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40

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BACK COVER:
The Wye on U.S. 16–385, 5 miles from Mt. Rushmore, Keystone, S. Dak. (Photo courtesy of South Dakota Department of Tourism, Pierre, S. Dak.)

IN THIS ISSUE

Articles

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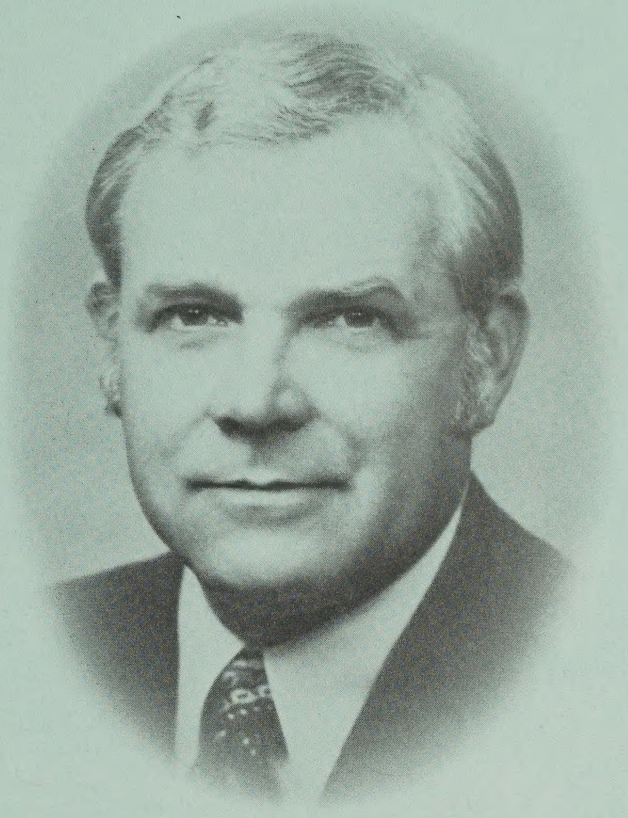

In exercising responsibility for the administration of the Federal Highway Program, the Federal Highway Administration with the full cooperation and support of the Department of Transportation has vigorously stressed the importance of a broadly based research and development program designed to provide new and improved methods and procedures for constructing and maintaining the Nation's highway transportation system.

The key element in this effort is the Federally Coordinated Program of Research and Development in Highway Transportation (FCP), which focuses all available resources on problem areas identified by the entire highway community. By coordinating the efforts of State transportation agencies, private industry and research organizations, universities, and Federal agencies through the FCP, the highest level of expertise is brought to bear on our most critical highway transportation problems.

The cooperative Federal-State relationship, which has long been recognized as the key element in the success of the Federal Highway Program, is certainly evident in the implementation of highway research. By means of this partnership venture, research and development outputs are applied to highway planning, construction, operations, and maintenance activities on a nationwide scale.

Norbert T. Tiemann
Federal Highway Administrator

Tignor, Samuel C., Hess, Joseph W.
Traffic control Interchanges, Diamond
Title Traffic signals - Lights

Signalization of Diamond Interchanges

by Samuel C. Tignor and Joseph W. Hess

In this article the authors describe a research project in the Federally Coordinated Program of Research and Development in Highway Transportation (FCP) on the signalization of diamond interchanges. This project was performed by the Federal Highway Administration in cooperation with the California Department of Transportation and the city of Los Angeles. Initial research provided simulation and analytic models for use in developing the signalization of diamond interchanges. Field implementation of new methods for fixed-time control showed a reduction in travel time, stopped time, and number of stops. Furthermore, the total delay was reduced by 25 percent when a real-time system was used to control the traffic signals.

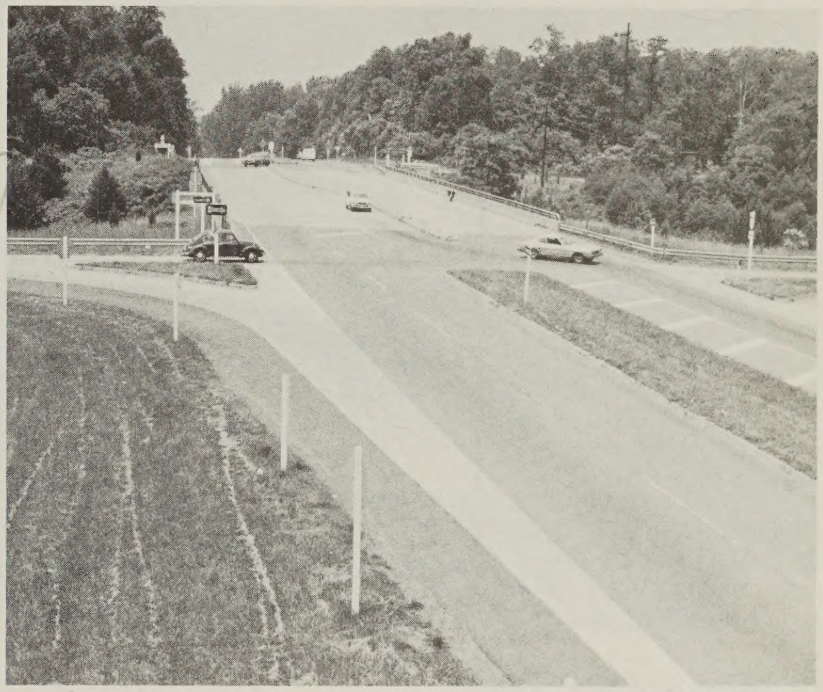

Figure 2.—An example of specialized channelizations, turning pockets, and lane markings.

Introduction

Diamond interchanges are commonly used because of their simplicity of design and because they can be constructed economically within a minimum right-of-way. (See fig. 1.) In rural areas where demands are light, little operational difficulty is encountered with the use of diamond interchanges. However, in urban and some suburban areas where land use characteristics are considerably different, traffic demand at diamond interchanges is frequently heavy, particularly during peak commuter periods and noon hours. Layout or upgrading of diamond interchange complexes should employ good design criteria. There is no substitute for good basic geometric design and layout of an interchange; design techniques incorporating good delineation, provision for adequate turning pockets, channelizations, and multi-lane widening of the ramp exit terminal at the cross street should be used whenever possible (fig. 2). Good design not only improves interchange capacity and safety, it also enhances traffic control. For interchanges in or near major urban areas, traffic operational plans should recognize the possibility of substantial future changes in land use and traffic demand. For example, as traffic demands increase it is often necessary to replace stop sign controls with traffic signals (fig. 3). This article describes a program of research and development on traffic control of diamond interchanges.

Figure 3.—At unsignalized diamonds in surburban areas with large demands, required driver maneuvers generate numerous conflicts.

← *Figure 1.—Diamond interchange within a minimum right-of-way.*

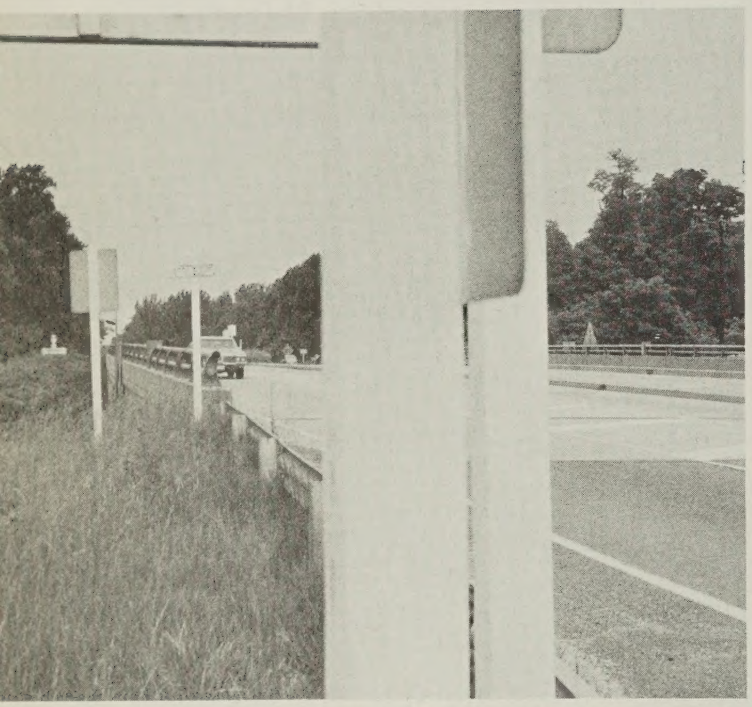

Figure 4.—Off-ramp driver visual obstructions caused by roadside hardware.

Figure 5.—A separate left-turn lane.

Principles of Good Design

Traffic signal control is normally not selected or determined independently from site geometrics. Three major factors—sight distances, roadway width, and the effectiveness of signalization plans—influence good diamond interchange operations. By providing good crossroad sight distance at exit ramp terminals, operation of diamond interchange off-ramps can often be maintained at a capacity and safety level which can delay the need for signalization. A special American Association of State Highway and Transportation Officials (AASHTO) Traffic Safety Committee found that "exit ramps built immediately adjacent to the overpasses had sharply restricted sight distances" because of the visual obstruction caused by the bridge parapet wall and railing *(1)*.[1] In addition to improving safety and arterial street operations, exit ramp terminals moved away from the ends of the overpass can eliminate or postpone the need for traffic signals at exit terminals. On the other hand, drivers must have sufficient sight distance to view traffic approaching the interchange. The road profile can be a critical factor, especially on older roads which were lightly traveled farm roads before the highway was constructed. Off-ramp sight distance can also be restricted by buildings, shrubbery, roadside hardware, or vehicles parked on the arterial street, as shown in figure 4.

[1] Italic numbers in parentheses identify the references on page 47.

Roadway width is one of the most crucial variables affecting diamond interchange operation. The Highway Capacity Manual states that "both intersection approach capacity and service volumes vary directly with the width of approach" *(2)*. Approach width and traffic demand are primary considerations used to determine the number of lanes. For many diamond interchange applications it is desirable to use special or exclusive lanes for left turns, right turns, and through movements.

The AASHTO red book gives the following recommendation:

> It frequently will be found necessary to widen a one-lane exit ramp to two or even three lanes at its junction with a cross street in order to achieve a balance between the capacity of the intersection and that of the ramp roadway, thereby avoiding a backup onto the freeway. Unless design peak hour volumes equal or exceed 1,500 vph, there is seldom need to provide a full-length, two-lane ramp *(3)*.

Free right-turn lanes are effective for accommodating large off-ramp volumes from freeways.

Separate turning lanes are sometimes needed for the arterial street in order to assure a good level of service through the diamond interchange. Typically, separate turning lanes are used in the median or at the curb for on-ramp movements. These separate turning lanes permit through traffic to progress along the arterial street without being delayed by traffic waiting to enter the freeway on-ramps. (See fig. 5.) Depending upon the on-ramp demand, sometimes more than one separate left-turn lane is needed to prevent long queues from congesting the storage area between the intersections.

