Some commercial nuclear backscatter gages have shown as much as a 100 kg/m³ difference between responses from samples of the two different materials compacted to equal densities.
- Variations in temperature have little impact on the accuracy of the density readout. A change in ambient temperature from 6° C to 39° C caused less than one-quarter of 1 percent change in the recorded density.
- The air gap between the sensor and the concrete surface affects the CMD's readout. A 3 mm variation from the normal 25 mm air gap caused approximately a 25 kg/m³ change in the displayed density.
- Reinforcing steel in typical quantities and at typical depths in reinforced pavements has little effect on the CMD's readout. Density changes caused by a 150 mm square mesh of #6 (4.9 mm diameter) reinforcing wire with a 50 mm cover were not perceptible in the CMD readout. During the subsequent field trials, #6 reinforcing rod (19 mm) with 80 mm of cover (nominally) in a continuously reinforced concrete pavement did not noticeably affect the readout.

In addition to the research study data reported here, an investigation was made at the Fairbank Highway Research Station in McLean, Va., to establish the CMD's depth sensitivity, that is, the contribution of the concrete at various depths to the CMD readout. Table 2 shows the results of those tests which were

Table 1.—Laboratory comparisons of CMD with conventional test results

CMD as compared with:	Static tests			Dynamic tests		
	n	Mean difference[1]	Standard deviation[2]	n	Mean difference[1]	Standard deviation[2]
		kg/m³	*kg/m³*		*kg/m³*	*kg/m³*
Weighed density	36	−12.5	15	—	—	—
Vibrated unit weight	33	7.9	19	72	−16.5	24
Commercial nuclear gage density	34	−9.3	14	58	−27.2	41
Core density	34	3.6	13	58	−6.7	19
Combination core density (75 percent from top half of core, 25 percent from bottom half)	34	8.5	13	58	−7.5	20

[1] Mean difference $= \dfrac{\sum_{i=1}^{n} (\rho_{i_A} - \rho_{i_B})}{n}$

where ρ_{i_A} and ρ_{i_B} = densities by methods A and B, respectively.

[2] Standard deviation $= \left[\dfrac{\sum_{i=1}^{n} (\rho_{i_A} - \rho_{i_B} - \text{Mean difference})^2}{n-1} \right]^{1/2}$

based on combinations of 1 780 kg/m^3 magnesium sheet and 2 780 kg/m^3 aluminum sheet. This depth sensitivity is significantly better than that obtained with the commercial static backscatter density gages.

Calibration procedure

A procedure involving initial calibration in the laboratory with plastic (fresh) concrete samples and subsequent periodic recalibration in the field with a magnesium /aluminum (Mg/Al) block was developed and used throughout this research study. The laboratory calibration is accomplished on two plastic concrete samples, one high density and one low. The CMD is adjusted initially to indicate the correct densities of the two concrete molds. The device is then placed on the Mg/Al block, and a density measurement is obtained (low density). This value is recorded, and the measurement is then repeated with a 3 mm steel plate on top of the Mg/Al block (high density). These two densities are retained and, whenever calibration is necessary in the field, the source/sensor unit of the CMD is returned to the block and is readjusted until the two density values are duplicated.

Field Trial

Because the laboratory evaluations indicated that the CMD had satisfactory accuracy and reliability, a field test was undertaken during the 1977 construction season. With the cooperation of the Wisconsin Department of Transportation, a site was selected near Rice Lake, Wis. The pavement, an upgrading of U.S. 53, was 7.3 m wide in each direction, 200 mm thick, and continuously reinforced with 19 mm diameter steel rod. Rods were placed on 230 mm centers and approximately midway in the pavement depth. The pavement had a uniform crown of 16 mm/m, with the section gradually reduced to a flat in the curves.

The traversing track for the prototype CMD was designed to allow maximum transverse travel of 3.7 m. On the Rice Lake project, this distance was reduced to 3.0 m because of the location of certain components of the slipform paver and the angle the paver operator used on the extrusion meter.

Figure 5 is an artist's rendition of a short segment of the actual CMD recorder chart output from the Rice Lake project. As the CMD sensor traverses the pavement, information on density and longitudinal and lateral position are inscribed on the chart paper. The longitudinal position information comes from a small wheel that travels on the ground or the paver track and actuates a limit switch which causes a mark to be made on the chart for each revolution of the wheel (0.95 m). The position of the transverse indicator pen changes when the source/sensor unit reaches either end of the guide beam. The density readout in the figure is

Table 2.—Effect of sample depth on CMD response

Depth	Estimated contribution to response
mm	*Percent*
0–25	37
25–50	35
50–75	17
75–90	6
Over 90	5

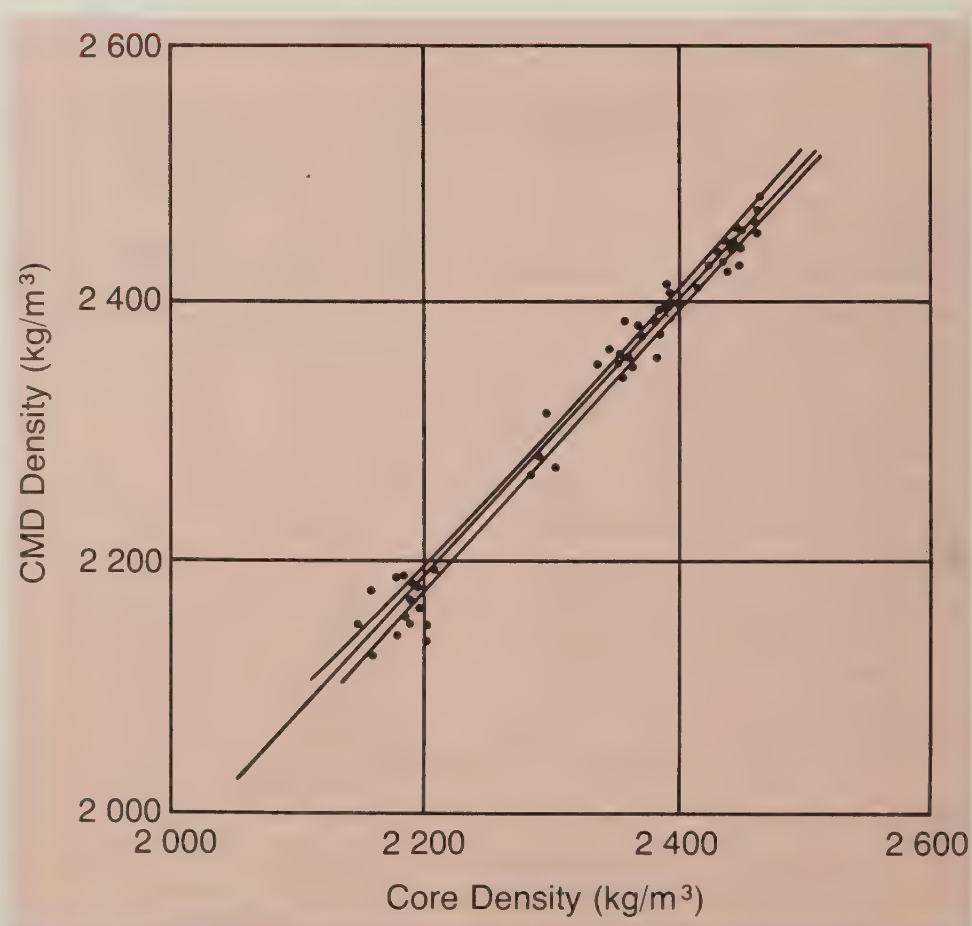


Figure 4.—Correlation between CMD density and core density measurements, laboratory static and dynamic tests (regression line and 95 percent confidence limits).

averaged with a 3-second time constant.