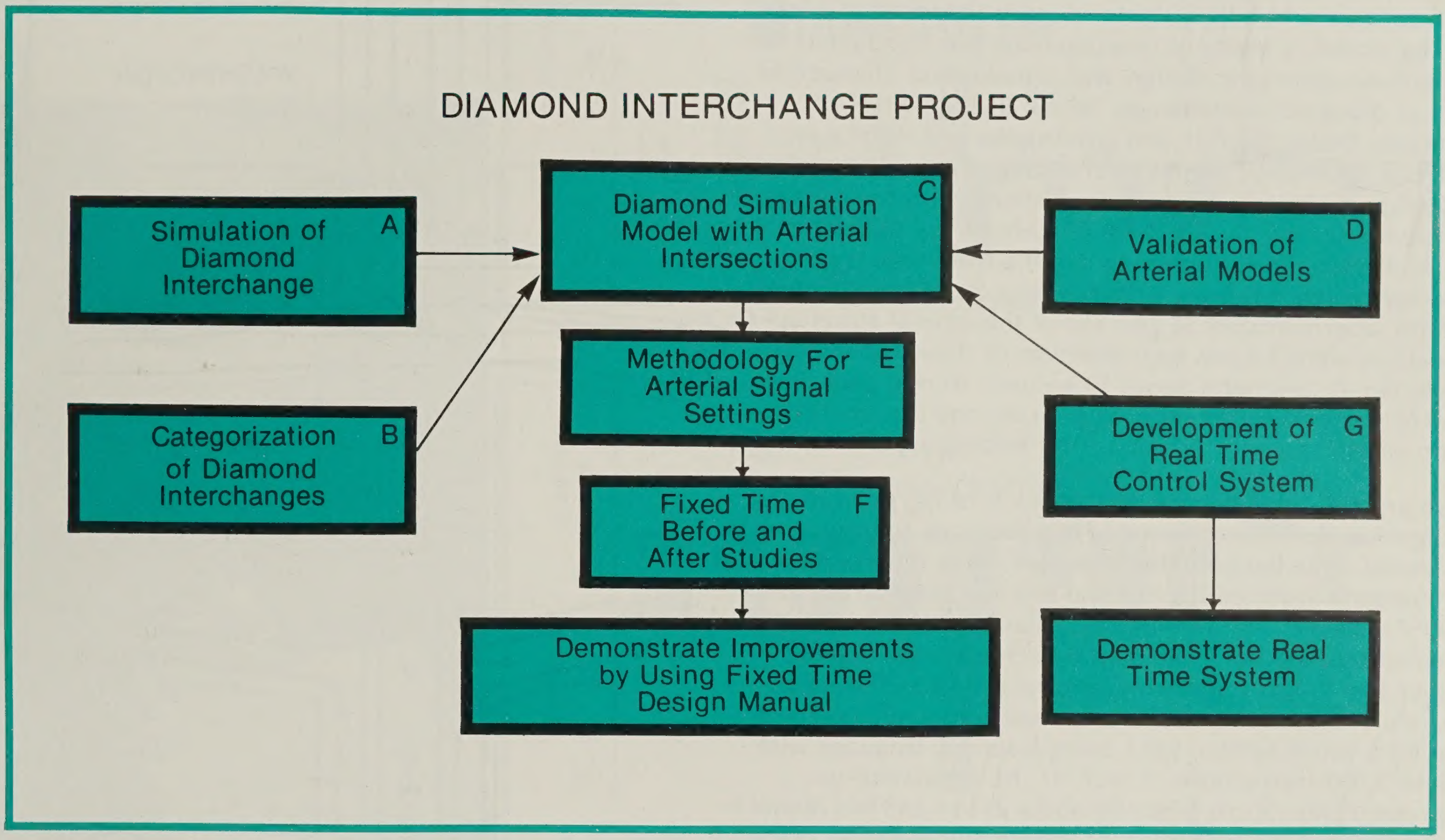

Figure 6.—Diamond interchange project.

Signalization of Diamond Interchanges

Effective diamond interchange signalization depends on a number of considerations. Moskowitz *(4)* makes the following suggestions for diamond signalization:

◆ Keep as many movements going simultaneously as possible.

◆ No single movement should be stopped more than once for any given cycle.

◆ Progressions should be provided through both intersections.

◆ Accordion type action should be avoided.

◆ With the completion of each movement, no vehicles should remain within the cross street between the ramps.

Factors such as these should be remembered when developing diamond interchange signal timing plans. Historically, the manner in which these plans are developed has varied widely. Researchers such as Pinnell and Capelle *(5)* and Woods *(6)* have developed signal timing methods which are sometimes used for diamond interchange applications. In general, timing methods based on the assumption of random vehicle arrivals are preferred over other methods. For maximum use of green time, phasing schemes should be used which permit phase overlap. The design of traffic signal control systems normally incorporates these features. For many applications, fixed-time traffic control systems are most appropriate. Future widespread utilization of minicomputers or microprocessors should provide needed flexibility for control where traffic demands are unpredictable, heavy, or excessively variable among the approaches and nearby intersections.

Research Program on Diamond Interchanges

The Federal Highway Administration (FHWA) research program on diamond interchanges began in 1966. Initial research developed a validated simulation model for studying geometric and fixed-time control aspects of diamond interchanges and their adjacent freeway sections. At the completion of this initial research, decisions were made to enlarge the simulation model to include the nearby street intersections and to extend control research to computerized real-time control. These decisions were made in cooperation with the California Department of Transportation and the city of Los Angeles, both of which contributed significantly to the success of the project. Figure 6 shows the organization of the diamond interchange project, with the previously described freeway simulation development in block A.

Before the nearby adjacent arterial intersections were added to the model, a series of investigations was conducted to determine geometric design and signalization characteristics of diamond interchanges (block B). The cities of Chicago, Dallas, Detroit, and Los Angeles and the States of Arizona, California, Illinois, and Michigan contributed data from approximately 160 signalized diamond interchanges, reference *(7, vol. 1)*. About 80 percent of the traffic signal controllers in these studies were of the fixed-time type, the average distance between ramp intersections was 255 feet (78 m), approximately 55 percent of the arterial street approaches were 3 lanes per direction of flow, the average cycle length used was about 70 seconds during peak flow and 60 seconds during off-peak periods, and the most common phase sequencing was of the leading type.

Two arterial simulation models (block C of fig. 6) were developed to determine improved methodology for operating diamond interchange traffic signals. One is a microscopic simulation model, coded in Jovial and executed on the IBM 360/67 system *(7, vol. 3)*. This model has approximately 9,500 instructions and has a simulation-to-computer time ratio of about 3 to 1. This microscopic model is best suited for research. The second model is a macroscopic simulation written for a Varian 620 I or 620 L using assembly language with about 1,900 instructions *(7, vol. 4)*. Its simulation-to-computer time ratio is generally about 25 to 1 and this model is best suited for pretesting proposed signal-timing plans prior to field implementation. A traffic engineer would find this model particularly useful for his usual traffic signal-timing problems.

Before either of the two simulation models was used for algorithm development each was first validated for realism (block D). Simulation results were compared to two operating diamond interchanges, one in which the freeway overcrossed the arterial street (Ventura Freeway–Coldwater Canyon Road) and the other in which the arterial street overcrossed the freeway (Santa Monica Freeway–Western Avenue). The validation criterion was average travel time and statistical tests showed the models to be acceptable *(7, vol. 5)*.

As shown in block E of figure 6, new methodology for setting arterial traffic signals was developed. The development of the methodology considered the works of Webster *(8)* on optimum cycle and splits, Little *(9)* and Yardeni *(10)* on offsets, and Inose *(11)* on optimal cycle, splits, and offsets.

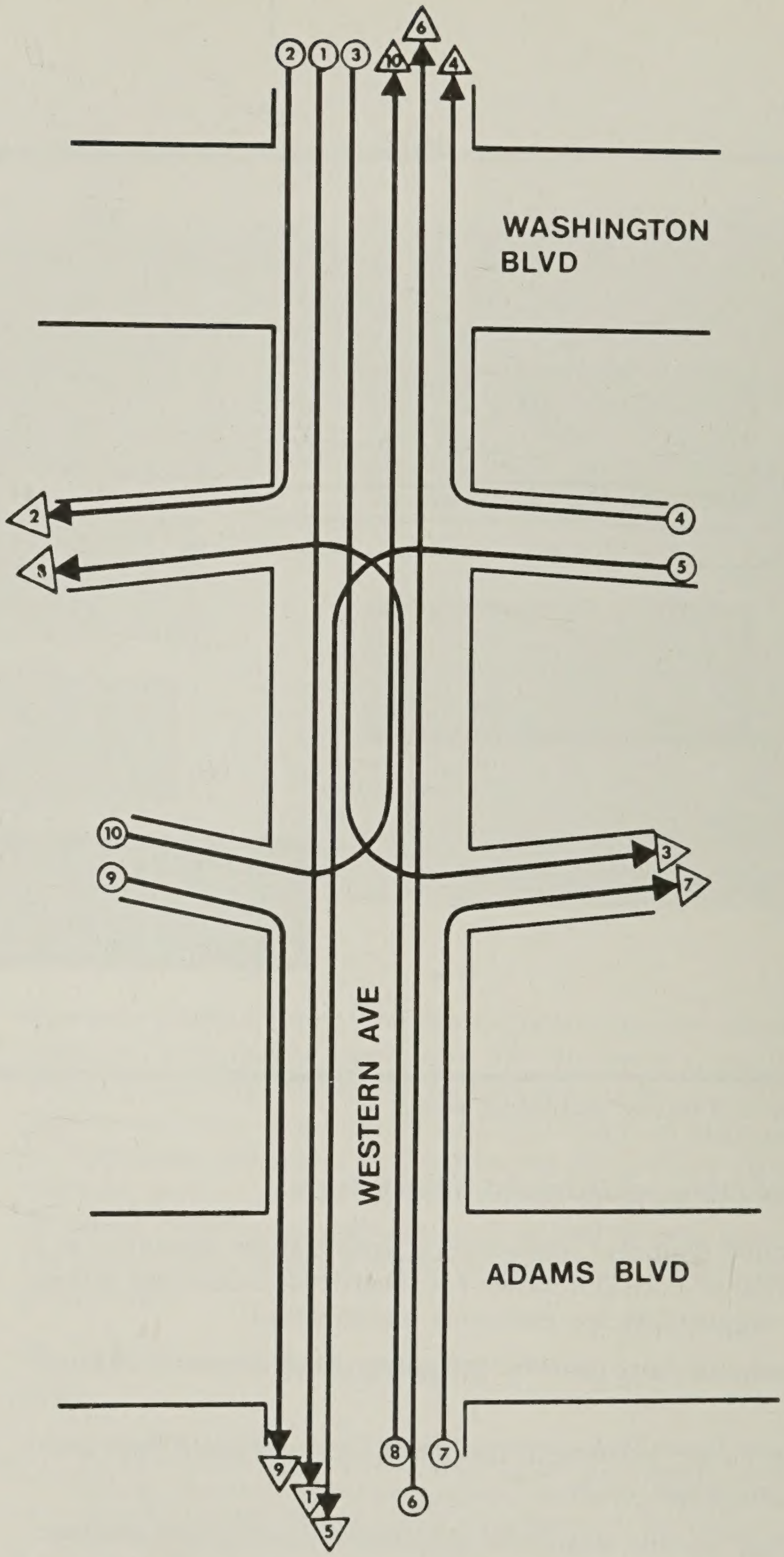

Figure 7.—Selected origin-destination test-car paths.

The new approach is based upon a combination of Webster's and Inose's methods *(7, vols. 12, 14)*. This combination was found to be the best of 12 proposed methods in minimizing the average vehicle delay as tested by simulation runs using the macroscopic model. The new method provides for up to four phases for optional turning lanes and left-turn pockets, for pedestrian crossings, and for right-turning vehicles. It also provides traffic signal settings for two adjacent intersections in proximity to the diamond intersections.

Figure 8.—TRADAC equipment.

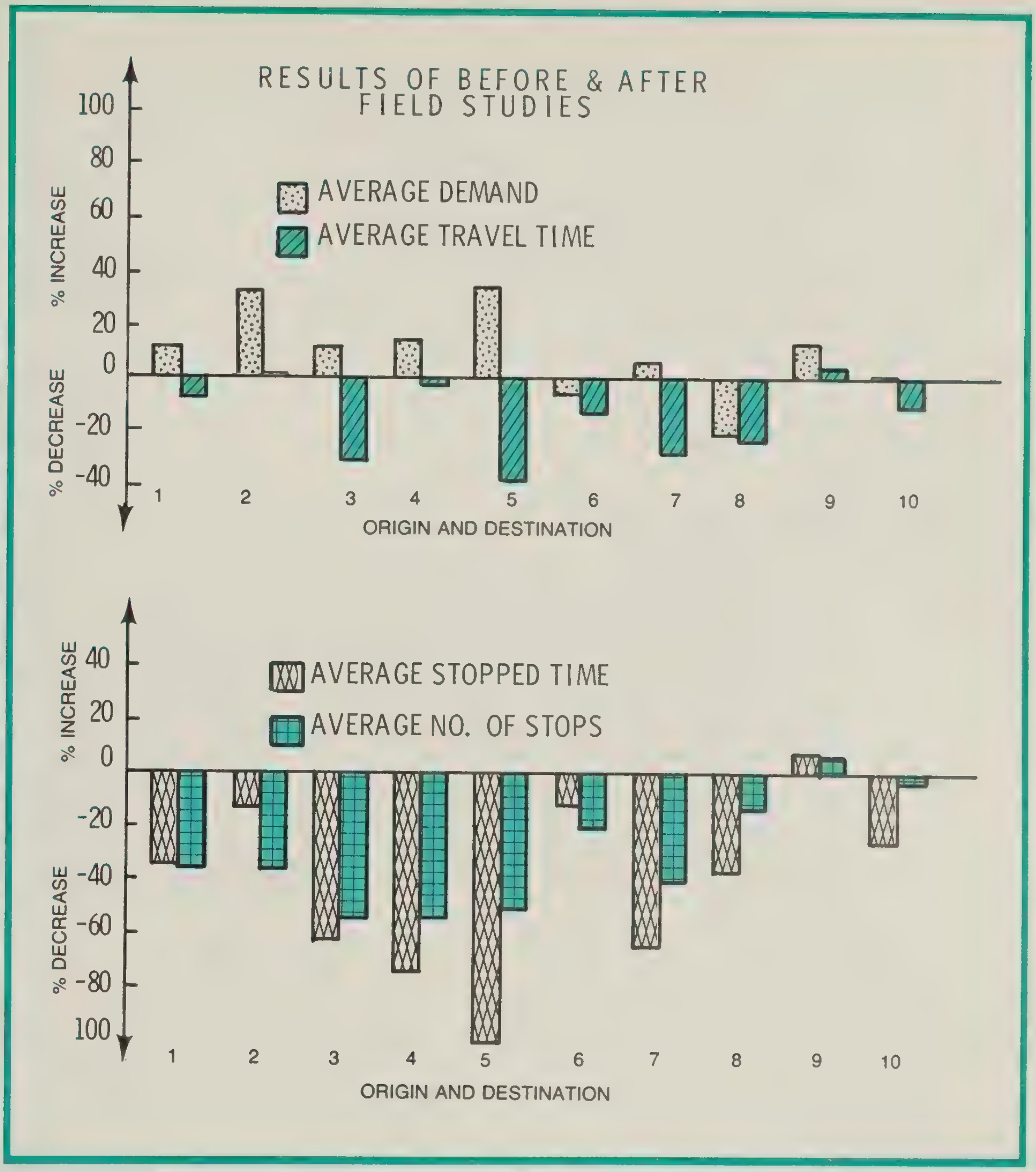

Figure 9.—Improvements found with improved fixed time methodology.

To test the effectiveness of the improved methodology for setting arterial traffic signals at diamond interchanges, before and after field tests were conducted (block F of fig. 6). The field studies were performed at the Western Avenue and Santa Monica Freeway interchange in Los Angeles *(7, vol. 5)*. Western Avenue is a major arterial street with an average daily traffic of about 39,000. During peak periods Western Avenue carries three lanes of traffic in each direction. Table 1 presents the before and after signal-timing plans which were tested.

Table 1.—Before and after signal-timing plans, in seconds[1]

Intersection	Phase A Before	Phase A After	Phase B Before	Phase B After	Phase C Before	Phase C After	Offset Before	Offset After
Washington Blvd.	46.0	31.2	34.0	28.8	—	—	77.6	7.5
North ramps	43.2	24.0	19.2	22.0	17.6	14.0	40.8	24.0
South ramps	36.8	22.0	19.2	14.0	24.0	24.0	0.0	0.0
Adams Blvd.	46.4	34.0	33.6	26.0	—	—	47.2	11.0

[1] *Before* cycle=80, *after* cycle=60.

Figure 7 illustrates the 10 paths used by test cars in the before and after studies. These paths represent the heaviest traffic movements through the intersection. Field data were collected through moving vehicle studies using an instrumentation system called TRADAC. The TRADAC equipment, shown in figure 8, monitors four trip parameters; elapsed travel time, total stop time, number of stops, and number of brake applications. The results of the before and after studies are shown in figure 9. In reviewing the results it should be noted that although demand increased for nearly all the paths, the average travel time, average stopped time, and average number of stops decreased. As a result of these findings it was concluded that the new methodology for setting diamond interchange traffic signals provided substantial improvements.

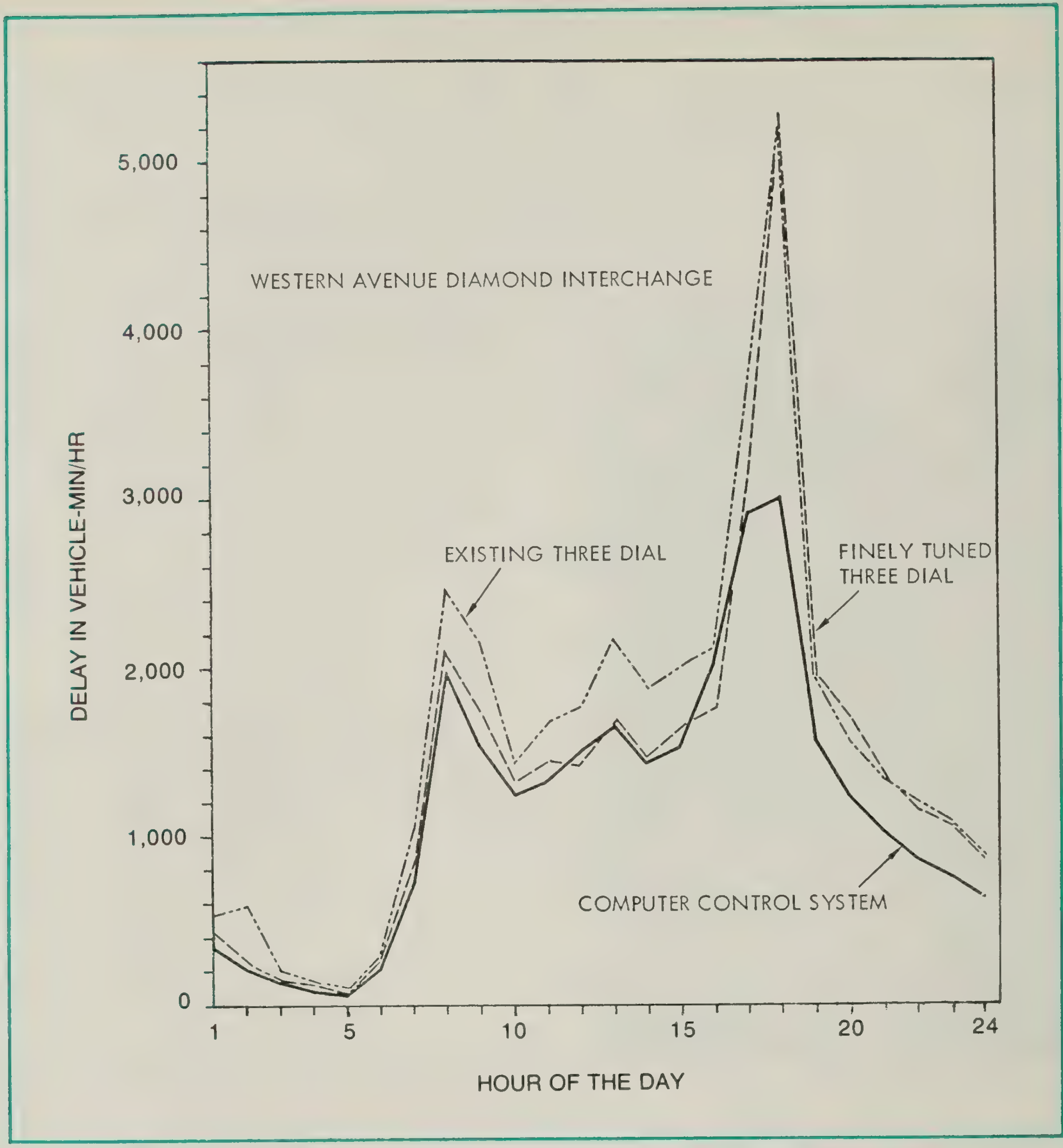

Figure 10.—Improvements expected with computer-controlled diamond interchange traffic signals.

Another phase of the research (block G of fig. 1) called for determining the feasibility of developing a second-generation, real-time method for controlling the traffic signals of a diamond interchange complex *(7, vol. 5)*. The simulation models were used to determine the relative improvement that might be expected with real-time control. The results of these studies are shown in figure 10. This illustration represents an 11- and 25-percent reduction in total delay over the existing system by using the new fixed-time and real-time control methods, respectively. Each graph shown was developed from a simulated data base of approximately 79,500 vehicles.