As in the laboratory study, conventional density measurements were made for comparision. These conventional procedures included rodded unit weight, and, after the pavement hardened, core densities and commercial nuclear gage densities. Again it should be noted that conventional tests do not necessarily measure the same volume of material that the CMD measures.

Field Results

Table 3 summarizes the comparison of the CMD results with the various conventional density measurements. Correlation was good between the results from the CMD and results from the other procedures. The regression analysis showed that the largest difference between a CMD reading and the corresponding combination core density (75 to 25 percent) was 53 kg/m³ at a CMD density of 2 494 kg/m³.

In addition to the comparison data, the CMD records produced considerable information about the quality of the concrete. One area of interest was the effect of the internal vibrators on the density of the finished pavement. By turning the CMD traversing motor on and off, the source/sensor unit was statically positioned at various transverse positions. Thus, it was possible to generate density values at and between the internal vibrators.

The density profile across a width of pavement is shown in figure 6. The location of six vibrators relative to the pavement edge is shown, in section view, in the upper half of the figure. The vibrators were spaced from 380 to 560 mm apart. The graph below the section shows the densities at and between these six vibrators. The average density at the vibrators was 2 415 kg/m³, as indicated by the upper broken line. The average density between the vibrators was 2 380 kg/m³, as indicated by the lower broken line. Thus, the average difference was 35 kg/m³. The closest vibrators were #3 and #4; the densities at these vibrators and between them were almost identical. Data of this type would be of considerable value to both the contractor and the contracting agency in establishing vibrator patterns and frequencies and in detecting malfunctioning vibrators.

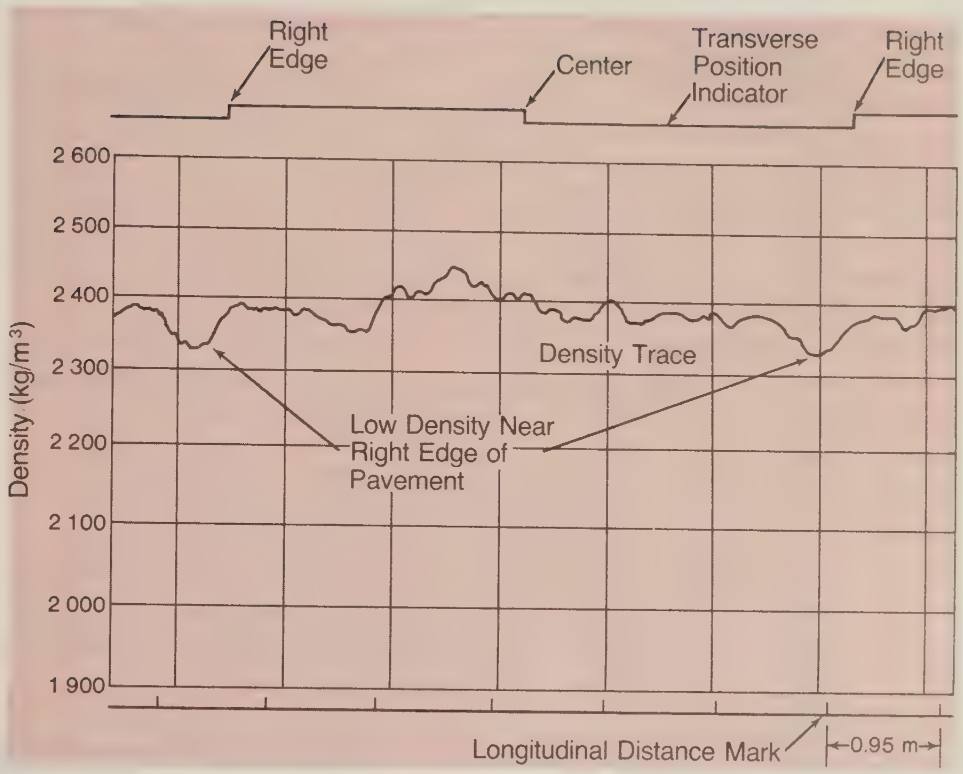


Figure 5.—Typical density trace, source/sensor unit traversing pavement.

Another observation that was made during the field tests concerned the density near the edge of the pavement. The strip chart output in figure 5 shows a significant drop in density near the right edge of the pavement. This phenomenon occurred only occasionally and for unexplained reasons, but showed up for several successive transverse cycles. Some of the roughness in the density trace is attributable to the randomness of gamma ray emission noted earlier, but the larger variations represent changes in the concrete itself. At the point nearest the edge, the distance from the source/sensor unit to the edge was still larger than the minimum established in the laboratory tests for accurate measurement. The 80 kg/m³ decrease in the CMD reading lasted for approximately 1 000 mm of total CMD travel or 700 mm of transverse travel. Because the low density area occurred at the end of the traverse, the source/sensor unit passed over the low density area first in one direction and then immediately afterward in the opposite direction. This means that the low density area was half of 700 mm, or approximately 350 mm wide. The source/sensor unit speed (paver plus transverse) was about 0.085 m/s.

During the field trials, 6 to 8 km of pavement were monitored. The density as measured by the CMD ranged from 2 320 kg/m³ to about 2 490 kg/m³. For the most part, these variations were substantiated by cores and by static nuclear density measurements made with a commercial gage. The exact reasons for the variation in density are unknown. The most likely possibility was that the degree of consolidation changed. This was brought out by the analysis of densities at and between vibrators. A second possibility was that concrete mix proportions changed somewhat because of changes in moisture content and other variables. Some of the variation may also be attributed to changes in entrained air content. At one point, a marked increase in density was observed for about 30 seconds; later, it was found that there had been a problem with the equipment that adds air-entraining agent. There were also slight variations in density from morning to afternoon. These may have been caused by loss of moisture or by differences in hauling distance from the batching site to the paver.

Additional Field Trials

In 1978, after completion of the original research contract, the Iowa and Illinois Departments of Transportation, in cooperation with FHWA's Office of Development, evaluated the CMD on paving projects. The State highway agencies have been invited to try the device in the field, both to further evaluate its usefulness and to study the effects of various slipform paving practices on consolidation. Results of these additional field trials will be reported in 1979.

Summary and Conclusions

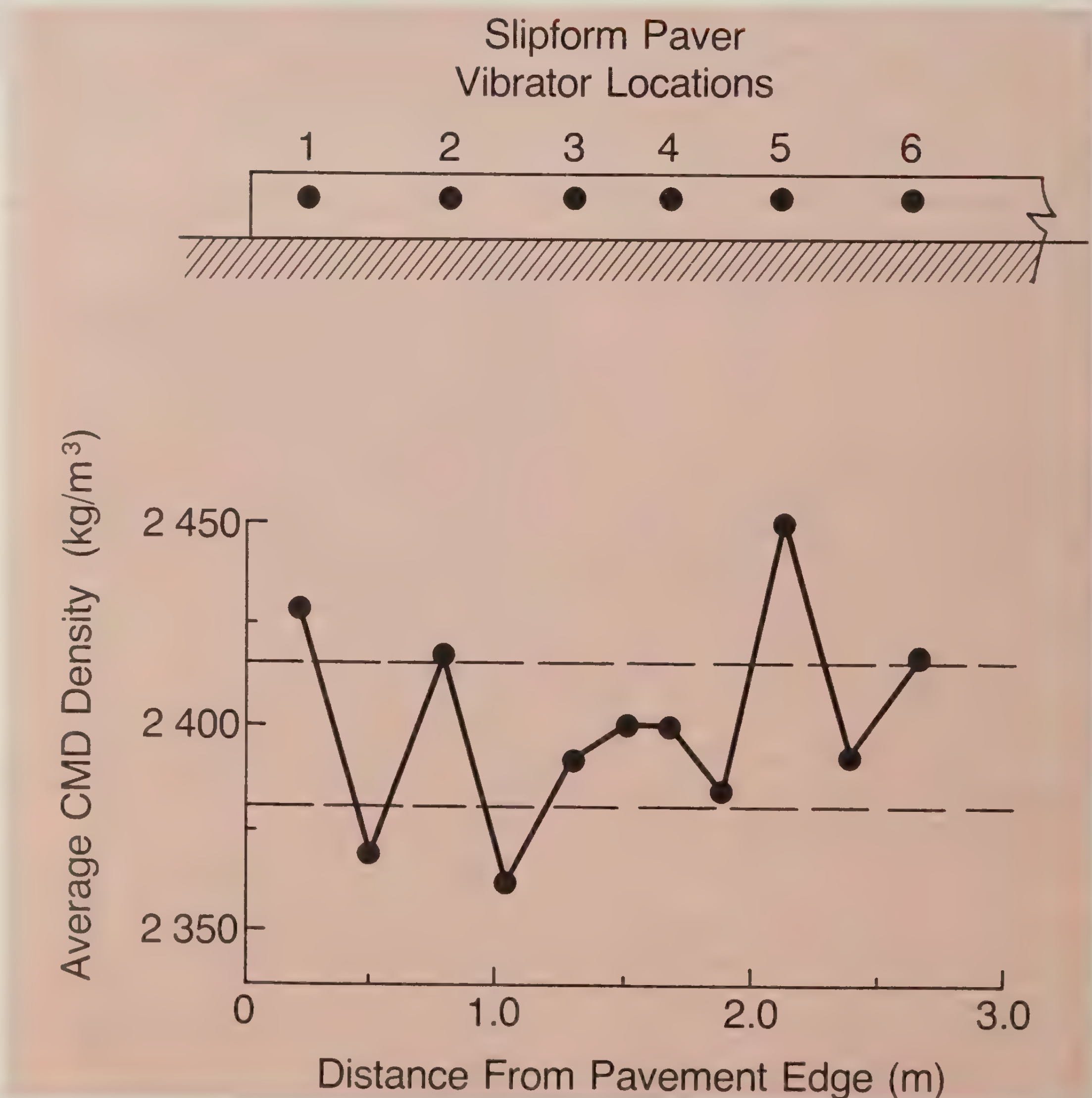
A nuclear backscatter device to continuously and automatically monitor the degree of consolidation of plastic concrete has been developed. Laboratory and field evaluations indicate the CMD is capable of sensing and displaying changes of density while traversing

Table 3.—Field comparisons of CMD and conventional test results[1]

CMD as compared with:	n	Mean difference	Standard deviation
		kg/m³	*kg/m³*
Commercial nuclear gage density	31	0.3	30
Core density	30	−1.4	31
Combination core density (75 percent from top half of core, 25 percent from bottom half)	30	−1.4	23

[1] See footnote to table 1 for definition of statistical parameters.

Figure 6.—Effect of vibrators on density.



the pavement surface at speeds up to 0.085 m/s. The device can duplicate core density measurements within a 35 kg/m^3 confidence range (95 percent confidence limits).

FHWA's Offices of Research and Development believe the CMD offers highway agencies and contractors a valuable new tool for controlling the quality of concrete pavement as it is being placed. Better control of consolidation should result in increased durability and better overall quality in new portland cement concrete pavements. Moreover, contractors can be allowed greater flexibility and efficiency in the operation of their pavers if method specifications can be replaced by end result density requirements. Also, remedies such as revibration can be applied when problems such as vibrator malfunction occur.

REFERENCES

(1) "Consolidation of Concrete for Pavements, Bridge Decks, and Overlays," NCHRP Synthesis No. 44, *Transportation Research Board,* Washington, D.C., 1977.

(2) P. L. Lee and G. J. Eggert, "Development of a Device for Continuous Automatic Monitoring of Consolidation of Fresh Concrete," Report No. FHWA-RD-78-27, *Federal Highway Administration,* Washington, D.C., March 1978.

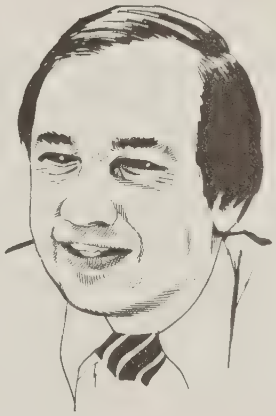
(3) B. B. Gerhardt, "The Effect of Vibration on the Durability of Unreinforced Concrete Pavement," Report No. CDOH-P&R-R&SS-75-2, *Colorado Division of Highways,* Denver, Colo., June 1977.

(4) G. Calvert, "The Iowa Method of Bridge Deck Resurfacing," *Concrete Construction,* vol. 23, No. 6, 1978, pp. 323-328.