The improvement expected by the computer-controlled system was large enough to justify development and testing of a prototype real-time control system *(7, vols. 9, 10)*. The research installation utilized 49 inductive loop detectors, a Varian 620 I minicomputer with 16K core storage (8K was needed for data evaluation purposes), hard-wire communication, and off-the-shelf, pretimed, electromechanical intersection controllers modified for real-time phase length adjustment and skipping. Cycle lengths, splits, and offsets were computed at prescribed time intervals utilizing the volume inputs detected by the loop detectors. In this way the control system adapted to continuously changing demands and was not restricted to three signal-timing plans as it would have been with three-dial equipment. The real-time software program was coded in Varian 620 I or L assembly language with about 3,500 instructions.

Figure 11 illustrates the typical daily performance of the computer-controlled system compared to the fixed-time control method. Further evidence of the real-time system's

Figure 11.—Distribution of total delay.

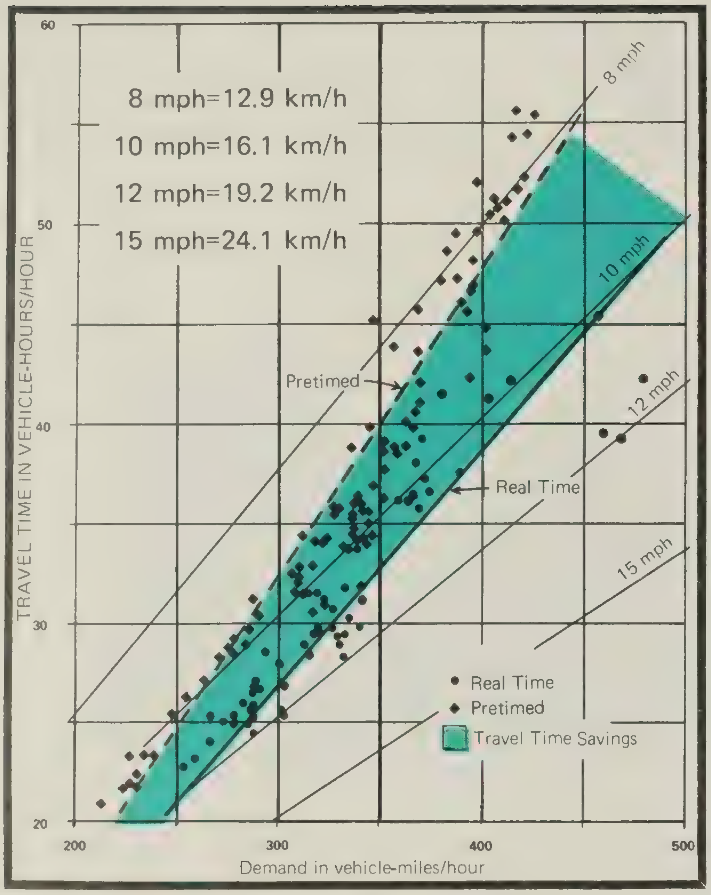

Figure 12.—Improvements with real-time system.

benefits is shown in figure 12, which illustrates the relationship of total vehicle-hours and demand. For example, at a demand of 350 vehicle-miles per hour the travel time for the pretimed method of control is 40 vehicle-hours per hour compared to 32.5 for the real-time control method. This difference represents an approximate 20 percent reduction in travel time. The overall time savings is represented by the shaded area.

Part of the research project was an evaluation of the performance of the real-time system when fewer detectors were used *(7, vol. 9)*. A minimum detector configuration (fig. 13) was evaluated as a possible alternative to the full 49-detector configuration. The minimum 18-detector configuration provided the same level of system improvements; for most applications about 20 detectors should be sufficient for real-time control of a diamond interchange complex.

The cost of installing a real-time traffic control system at a diamond interchange, similar to the one installed in Los Angeles, is estimated to be between $66,000 and $91,000 *(12)*. Currently the California Department of Transportation under contract to FHWA is developing a diamond interchange microprocessor system. It is expected that installation costs can be further reduced with this new development.

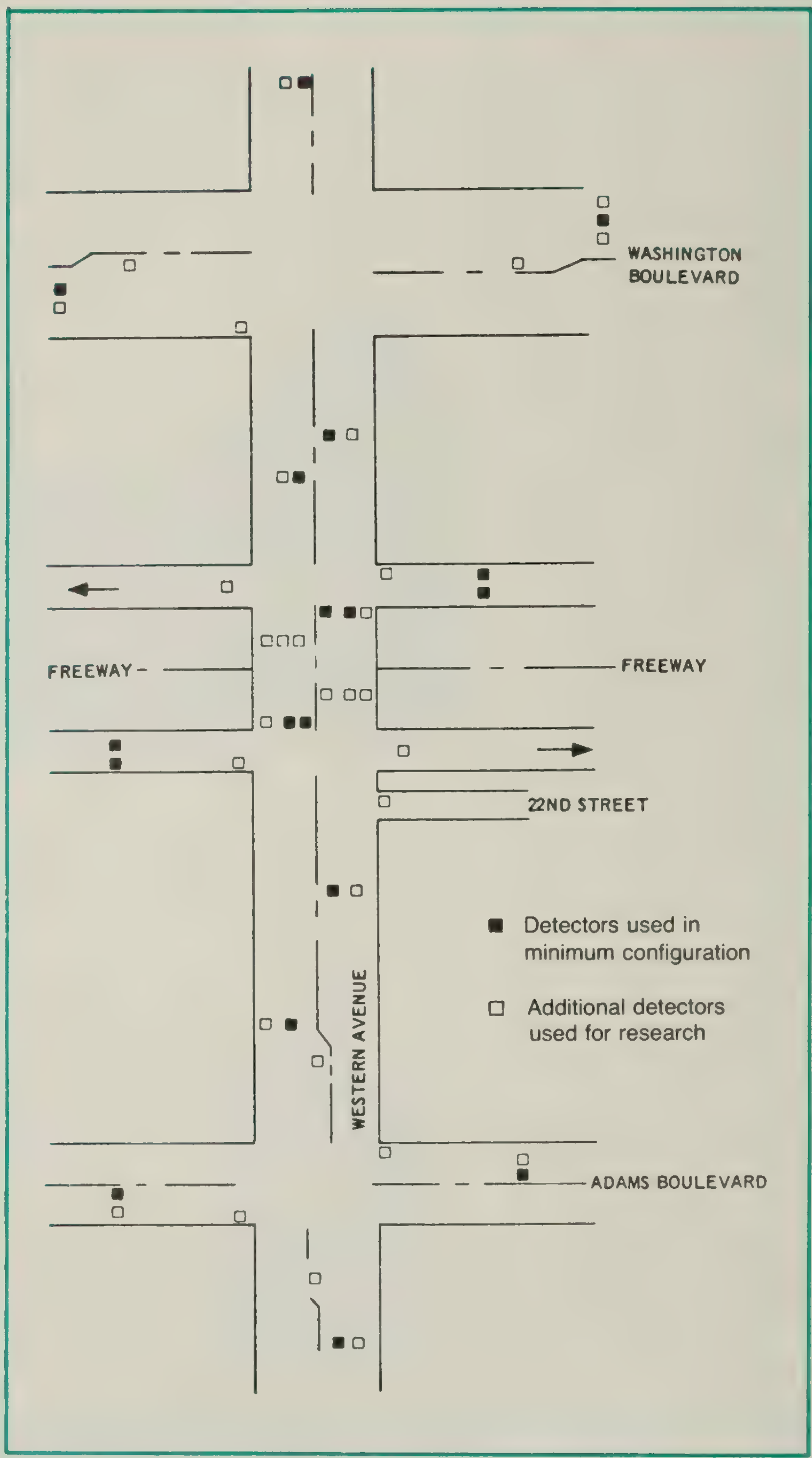

Figure 13.—Minimum detector configuration.

Project's Next Phase

The Implementation Division of FHWA's Office of Development is the coordinator of the next phase of the diamond interchange project. This phase will emphasize development and distribution of special user documents to assist in implementation of many of the previously described results. (See fig. 14.) Currently the Implementation Division also has underway some field tests and evaluation activities relating to diamond interchanges.

Figure 14.—An application of a heavy traffic corridor featuring a series of signalized diamond interchanges with mass transit in the median, all within a narrow right-of-way.

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[2] Reports with PB numbers are available in paper copies and microfiche from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161.

Rapid Measurement of Concrete Cover on Bridge Decks

by Kenneth R. Moore

Premature deterioration of concrete bridge decks due to corrosion of the reinforcing steel is a serious problem. Corrosive agents, such as water and deicing salts, are principal contributing factors to this problem. Evaluation tools for testing concrete bridge decks and for determining the effectiveness of bridge deck protective systems are currently being studied by the Federal Highway Administration. One of these studies involves the development of a Rolling Pachometer that can be used for nondestructive determination of the cover over the reinforcing steel on concrete bridge decks. This unit is an effective tool for both inspection and quality control. The Rolling Pachometer has proved to be accurate, reliable, and capable of gathering data at a rate 20 times that of conventional, hand-held methods.

Introduction

Many concrete bridge decks are plagued with spalling and delamination at the level of the top mat of reinforcing steel. Research has shown that this distress is caused by the intrusion of water-born chlorides from deicing operations or marine environments. The chlorides induce corrosion of the reinforcing steel which in turn causes concrete distress. The closer reinforcing steel is to the surface, the more likely it is to be attacked by corrosive agents. The problem of premature bridge deck deterioration is a serious one, and the Federal Highway Administration (FHWA) is currently sponsoring a nationwide program to evaluate several new bridge deck protective systems.

While a future solution to spalling problems may lie in waterproof membranes, epoxy coated reinforcing steel, polymer concrete composites, or internally sealed concrete, bridges currently planned or under construction that do not have special protection against corrosive agents can be provided with additional service life by insuring the attainment of greater concrete cover over the reinforcing steel.

Chloride analyses of bridge deck concretes have confirmed that the chloride content decreases rapidly with depth within the concrete.[1] Thus, reinforcing steel with only a small amount of concrete cover will be much more susceptible to corrosion-caused distress than steel placed deeper within the concrete.

Crucial to the final depth of cover are the variability of steel positioning prior to concrete placement and movement of the steel during placement of the deck concrete. Recently, there have been instances where reinforcing steel has been visible on the concrete surface of several newly constructed Interstate highway bridges. If a bridge inspection reveals the existence of an insufficient concrete cover, several corrective steps should be considered for future projects:

- Increase the design depth of the reinforcing steel.
- Increase the quantity and quality of steel supports. This will reduce the variability in placement and movement during construction.
- Intensify bridge deck construction inspection procedures.
- Use a less permeable concrete mix (lower water-cement ratio) that will be more resistant to penetration by water and deicing salts.

For decks already constructed but not yet chloride contaminated, a waterproof membrane or other protective overlay could be applied.

Past Inspection Methods

Since 1972, State highway departments and FHWA Region 15's R&D Demonstration Project No. 15 have conducted measurements of concrete cover on bridges in many States using a James Pachometer. This is an electronic device for nondestructively measuring the distance from the bridge surface to the top edge of the reinforcing steel. The apparatus employs the principle of magnetic flux.