(5) P. L. Lee and G. J. Eggert, "Instrument Manual for Prototype Concrete Consolidation Monitoring Device," Report No. FHWA-RD-78-96, *Federal Highway Administration,* Washington, D.C., May 1978.

Terry M. Mitchell is a materials research engineer in the Materials Division, Office of Research, Federal Highway Administration. He joined FHWA in 1971 and currently is the project manager for the Federally Coordinated Program of Highway Research and Development Project 4F, "Develop Rapid and More Significant Test Procedures for Quality Assurance."

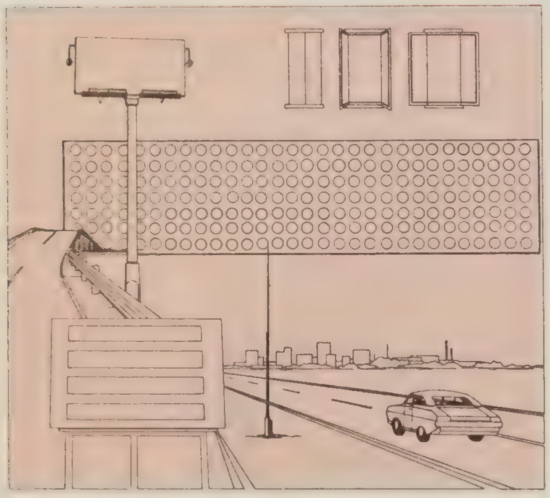
Glenn J. Eggert has recently become Manager—Special Equipment Engineering and Development, Allen-Bradley Company. Previously, he was assistant director of the Product Development Center, Rexnord Inc., where his responsibilities included managing new product development activity and contract research in such fields as material handling, construction equipment, power transmission components, and instrumentation and controls.



Phillip L. Lee is a research engineer in the Product Development Center, Corporate Research and Development Group, Rexnord Inc. For the past 13 years he has been involved with the development of products for power transmission, material handling, and process control. His contributions have been principally in the area of magnetic devices, electric controls, and instrumentation. He is presently engaged in developing and applying automatic techniques to aggregate sieve analysis.

Recent Research Reports You Should Know About

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



Human Factors Requirements for Real-Time Motorist Information Displays, Volume 1: Design Guide (Report No. FHWA-RD-78-5)

by FHWA Traffic Systems Division

This report gives practical guidelines for the development, design, and operation of visual and auditory driver displays for freeway traffic management. The emphasis is on the recommended content of messages to be displayed in various traffic situations; the manner in which messages are to be displayed—format, coding, style, length, load, redundancy, and number of repetitions; and the location of the messages with respect to the situations they are explaining. The guidelines for visual and auditory messages to be displayed in incident management/route diversion situations are based on research and operational experience.

This report is concerned with human factors design considerations with respect to motorist information displays for traffic management in freeway corridors. Once the decision to implement a motorist information system has been made, the report should provide guidance in message selection and the manner of message presentation for specific situations. Although the report is primarily intended for traffic engineers working in city, county, State, or private organizations, it should be useful to traffic engineering students or trainees and to FHWA engineers at the district and regional levels who are responsible for project review and approval.

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

Bicycle-Safe Grate Inlets Study, Volume 2: Hydraulic Characteristics of Three Selected Grate Inlets on Continuous Grades (Report No. FHWA-RD-78-4)

by FHWA Environmental Design and Control Division

This report describes the additional hydraulic tests conducted on three selected grates of 0.38 m and 0.91 m widths on continuous grades. These grates, the curved vane, the parallel bar with transverse rods, and the parallel bar with spacers, were identified in Volume 1 (Report No. FHWA–RD–77–24) as possessing best overall qualities for bicycle safety, hydraulic efficiency, and debris handling characteristics.

From the design curves presented in this report and those presented in Volume 1, hydraulic efficiencies can be estimated for three grates of any size within the widths 0.38 m to 0.91 m and lengths of 0.61 m to 1.22 m. Numerous design curves are presented to aid the hydraulic design engineer with grate inlet selection.

Subsequent reports will cover the results of tests on these three grates at the low point of a sag vertical curve (sump condition) and on the design of slotted drains.

The report is available from the Environmental Design and Control Division, HRS–42, Federal Highway Administration, Washington, D.C. 20590.

Runoff Estimates for Small Rural Watersheds and Development of a Sound Design Method, Volume I (Report No. FHWA–RD–77–158), Volume II (Report No. FHWA–RD–77–159), and Volume III (Report No. FHWA–RD–77–160)

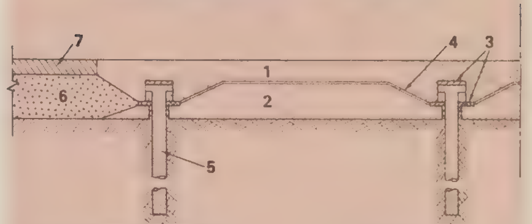
by FHWA Environmental Design and Control Division

The research described in these volumes led to the development of a method for determining flood frequency for ungaged rural watersheds smaller than 130 km². The method is based on analyses of data from more than 1,000 small watersheds throughout the United States and Puerto Rico. The method relates the runoff peak to easily determined hydrophysiographic parameters. After beginning with a seven-parameter method, it was found that three parameters, namely area, rainfall erosivity factor, and difference in elevation from the top to bottom of the watershed, produced peak flow estimates of virtually equal reliability. Consequently, the three-parameter equations were recommended for design purposes, and nomographs for solving the equations were developed for each hydrophysiographic zone of the United States and Puerto Rico.

Volume I, **Research Report,** and Volume II, **Recommendations for Preparing Design Manuals and Appendices B through H,** are intended for general use; Volume III, **Appendix A,** containing plots of annual flood frequency curves for each watershed, is intended primarily for those wishing to verify equations or develop new equations.

The reports are available from the Environmental Design and Control Division, HRS–42, Federal Highway Administration, Washington, D.C. 20590.

Pile-Supported Pavement



Unique Concepts and Systems for Zero-Maintenance Pavements, State of the Art, Report No. FHWA–RD–77–76

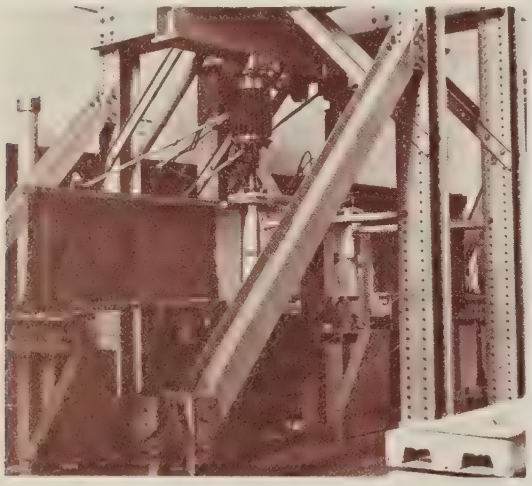
by FHWA Structures and Applied Mechanics Division

This report describes a research study which identified eight innovative concepts having potential as Zero Maintenance (Z-M) pavements. For this study, Z-M pavements were defined as those which will provide at least 20 years of maintenance-free service on high-volume expressways and at least 10 additional years with only routine maintenance before major rehabilitation is required.

The report proposes new ideas and materials for subbase and base course construction, composite pavements, prefabricated elements, and pavement joints. Syntheses of research findings on the individual pavement subsystem components (subgrade, subbase, base, and surface) were used to develop the eight pavement systems that may satisfy Z-M requirements for strength, durability, and acceptable performance.

These pavement systems may be grouped in three general categories: Those which improve conventional pavement designs through use of techniques for strengthening and protecting the subsystem components, those in which the mechanisms for stress distribution and load transfer to the subgrade are modified, and those in which the vehicular load forcing functions and the mechanism of load transfer to the subgrade are both changed. Additional research on this subject is recommended.

This report is available from the National Technical Information Service, 5285 Port Royal Rd., Springfield, Va. 22161 (Stock No. PB 273110).



Determination of Tolerable Flaw Sizes in Full-Size Welded Bridge Details, Report No. FHWA-RD-77-170

by FHWA Structures and Applied Mechanics Division

This report summarizes the findings of a laboratory and analytical investigation of the fatigue and fracture resistance of full-size welded steel bridge members which incorporate such commonly used details as lateral attachments, cover plates, flange transitions, transverse stiffeners, and gusset plates. The study to determine the tolerable flaw size of typical full-size weldments used in bridges was prompted by recent fractures of steel bridge members in the United States and the current practice of designing welded bridge details with thick, high-strength steel plates.

Twenty-four full-size welded beam specimens were cyclically loaded at room temperature for a minimum of 2 million cycles and then at temperatures of −40° C and lower until rapid fracture occurred. The structural details were chosen from the American Association of State Highway and Transportation Officials' categories for fatigue design of steel bridge members.

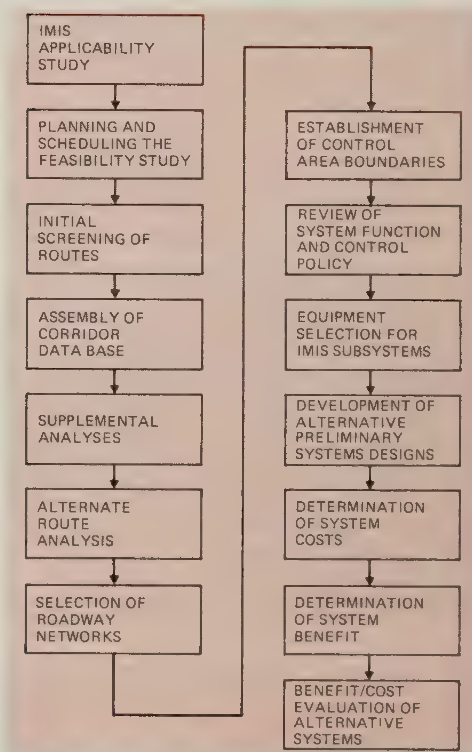
The cover plate and the lateral attachment are the two Category E details used. The transverse stiffener is an intermediate Category C detail. The flange thickness transition, a Category B detail, provided a measure of the upper bound fatigue strength. Six beams were fabricated for each of the four detail categories. Each detail type was fabricated in three American Society for Testing and Materials grades of steel, A36, A588, and A514. The fatigue test results were correlated with available data from previous tests with smaller beams, and only the cover-plated beams exhibited a dimensional effect when compared to previous fatigue studies.

A characterization of the materials used in the fabrication of these beams was made by several fracture toughness tests. The fracture resistance of each beam was estimated using the Linear Elastic Fracture Mechanics Theory and was compared to the material fracture toughness test results. The smallest crack size which is stable in a particular weldment at the lowest service temperature must be determined in order to establish the inspection sophistication and the inspection frequency necessary to insure the fracture safety of structures.

The results of this study confirm the need for adequate fatigue design in any fracture control plan for steel bridge members.

This report is available from the National Technical Information Service, 5285 Port Royal Rd., Springfield, Va. 22161 (Stock No. PB 281877).

Integrated Motorist Information System (IMIS) Feasibility and Design Study, Phase II: Generalized Methodology for IMIS Feasibility Studies.—Volume 1 (Report No. FHWA-RD-78-23) and Volume 2 (Report No. FHWA-RD-78-24)



by FHWA Traffic Systems Division

As traffic demands continue to increase and economic and social pressures prevent major new roadway construction, particularly i urban areas, the need for more efficient use of existing highway networks becomes more important. The IMIS attempts to use present state-of-the-art equipment and real-time traffic-responsive control techniques for effective management of traffic in an existing highway corridor. By combining individual remedial measures in an integrated design, a more cost-effective system can be provided.

The first phase of IMIS concentrated on determining the feasibility of implementing an IMIS in the Northern Long Island corridor of New York State. The second phase, which is covered in these reports, generalizes the methodology for performing an IMIS feasibility study.

In Volume 1, **IMIS Feasibility Study Handbook,** the methodology is put into handbook form to provide practicing traffic engineers with a step-by-step procedure for accomplishing such a study. Volume 2, **Validation and Application of Feasibility Study Handbook,** gives the results of applying the Handbook to a freeway corridor in California. It is a complete example of an IMIS feasibility study.

Major topics of this study include criteria for IMIS applicability, assembly of necessary base data, selection and evaluation of candidate routes for control, equipment trade offs, development of alternative preliminary designs, and system evaluation on a benefit/cost basis. Costs and trade offs are considered for the selection of traffic surveillance and control subsystem elements and techniques. Design related topics include selection of communications media, alternate routes, diversion points, and ramp metering locations. This study should be of interest to those concerned with design of freeway surveillance and control systems.

Phase III of this study will result in the actual design of an IMIS for the corridor studied in Phase I.

These reports are available from the Traffic Systems Division, HRS-32, Federal Highway Administration, Washington, D.C. 20590.

Evaluation of Warning and Information Systems: Part I, Changeable Message Signs, Report No. CA-DOT-07-3130-1-75-5

by California Department of Transportation

Methods must be available to communicate traffic condition information to drivers for a freeway control system to be effective. In recent years, variable or changeable message signs (CMS) have been used for this purpose. The actual usefulness of this information and its impact on both the driver and traffic flow have been the subject of much discussion.

The California Department of Transportation, in cooperation with the Federal Highway Administration, evaluated a system of 35 CMS's on the Santa Monica Freeway in Los Angeles, Calif. The signs were installed along the freeway and activated through use of leased telephone lines via a minicomputer. Most messages were precoded into memory and specifically selected by the system operator. The real-time, electronic traffic surveillance system in place at the site provided incident detection, end-of-queue location, approach speed and travel time through a congested area, and estimated duration of congestion.