The Region 15 crew consisted of three men using two hand-held pachometers. In evaluating the coverage on bridge decks, use of the hand-held pachometer proved to be a tedious and time consuming process. Two men on their hands and knees recorded values from the meter scale as the probe was positioned and the meter peaked over each rebar. Measurements were taken on a 5-foot (1.5 m) grid with the curb as a reference. Longitudinal and transverse reinforcing steel measurements were recorded and an average depth of cover or standard deviation for depth of cover was computed from the data.

The Rolling Pachometer

To expedite the process of bridge deck evaluation, the Oklahoma State Department of Highways envisioned a device that could be rolled across a bridge deck at a fixed rate of speed and would record the depth of steel on a strip chart recorder.

The FHWA's Engineering Services Division, Office of Development, at the Fairbank Highway Research Station designed and fabricated a prototype system. This consisted of a hand-held pachometer, an inverter, a 12-volt car battery, a hot stylus strip chart recorder, a speedometer, and electronic equipment comprised primarily of a filter and amplifier. The detector probe was suspended on the cart one-fourth inch (.635 cm) from the bridge deck surface. This distance was determined after considering bridge deck roughness and instrument response.

To establish its feasibility, the Rolling Pachometer prototype was tested by the Engineering Services Division and Region 15 for several weeks on local bridge decks. Results of the evaluation were encouraging: The accuracy and repeatability of the system proved excellent.

With the advent of good battery-operated strip chart recorders, a second generation Rolling Pachometer was developed. This system contains a modified James Hand-held Pachometer; a battery-operated, two-channel, pressurized ink recorder; a speedometer; and associated electronics. The entire system operates on the recorder's rechargeable batteries.

Being battery operated, the recorder eliminates the need for the inverter and the car battery that were used in the prototype. Consequently, the weight and size of the Rolling Pachometer are reduced significantly. The prototype weighed 170 pounds (77.1 kg) compared to 88 pounds (39.9 kg) for the present version which also includes a lexan cover for protection. Its electronics consists of an amplifier, filters, voltage regulators, adjustable high and low reinforcing bar limit controls, a magnetic sensor, and counters for processing and displaying distance marks on the chart graph.

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Because of the dynamic nature of data collection and the magnetic flux principle used, a constant speed of 1 mph (1.609 km/h) is required. The speedometer works in conjunction with the magnetic sensor which is located on one of the wheels. Pulses from the sensor, which are processed and used to indicate speed, are also fed into counters which trigger a distance mark on the left side of the chart paper (fig. 1). These distance marks occur every 18 inches (457.2 mm) of travel; and, when used with the manual event marker switch located in the handle, allow the operator to correlate the data on the chart paper to a particular area of interest on the bridge. The manual event mark is displayed on the right side of the chart paper.

Data, representing variations of the pachometer meter as the probe passes over the reinforcing bars, is displayed on Channel 2 of the chart paper. As the bridge is traversed, the sinusoidal nature of this recorded data represents the peaks which originally were obtained more laboriously with the hand-held pachometer.

Each peak represents a reinforcing bar and, by measuring the distance from each peak to the edge of the chart paper graph, actual depth of cover can be determined from a calibration curve.

High limit and low limit controls allow the operator to preset conditions on the graph that will identify reinforcing bars which are either higher or lower than a specified depth. Rebars which are out of the designed depth specification are identified by a spike on Channel 1 of the chart paper. A spike to the right of center indicates a rebar that is higher than the preset limit. A spike to the left of center indicates a rebar that is lower than the preset limit. This feature enables the spikes to be counted, thus easily determining the percentage of rebars out of specification rather than having to read all peaks on the graph to determine the actual depth of each rebar. Figure 2 shows operator performing preliminary calibration checks and adjusting the high and low limit controls.

Calibration curves for the Rolling Pachometer are developed by constructing a test track which allows for simulation of actual bridge conditions. Various reinforcing bar sizes and grades can be inserted into the track. Calibration runs are made across the track with rebars placed at each of four depths and at several common spacings.

The peak readings on the recorder graph for each run are then averaged, and a calibration curve is developed for each size of rebar, grade of steel, and spacing required. While some of the peak variations between different grades and spacings are minute, greater accuracy can be obtained if all contributing factors are considered.

Rolling Pachometer Features

The Rolling Pachometer has many features that have been designed to ensure ease of operation. Calibration and operating procedures have been simplified as well as means for interpreting data. Some of these features are:

- Rechargeable batteries.

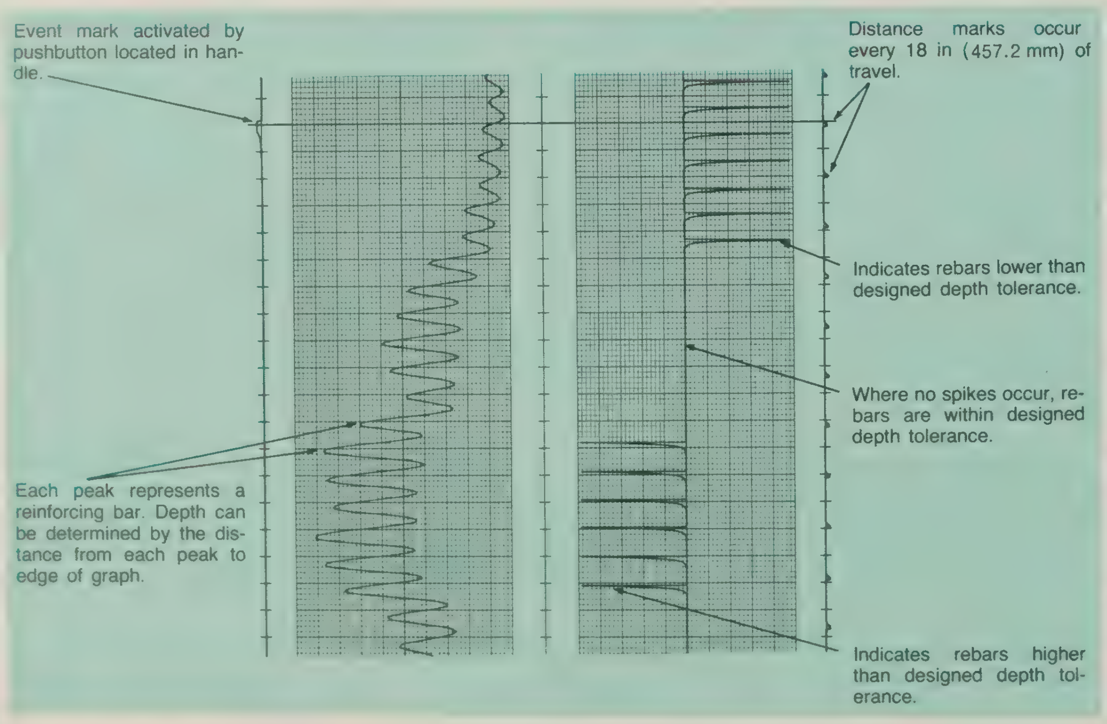


Figure 1.—Typical chart recorder presentation.

- Battery condition indicator on the recorder.
- No electrical shock hazard.
- Manual event mark switch in the handle.
- Instant record of acquired data.
- Automatic distance marks for the correlation of data.
- Go-no-go limit controls to simplify data reduction.
- Writing system (ink) stops when the battery or the chart paper is low.
- Easy calibration.
- One-person operation.
- Four to five hours of continuous operation.
- Size: 12 in by 40 in by 20 in (305 mm by 1,016 mm by 508 mm).
- Weight: 88 pounds (39.9 kg.).

A shipping case, which is provided, enables the unit to be transported easily from one bridge site to another or longer distances via commercial freight.

Evaluation

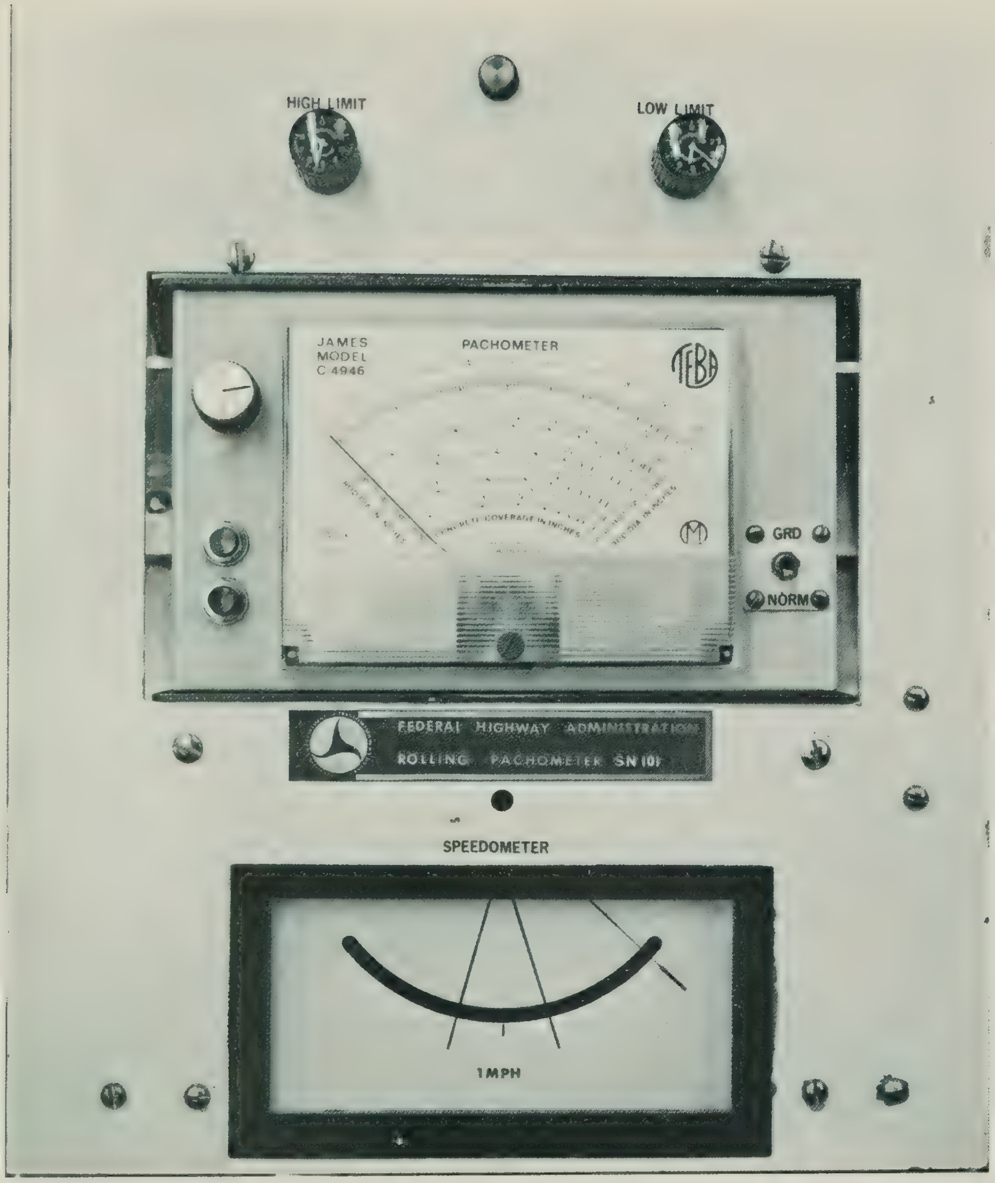
In conjunction with the Implementation and Engineering Services Divisions, FHWA, the Oklahoma, Pennsylvania, and West Virginia State highway departments participated in field test evaluations of the Rolling Pachometer. Results of the evaluations in these three States were encouraging.