The evaluation report states that a majority of drivers found the CMS's useful. The message "Slowing Ahead" led drivers to reduce their speed about 4.8 km/h, and decelerations approaching downstream congestion were less severe. Sign messages warning of lane blockages induced lane changing away from the blocked lane. Signing also increased diversion at offramps that were greater than 0.8 km upstream from congestion-producing incidents. An important finding was that injury and fatal accidents were reduced by 17 percent with the use of CMS. The report also recommends that the CMS spacing can be increased from the 1.2 km sign spacing used on the Santa Monica Freeway.

The report is available from the National Technical Information Service, 5285 Port Royal Rd., Springfield, Va. 22161 (Stock No. PB 266102).

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

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

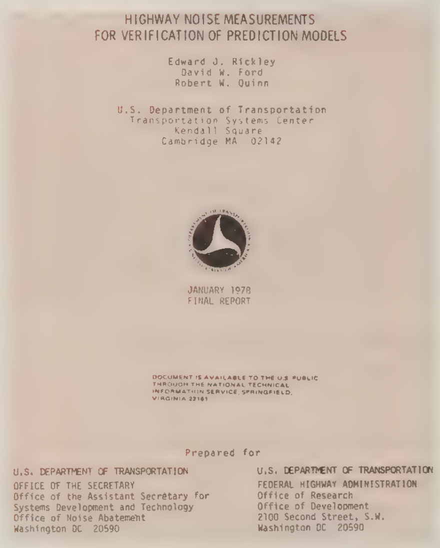
U.S. Department of Transportation
Federal Highway Administration
Office of Development
Implementation Division, HDV-20
Washington, D.C. 20590

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

Highway Noise Measurements for Verification of Prediction Models, Report No. DOT-TSC-FHWA-78-1

by U.S. Department of Transportation's Transportation Systems Center

This report includes the compilation of the raw data obtained during noise measurement projects in four States: North Carolina, Florida, Washington, and Colorado. The projects were initiated because of the wide differences in the results being obtained between field


HIGHWAY NOISE MEASUREMENTS
FOR VERIFICATION OF PREDICTION MODELS

Edward J. Rickley
David W. Ford
Robert W. Quinn

U.S. Department of Transportation
Transportation Systems Center
Kendall Square
Cambridge MA 02142

JANUARY 1978
FINAL REPORT

DOCUMENT IS AVAILABLE TO THE U.S. PUBLIC THROUGH THE NATIONAL TECHNICAL INFORMATION SERVICE, SPRINGFIELD, VIRGINIA 22161

Prepared for

U.S. DEPARTMENT OF TRANSPORTATION
OFFICE OF THE SECRETARY
Office of the Assistant Secretary for Systems Development and Technology
Office of Noise Abatement
Washington DC 20590

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
Office of Research
Office of Development
2100 Second Street, S.W.
Washington DC 20590

measurements and the prediction methods for the two approved highway noise models, the Transportation Systems Center Model and the National Cooperative Highway Research Program Model 117.

Each State chose four measurement sites according to prescribed guidelines. Measurements were made over a 24-hour period at one site in each State to provide a history of traffic noise and to show corresponding changes in noise levels relative to changes in traffic volume, speed, and mix. Three 50-minute continuous measurements were performed at the other three sites in each State. Individual truck pass-by events were also recorded and categorized. All continuous measurement data have been reduced to provide statistical noise indexed in 10-minute intervals and a composite 50-minute segment.

The data obtained from this measurement program and presented in this report expand the existing highway traffic noise base. The State highway agencies and planning groups use the noise data to validate and modify the computer-based noise prediction models currently being used. This noise base information also provides the empirical data necessary to correct any prediction inaccuracies in these existing methods.

The report is available from the Implementation Division. The data contained in the report is also available on computer magnetic tape; copies are available from the Implementation Division.

Control Strategies for Signalized Diamond Interchanges (Report No. FHWA-TS-78-206) and Executive Summary (Report No. FHWA-TS-78-207)

by FHWA Implementation Division

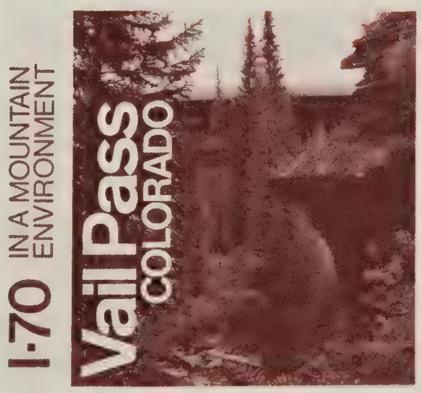
This study was made to determine the comparative efficiency of five signal timing control methods: Traffic Engineering Method, FHWA Design Manual Method, Fully Actuated Operation Method,

Microprocessor With Lagging Left-Turn Method, and Microprocessor With Leading Left-Turn Method. A before-and-after evaluation of these methods was made by using time-lapse photography to supply vehicle data. The site selected for application of these comparisons was the diamond interchange of Interstate 95 and Golfair Boulevard in Jacksonville, Fla.

Evaluation results indicated that the most efficient method of signal control would be the microprocessor which would allow an intersection to operate under actuated control during certain periods of the day and under predetermined timing patterns during other periods. It should be noted that the study procedure is appropriate for signal timing studies made at diamond interchanges, at other interchange configurations with a ramp terminal and nearby signalized intersections, and at a two-way major street intersected by one or more one-way street pairs.

The reports are available from the Implementation Division. A 10-minute slide-tape presentation covering the technical details of this project is available from FHWA division and regional offices (see page 135).

I–70 In a Mountain Environment, Vail Pass, Colorado, Report No. FHWA–TS–78–208



by Colorado Division of Highways and FHWA Implementation Division

This report documents the ideas, designs, and construction techniques used in the planning, design, and construction of Interstate Highway 70 over Vail Pass, one of the highest locations of an Interstate Highway in the United States. The report also describes how the challenge of fitting an Interstate Highway into a sensitive and scenic mountain environment was met, how the decision to build along existing U.S. Route 6 provided an opportunity to repair the scars on the landscape made during the construction of U.S. 6, and how commitments made in the environmental impact statement were carried out in the design and construction of I–70. The full-color report contains many photographs and will interest both the highway professional and the layperson. It can be a source of ideas for solutions to environmental problems that may be encountered throughout the country.

The report is available from the Implementation Division.

Superintendent's Manual on Compaction, Training Aid Series 12

by National Asphalt Pavement Association

There is evidence that inadequate densification of asphalt pavements during construction can substantially reduce their performance life. The need for training in this important construction phase has been recognized in many areas of the country. This report discusses why asphalt pavement should be compacted and how this should be done. Because the procedures used for compacting the conventional mixes are quite different from those used for open-graded mixes, the procedures are treated separately in the report. The report is written to be readily understood by highway agency paving inspectors.

The report is available from the Implementation Division.

Lime Fly Ash Stabilized Bases and Subbases

by FHWA Construction and Maintenance Division, Implementation Division, Materials Division, and Highway Design Division

This 20-minute slide-tape presentation explains the use of lime fly ash aggregate (LFA) mixtures in base and subbase construction. Fly ash, a powdery byproduct of coal combustion, is usually recovered from flue gases in electric power generating plants. It can be combined with lime, water, and aggregates to form cement mixtures. The presentation summarizes material characteristics and testing, design, and construction procedures. It is primarily directed to personnel in highway agencies who administer or participate in materials testing, design, construction, or maintenance. The presentation may also interest environmentalists who support the substitution of potential waste materials, such as fly ash, for potentially scarce materials, such as cement or asphalt.

The slide-tape presentation is available from FHWA National Highway Institute and FHWA regional offices (see page 135).

Comparison of Ohio State University and Penn State University Skid System Water Nozzles, Report No. FHWA-RD-78-503

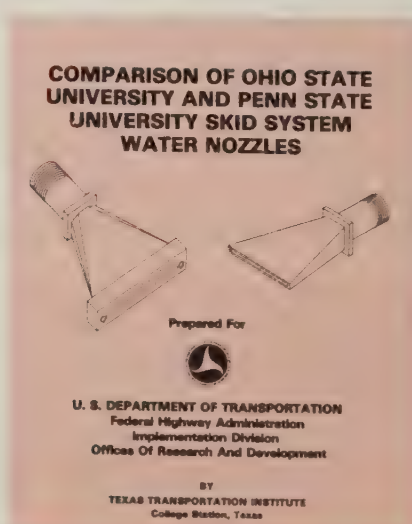
by FHWA Implementation Division

For some years, most skid measurement systems have used a watering nozzle known as the Pennsylvania State University (PSU) design. Due to the divergent nature of the PSU nozzle, it has been difficult, if not impossible, to adjust the water flow rate from the nozzle so that it will satisfy the American Society for Testing and Materials (ASTM) Standard E274-77 for pavement wetting at all speeds. In particular, if the water flow rate is adjusted properly at 64 km/h it will be out of specification at 32 km/h and/or 97 km/h.

The Ohio State University (OSU) Engineering Experiment Station has developed a working model of a nondivergent nozzle from a previous concept by the State of Virginia. This nozzle was compared to the PSU nozzle for accuracy, precision, and ease of calibration to the ASTM Standard. This report documents the findings. There were no significant differences in the measured skid number (SN) accuracy and precision at 32 km/h and 64 km/h. However, at 97 km/h the SN measurement using the PSU nozzle averaged more than 1 over those obtained when using the OSU nozzle. This significant difference indicates the effect of flaring out the water trace by the divergent nozzle and thus placing less than normal water thickness ahead of the test tire. The adjustmen of water flow rate of the OSU nozzl was easily accomplished at 32 km/h, 64 km/h, and 97 km/h.

This report will be of interest to those involved in the pavement skic resistance inventories, the operatior and the maintenance of skid measurement systems.

The report is available from the Implementation Division.



New Research in Progress

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

FCP Category 1—Improved Highway Design and Operation for Safety

FCP Project 1A: Traffic Engineering Improvements for Safety

Title: No Passing Zone Treatments for Special Geometric and Traffic Operational Situations. (FCP No. 31A1802)
Objective: Develop warrants, criteria, markings, and signs. Field test plan for designating no passing zones for special situations.
Performing Organization: Texas Transportation Institute, College Station, Tex. 77843
Expected Completion Date: June 1980
Estimated Cost: $150,000 (FHWA Administrative Contract)

Title: Determination of Traffic Signal Intensity Level. (FCP No. 31A1832)
Objective: Determine if a change in traffic signal intensity is needed.
Performing Organization: KLD Associates, Inc., Huntington Station, N.Y. 11746
Expected Completion Date: February 1980
Estimated Cost: $290,000 (FHWA Administrative Contract)

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

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

FCP Project 1V: Roadside Safety Hardware for Nonfreeway Facilities

Title: Bridge Rail Retrofit for Curved Structures. (FCP No. 31V3042)
Objective: Conduct a minimum of five full-scale tests of two bridge rail retrofit designs (tubular thrie beam with collapsing tubes and shaped concrete barriers) using compact sedan and school bus.
Performing Organization: Southwest Research Institute, San Antonio, Tex. 78284
Expected Completion Date: December 1979
Estimated Cost: $200,000 (FHWA Administrative Contract)

FCP Project 1W: Skid Resistance Inventory Procedures and Equipment

Title: Predictor Model for Seasonal Variations in Skid Resistance. (FCP No. 31W1022)
Objective: Develop and validate a generalized model for predicting minimum pavement skid resistance from measurements taken during the testing season. Provide predictor equations for specific U.S. geographic areas. Identify and measure changes contributing to variations in seasonal skid resistance. Develop mechanistic model to predict changes due to traffic and environmental conditions.
Performing Organization: Pennsylvania State University, University Park, Pa. 16802
Expected Completion Date: February 1981
Estimated Cost: $207,000 (FHWA Administrative Contract)

Title: Skid Resistance of Nontangent Sections of Roadways. (FCP No. 31W2014)
Objective: Investigate equipment and procedures for wet pavement, skid resistance measurements on short and transitory road sections such as curves, acceleration and deceleration lanes, intersections, egress and ingress lanes, ramps, grades, superelevations, cloverleafs, bridge decks, and other geometric irregularities. Evaluate applicability of standard test method ASTM E274 and alternates.
Performing Organization: Texas A&M Research Foundation, College Station, Tex. 77843
Expected Completion Date: September 1981
Estimated Cost: $260,000 (FHWA Administrative Contract)

FCP Project 1Y: Traffic Management in Construction and Maintenance Zones

Title: Traffic Management During Freeway Reconstruction and In Rural Zones. (FCP No. 41Y1702)
Objective: Develop and document practical guidelines for traffic management during freeway reconstruction. Develop guidelines for traffic management in rural work zones. Develop practical guidelines for the highway advisory radio in work zones.
Performing Organization: Texas Transportation Institute, College Station, Tex. 77843
Funding Agency: Texas State Department of Highways and Public Transportation
Expected Completion Date: March 1980
Estimated Cost: $100,000 (HP&R)