Repeatability, stability, and operation under various temperature extremes from morning to afternoon proved excellent. In comparisons with other calibrated, hand-held pachometers, the results were also very good. However, a constant bias effect was noted in both the hand-held and Rolling Pachometers in each of the State evaluations. The majority of peaks on the chart graphs indicated that rebars were closer to the surface than they were found to be when measured after coring operations.

The error introduced in both units ranged from approximately one-eighth inch (3.2 mm) to one-quarter inch (6.4 mm) depending on the depth of the reinforcing steel. The bias effect was caused by magnetite—small iron-oxide particles. Varying quantities of magnetite are found in most sand used in concrete mixtures. Since the percentage of magnetite varies according to where the sand is obtained, it is impossible to predict an average bias and therefore to determine a calibration curve correction factor which would apply to all bridge decks.

A suitable method for taking the magnetite bias effect into account when testing a given bridge is to core over several reinforcing bars and then compare the actual depth measurement with the peak reading from the chart paper graph for the same rebars. Subtract any disparity in the two measurements from the calibration curve prior to setting the high and low limits and proceeding with the bridge deck evaluation. The corrected calibration curve should then be used for data interpretation.

Figure 2.—Operator calibrating instrument.



Rolling Pachometer—operator's view.

If a new deck is being placed, a simple measurement with the probe over a sample 6 in by 12 in (152.4 mm by 304.8 mm) by bridge deck thickness of concrete will indicate the amount of bias to subtract from the calibration curve. More accurate data will result if this procedure is followed.

In many cases, magnetite bias is negligible, and it is not necessary to apply a correction factor. Experience to date indicates the amount of error introduced by magnetite is generally less than .25 in (6.4 mm) when measuring rebars to a depth of 3 in (76.2 mm).

When it is considered impractical to core, the Rolling Pachometer will generate data within its accuracy specification of ±.25 in (6.4 mm).

Demonstration Project

Region 15 is currently touring various States throughout the country on Project 33, a demonstration of bridge deck evaluation techniques. Included in this project are two Rolling Pachometers. Complete documentation is available to enable State highway departments to build their own rolling pachometer or to have one constructed for them. Several industrial contractors have expressed an interest in manufacturing this device.

Conclusions and Recommendations

A study of comparisons in time and labor costs between the use of a Rolling Pachometer and the conventional hand-held pachometer was conducted by the Pennsylvania Department of Transportation. The study revealed that an identical number of measurements at four bridge locations required two men and a total of 6 days with the hand-held pachometer and only one man for a total of ½ day with a Rolling Pachometer. The amount of labor costs saved was estimated to be greater than 90 percent of that incurred with the hand-held device.

The Rolling Pachometer is an effective device for inspecting the cover on bridge decks. It produces a permanent record for each bridge which can be used for comparisons in future years. When used as a quality control tool, the depth of cover can be measured as soon as the concrete has hardened. If the cover is deficient, corrective action can be taken prior to placement of additional decks.

The Rolling Pachometer has proved to be accurate, repeatable, and reliable while gathering data at a rate 20 times that of conventional hand-held methods. It is recommended that this instrument be incorporated into bridge deck inspection procedures.

Permanent Bridge Deck Repair

by Ken Clear

—courtesy, Portland Cement Association.

Corrosion of reinforcing steel in portland cement concrete bridge decks due to deicer-borne chlorides has resulted in widespread bridge deck deterioration. When permanent repair of these deteriorated decks is desired, present knowledge dictates that all concrete containing chloride in excess of the corrosion threshold should be removed.

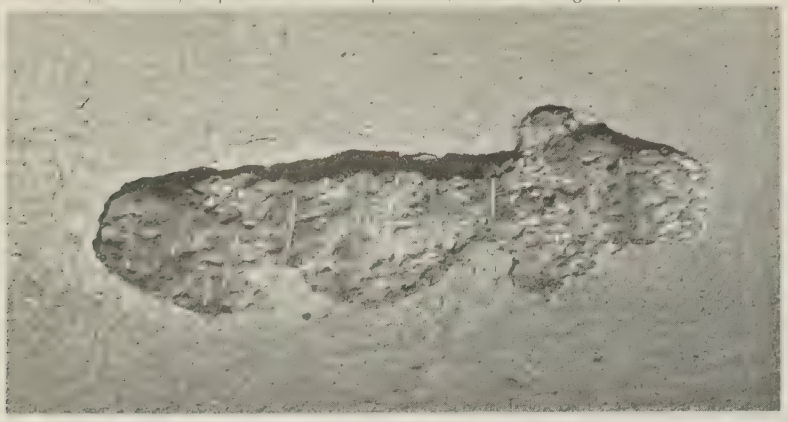
This article summarizes the discussions and recommendations of the interim report, "Evaluation of Portland Cement Concrete for Permanent Bridge Deck Repair" *(1)*.[1] The effect of chloride in reinforced concrete; the chloride content corrosion threshold; suggestions for determining the concrete that must be removed prior to repair; the use of a rotary hammer to obtain portland cement concrete samples for chloride analyses; and the wet chemical analysis procedure for total chloride developed by the Federal Highway Administration are summarized.

Introduction

Research has shown that the most prevalent cause of bridge deck distress is corrosion of the reinforcing steel due to the intrusion of chlorides into the concrete from repeated deicer applications for snow and ice removal. The damage results when the pressures created by corrosion induce delamination of the concrete near the level of the top mat of reinforcing steel and

[1] Italic numbers in parentheses identify the references on page 62.

—courtesy, Kentucky Department of Transportation, Bureau of Highways.



—courtesy, Virginia Highway & Transportation Research Council

Figure 1.—Rebar corrosion and spalling of bridge deck concrete.

Mechanical detector

subsequent spalling of the surface concrete. A similar problem exists in marine environments where retaining walls, piles, piers, and the undersides of bridge decks are exposed to the salts in sea water. Figure 1 shows this *bridge deck problem*.

The Federally Coordinated Program of Research and Development in Highway Transportation (FCP) hopes to find solutions. Waterproof membranes, epoxy-coated reinforcing steel, latex modified concrete, internally sealed concrete and polymer impregnated concrete are being developed. Cathodic protection, neutralization of chlorides, and polymer impregnation are being studied for application to existing structures.

Research has uncovered patching materials and overlay systems to aid in repair, but first the portion of the concrete that must be removed prior to patching and overlay must be determined. Pothole patching and bituminous overlay are used to temporarily restore a deck's riding surface. Removal

Chain drag

Figure 2.—Delamination detectors for defining areas of unsound concrete.

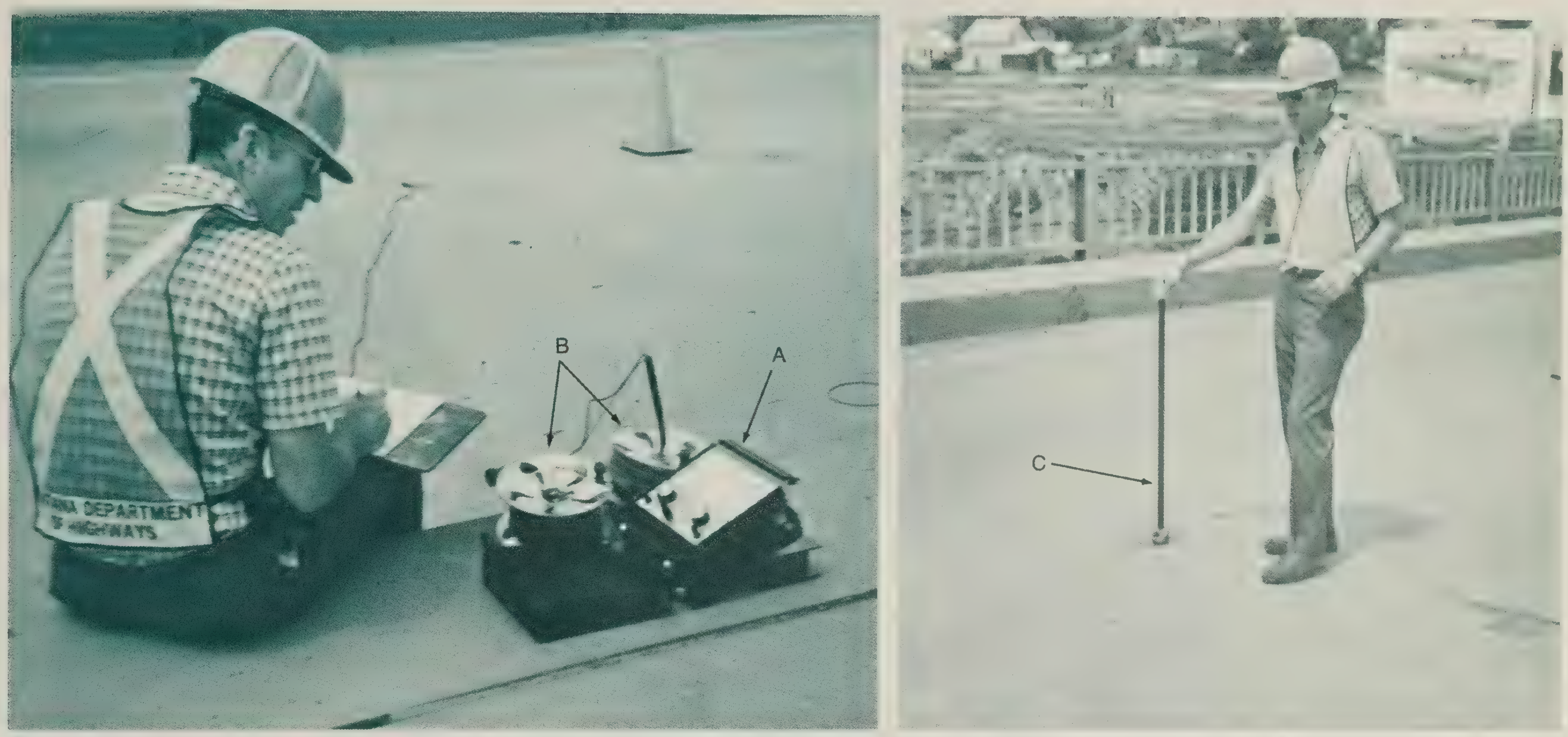


Figure 3.—Electrical corrosion detection device consisting of a very high impedence voltmeter (A), coils of lead wire (B), and a half cell (C); locates corroding rebar.

of the structurally unsound concrete or removal of only that concrete which surrounds corroding reinforcing steel (followed by replacement and overlay) are more expensive, but longer lasting procedures.

Although these procedures may be the most economical approach, they are temporary. Permanent repair is a procedure which will insure attainment of the design life of a bridge deck or other structure without costly future maintenance caused by rebar corrosion.

Need for Chloride Analyses

Chloride analyses are needed to define the areas of a deck where the conditions for corrosion are present even though no rebar corrosion is occurring. The tools generally used to determine the condition of a concrete bridge deck with respect to rebar corrosion do not measure this potential damage. The delamination detectors shown in figure 2 *(2)* will define the areas of loss of structural performance in the form of a cleavage plane within the concrete.

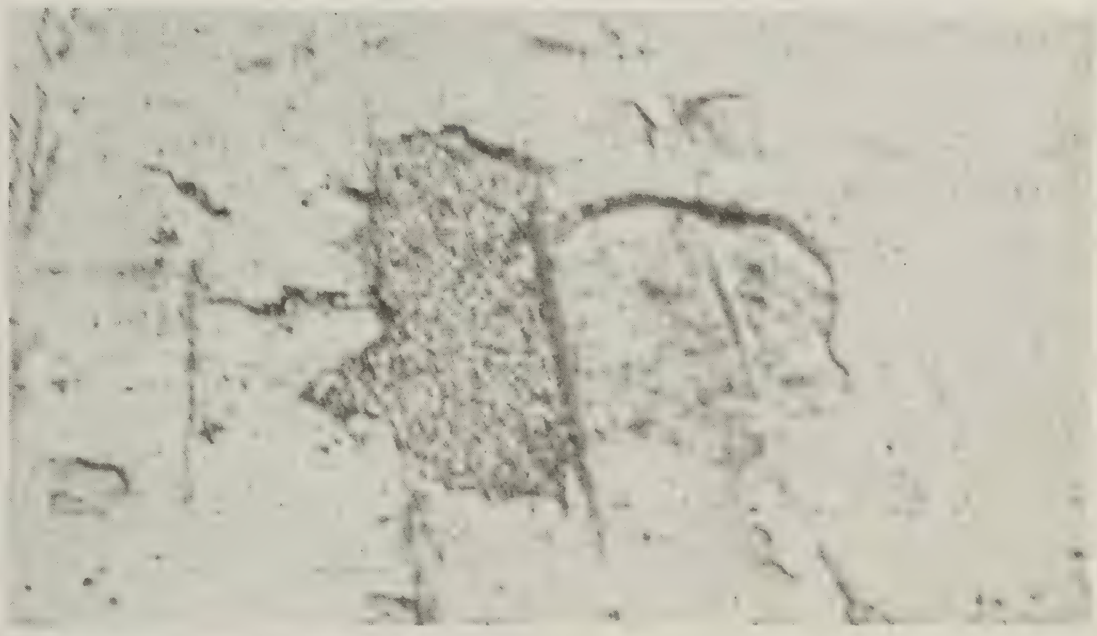


Figure 4.—Spall around a spall.

The electrical corrosion detection device shown in figure 3 aids in delineating the areas of active corrosion at a given time *(3, 4)*.

These tools may suffice if only a temporary repair and overlay to buy a few years time prior to reconstruction are desired. If permanent repair is planned, existing potentially destructive material should also be removed. This is stressed by the occurrence of previously sound concrete deteriorating around a patched area. For example:

> Concrete surrounding a corroding section of rebar is removed and replaced with an uncontaminated patch. The corrosion process starts anew in the chloride-contaminated but sound concrete around the patched area, and it is soon destroyed. Figure 4 shows this maintenance nightmare—the *spall around a spall.*

The theory that sealing the surface of a contaminated but sound concrete deck will prevent future deterioration by removing moisture and oxygen—two necessary elements in electrochemical corrosion of the reinforcing steel—is doubted by many on the premise that moisture and oxygen can penetrate the concrete deck from the underside. Thus, the only way to be assured of permanent repair is to remove all concrete in a potentially destructive condition, then prevent future deicing salts from reaching the reinforcing steel.

Effect of Chloride Ion on Reinforced Concrete

Chloride can be present in hydrated portland cement concrete in essentially three forms *(5)*:

1. Free chloride ion.
2. Chloride bonded strongly with the calcium silicate hydrates.
3. Chloride combined in compounds like the calcium aluminate chlorides.

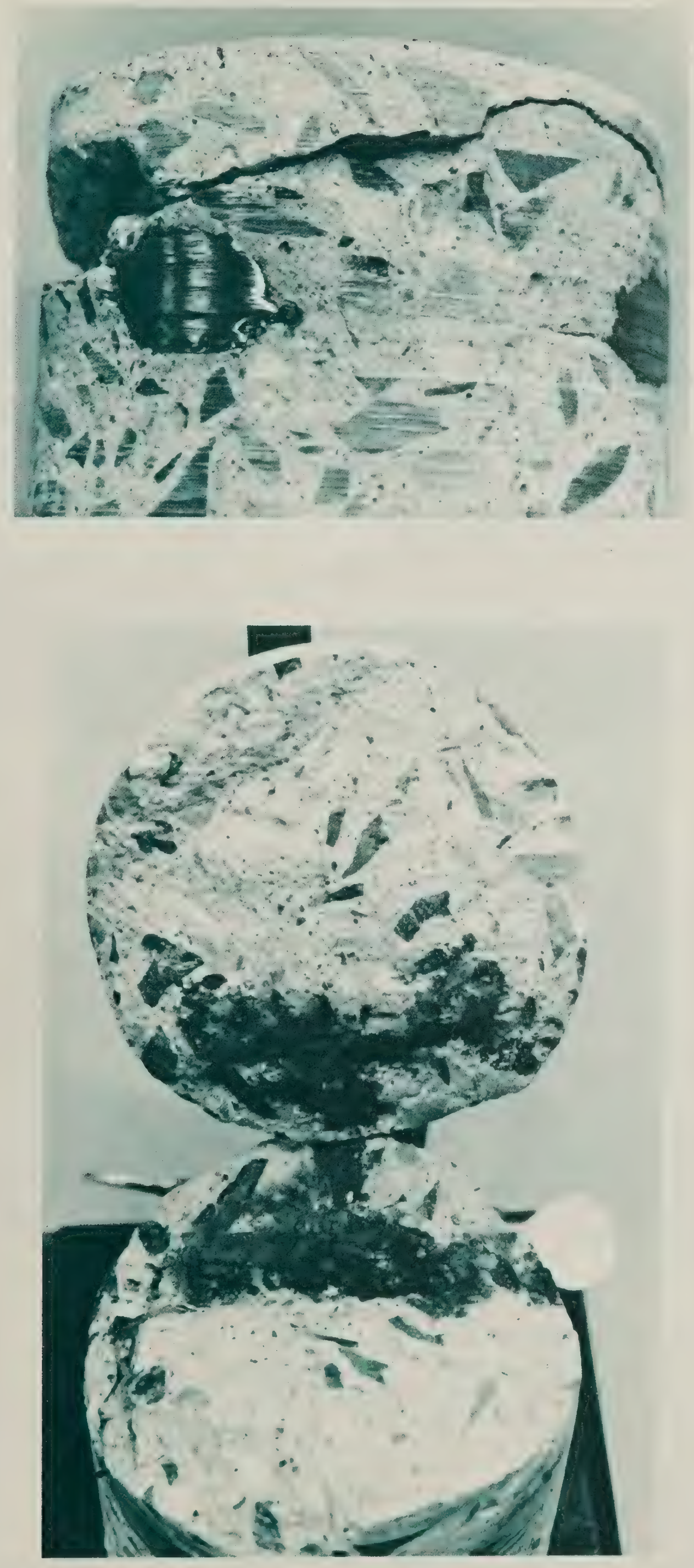
Free chloride can promote the corrosion process, and bonded chloride (item 2) may or may not cause corrosion depending on the strength of the bonds and other factors. It is believed that combined chloride (item 3) does not contribute to corrosion of reinforcing steel in portland cement concrete.

The effect of the chloride ion (item 1) is indirect. It does not attack reinforcing steel in concrete. Chloride does, however, provide the environment for the conversion of iron to iron (ferrous) oxide, the predominant form in which it occurs in nature. Reinforcing steel does not corrode in an uncontaminated (no chloride) portland cement concrete because of the high pH (12 to 13) afforded by the soluble calcium hydroxide (lime) liberated by the hydrating cement. This high pH initially forces the formation of ferric oxide (mill scale) on the surface of the steel. This oxide of iron is not detrimental because the reaction ceases after a surface film is formed and no further degradation occurs.

When soluble chlorides are introduced into the concrete, this protective high pH environment begins to neutralize. When sufficient chloride is present to destroy the necessary alkalinity and the normal passivity of the iron, the natural transformation of iron to rust occurs and an increase in volume occurs since the volume of the rust is greater than the volume of the parent iron.

This expansion causes tensile stresses to be exerted against the surrounding concrete. When these stresses exceed the tensile strength of the concrete, delamination and subsequent surface spalling occur. Figure 5 shows a core through the reinforcing steel of a 20 ft ^{2} (1.86 m ^{2}) concrete slab which has been subjected to daily applications of a 3-percent sodium chloride solution for 1.5 years. Note the corrosion buildup on the right side of the rebar and the concrete cracking which resulted. Similarly, the rebar in the core shown in figure 6 exhibits massive corrosion buildup to the point where delamination resulted.

Figure 5.—Corrosion caused concrete cracking.



What Does *Chloride Analysis* Mean?

The meaning of chloride analysis when referenced to a portland cement concrete depends on the forms of chloride that are being measured. Generally, there are two types of chloride analyses: Measurement of the free chloride, and measurement of the total chloride.

The ideal test method would be one that determined only the free chlorides (i.e., the more weakly bound chlorides readily available for destructive action) while ignoring that chloride which is combined. However, the division between free and combined chloride is not well defined and the resulting value for free chloride is very dependent upon the extraction medium and time. Therefore, it cannot be well defined.

The second option is to determine all the chloride present in a specific portland cement concrete and adjust the critical chloride content to compensate for the fact that all the chloride is not readily available to the corrosion process. This was the approach chosen by the Federal Highway Administration (FHWA) during development of the wet chemical analysis procedure for total chloride described in the FHWA report *(1)*. This approach eliminates the causes of variability in the chemical analysis portion of the work and reduces the variability between test methods, operators, and laboratories. It does not, however, eliminate adjusting the corrosion threshold (critical value of chloride required to induce corrosion) to reflect soluble or free chloride versus combined chloride.

Figure 6.—Corrosion caused delamination of the concrete at the rebar level.