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

FCP Project 3F: Pollution Reduction and Environmental Enhancement

Title: Ecology and Biological Control of Ice Plant Scale Pulvinaria in California. (FCP No. 43F1072)
Objective: Investigate the extent and rate of spread of pulvinaria scales, their host plant range, effect of climate, and the use of pesticides for control. Investigate the possible importation and release of insects to biologically control the scale.
Performing Organization: University of California, Berkeley, Calif. 94720
Funding Agency: California Department of Transportation
Expected Completion Date: June 1981
Estimated Cost: $148,000 (HP&R)

FCP Category 4—Improved Materials Utilization and Durability

FCP Project 4B: Eliminate Premature Deterioration of Portland Cement Concrete

Title: Polymer Concrete for Concrete Pavement Rehabilitation. (FCP No. 44B2362)
Objective: Develop procedures to repair transverse cracks, longitudinal cracks, punchouts, and spalls in concrete pavements using polymers. Prepare an implementation manual describing practical repair procedures.
Performing Organization: University of Texas, Austin, Tex. 78712
Funding Agency: Texas State Department of Highways and Public Transportation
Expected Completion Date: August 1981
Estimated Cost: $173,000 (HP&R)

FCP Project 4J: Corrosion Protective Coatings for Steel Structures

Title: Evaluation of Low-Solvent Maintenance Coatings for Highway Structural Steel. (FCP No. 34J1014)
Objective: Evaluate performance and general serviceability of the best available low solvents and determine how much improvement in these coatings may be expected in the near future. Determine the potential impact of environmental, safety, health, and other regulations upon bridge painting practices.
Performing Organization: Carnegie-Mellon Research Institute, Pittsburgh, Pa. 15213
Expected Completion Date: January 1980
Estimated Cost: $148,000 (FHWA Administrative Contract)

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

FCP Project 5D: Structural Rehabilitation of Pavement Systems

Title: Implementation of Rigid Pavement Overlay Design System. (FCP No. 45D2553)
Objective: Implement improved methods of pavement rehabilitation and design for the Texas State Department of Highways and Public Transportation.
Performing Organization: Center for Highway Research, Austin, Tex. 78701
Funding Agency: Texas State Department of Highways and Public Transportation.
Expected Completion Date: August 1981
Estimated Cost: $472,000 (HP&R)

Title: Modulus Characterization of MSHA Base/Subbase Materials for Use in Pavement Design and Rehabilitation. (FCP No. 45D3322)
Objective: Characterize various base/subbase materials currently being used by Maryland State Highway Administration for input into elastic layer programs. Include repeated load testing and a comparison with diametric testing for the stabilized materials.
Performing Organization: University of Maryland, College Park, Md. 20742
Funding Agency: Maryland State Highway Administration
Expected Completion Date: December 1980
Estimated Cost: $116,000 (HP&R)

FCP Project 5K: New Bridge Design Concepts

Title: Design of Slender Nonprismatic or Hollow Bridge Piers or Columns. (FCP No. 45K1052)
Objective: Develop design-oriented procedures for slender concrete bridge piers.
Performing Organization: University of Texas, Austin, Tex. 78712
Funding Agency: Texas State Department of Highways and Public Transportation
Expected Completion Date: August 1980
Estimated Cost: $120,000 (HP&R)

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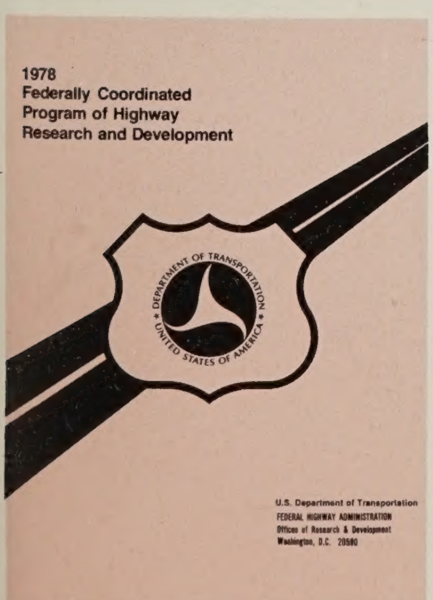
New Publications

The Offices of Research and Development (R&D), Federal Highway Administration (FHWA), have announced release of their fiscal year 1978 Annual Report on the Federally Coordinated Program (FCP) of Highway Research and Development.

This is the fifth year for the issue of this account of FHWA's progress in highway R&D. This year's 25-page document consists of two major parts. The first, a discussion of R&D activities and achievements, is organized into five major topics—safety research, traffic operations research, environmental research, structural research, and highway maintenance. This part is preceded by a brief statement on the background of the FCP. The second major part deals with the organization, funding, and services of the Offices of R&D. The report is prefaced by a message from Federal Highway Administrator Karl S. Bowers.

Copies of the report are on sale for $1.40 by the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Stock No. 050-001-00140-3).



The Offices of Research and Development (R&D), Federal Highway Administration (FHWA), have announced release of the 1979 edition of the Index of Research and Development Reports.

The indexed reports, identified by FHWA-RD numbers, originate from administratively funded contracts and FHWA R&D staff studies. Highway Planning and Research (HP&R) program reports are not listed. This edition cites those R&D reports printed from late 1971, when the numbering system began, to June 30, 1978, when the final data were submitted to the Highway Research Information System (HRIS). This index updates the December 1976 preliminary edition.

The index lists R&D reports alphabetically by title and includes all available bibliographic information for each report. A cross index located at the back of this document lists reports in the sequence of their FHWA-RD report numbers; a number follows which cites the page in the main section where complete bibliographic information is listed.

While supplies last, individual copies are available without charge to highway-related agencies and universities. Requests should be sent to the Federal Highway Administration, Engineering Services Division, HDV-14, Washington, D.C. 20590.

Copies of the index are on sale to the public by the National Technical Information Service (NTIS), 5285 Port Royal Road, Springfield, Va. 22161.

New R&D Report Covers

A new cover design will be used on standard, technical research and development (R&D) reports that are assigned FHWA-RD numbers after January 1, 1979. The new design portrays a colored diagonal stripe intersecting a shield. A logo within the shield indicates the subject matter of the report. The stripe, representing a highway, is color-coded according to the category of the Federally Coordinated Program (FCP) of Research and of highway Development under which it falls: Category 1, Improved Highway Design and Operation for Safety—red stripe; Category 2, Reduction of Traffic Congestion and Improved Operational Efficiency—dark blue stripe; Category 3, Environmental Considerations on Highway Design, Location, Construction, and Operations—light blue stripe; Category 4, Improved Materials Utilization and Durability—brown stripe; Category 5, Improved Design to Reduce Costs, Extend Life Expectancy, and Insure Structural Safety—gray stripe; Category 6, Development and Implementation for Research Introduction—green stripe; and Category O, Other New Studies—orange stripe. This new cover design will enable the reader to identify, at a glance, the FCP category.

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