When a wet chemical analysis is discussed here, it will refer to a procedure to determine total chloride in a normal portland cement concrete. The total chloride analysis procedure developed by the FHWA is the only test method recommended. The test method includes:

1. Weighing a 3-gram sample of the powdered material.

2. Adding distilled water and dilute nitric acid to dissolve the cement (and thus the chloride).

3. Boiling.

4. Filtering.

5. Titrating with silver nitrate and recording successive additions of silver nitrate and the corresponding change in voltage between the solution and a chloride selective electrode. Figure 7 shows the manual titrating apparatus normally used. However, the system can be automated when analysis of many samples is required. Figure 8 shows the automatic potentiometric titrator built in the FHWA laboratory. Nine samples are titrated simultaneously permitting a technician without chemistry training to titrate 75 to 100 samples per day.

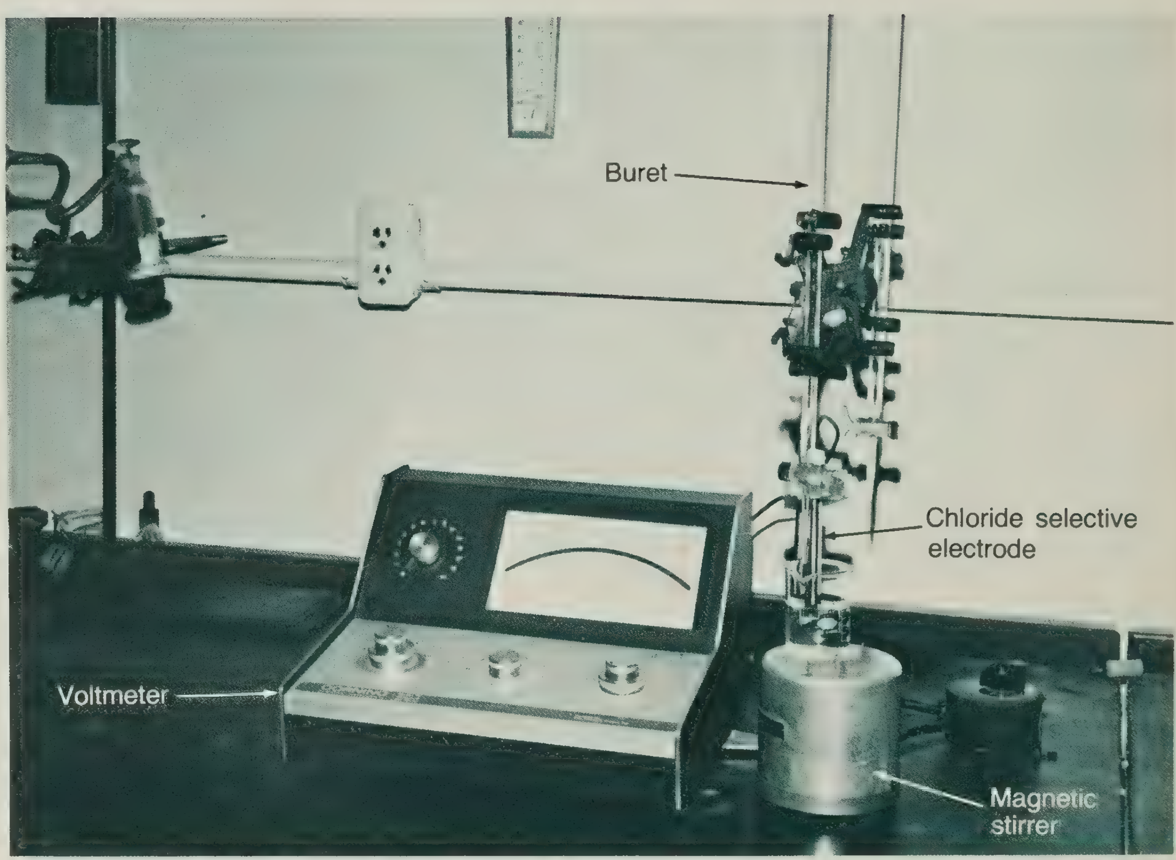


Figure 7.—Manual potentiometric titrator.

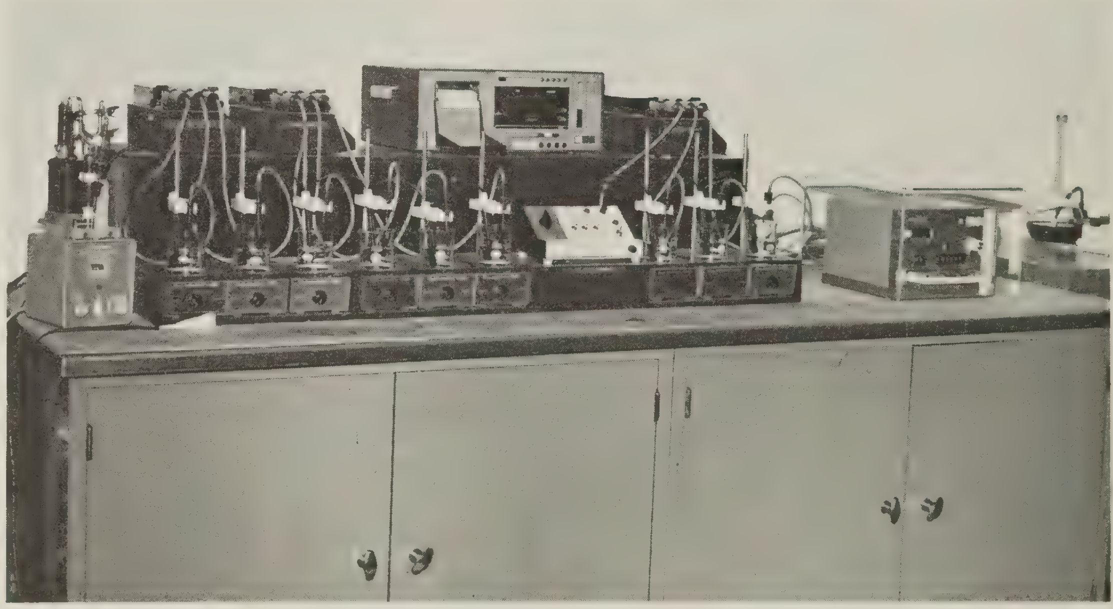


Figure 8.—Automatic titrator with magnetic tape recording of data for computer analysis.

If a different chloride analysis procedure is suggested, the accuracy of the method and the form of chloride being measured should be determined prior to using the results to determine the condition of a concrete bridge deck.

Chloride Content Corrosion Threshold

The chloride content corrosion threshold is the minimum quantity of *total* chloride required to initiate rebar corrosion in portland cement concrete when sufficient moisture, oxygen, and other necessary factors are present.

Establishing a universally applicable corrosion threshold is difficult. In a specific portland cement concrete, the chloride required appears to be dependent upon the initial pH of the concrete, the percent of the chloride present as soluble Cl^-, the quantity of cement, the moisture content, and other factors. It is impossible to determine all these factors for each concrete. However, a typical bridge deck concrete can be chosen, and upper and lower threshold limits determined based upon the range over which the parameters normally vary.

The total chloride content corrosion thresholds used in the FHWA laboratory are:

- CF=7.0 concrete (658 lbs cement/yd^3 −390 kg/m^3)

  Threshold=1.3 lbs Cl^-/yd^3=0.033% Cl^- in concrete

- CF=6.0 concrete (564 lbs cement/yd^3 −335 kg/m^3)

  Threshold=1.1 lbs Cl^-/yd^3 =0.028% Cl^- in concrete

The basis for these values is presented in the FHWA report *(1)*.

These values are the lower limit of the chloride corrosion threshold. A 75 percent soluble/total chloride ratio was used because this value results in a conservative threshold. Engineers generally agree that for most bridge deck concretes 50 to 75 percent of the total chloride is in the soluble (corrosion causing) form. If the 50 percent solubility ratio had been used, the total chloride content threshold values would be:

- CF=7.0 concrete—2.0 lbs Cl^-/yd^3 =0.051% Cl^- in concrete

- CF=6.0 concrete—1.7 lbs Cl^-/yd^3 =0.043% Cl^- in concrete

These values represent the approximate upper limit of the total chloride corrosion threshold. The varying solubility percentages do not alter the threshold enough to prevent the development of a usable test method.

Findings from a chloride analysis procedure which measures total chloride are compared directly to the threshold values. However, if a test method is used which measures only free chloride, the thresholds must be reduced before comparing them with the analysis results. It should also be kept in mind that the values are not in terms of the pounds of salt that are present. A little over 2 pounds (0.9 kg) of either sodium or calcium chloride is required to provide the equivalent to 1.3 lbs Cl^- (0.033% Cl^-).

Accuracy and Repeatability of the Wet Chemical Analysis for Total Chloride

Because of failure to recognize all the factors involved, many chemists and engineers have obtained poor results with other chloride analysis techniques and the statement "chloride can't be measured accurately" is often made. Because of the concerns expressed, several special studies of the accuracy and repeatability of the method have been conducted.

Tests of accuracy of the chloride determinations on cement paste were performed by FHWA *(5)*. A specific quantity of chloride was added to small portland cement paste specimens and the accuracy of the method was shown to be within 0.5 percent of the chloride present. For a paste with 1.0 lb Cl^-/yd^3 (0.0255% Cl^-), the 0.5 percent would be only 0.005 lb Cl^-/yd^3 (0.0001% Cl^-); obviously an excellent accuracy.

The completeness of the nitric acid extraction on concrete was also tested *(5)*. It was estimated that the test method defined the total chloride within 3 percent. For a concrete at the threshold, 1.0 lb Cl^-/yd^3 (0.0255% Cl^-), this translates to a variation of only 0.03 lb Cl^-/yd^3 (0.0008% Cl^-); a sufficient completeness of extraction.

Although the accuracy on concrete is difficult to measure, the repeatability is easily measured by dividing a specimen into parts and performing a chloride analysis on each part.

Researchers at Battelle Memorial Institute [2] performed repeatability tests on 10 specimens each of which was split into four parts (samples). A chloride analysis was then performed on each part. The findings (table 1) showed that the maximum difference between any of the four samples taken from a single specimen was only 0.19 lb Cl^-/yd^3 (0.0048% Cl^-), while on the average

[2] D. R. Lankard, P. J. Moreland, et al., "Neutralization of Chlorides in Concrete," Battelle Memorial Institute, First Quarterly Progress Report on Contract DOT–FH–11–7673 with the Federal Highway Administration, November 1973.

Table 1.—Repeatability of chloride determinations on portland cement concrete

Specimen No. [1]	Chloride content,[2] lbs Cl^-/yd^3 Sample 1	Sample 2	Sample 3	Sample 4	Range (Maximum difference) lbs Cl^-/yd^3
1	5.65	5.61	5.73	5.69	0.12
2	5.22	5.18	5.37	5.22	0.19
3	4.71	4.75	4.82	4.67	0.15
4	4.47	4.59	4.31	4.47	0.16
5	2.39	2.39	2.35	2.31	0.08
6	2.27	2.31	2.24	2.31	0.07
7	1.02	0.94	0.98	0.94	0.08
8	0.94	0.94	0.94	0.94	0
9	0.75	0.71	0.78	0.78	0.07
10	0.67	0.67	0.63	0.67	0.04
				Average	0.10

[1] 1-in thick, 4-in diameter concrete sections which were pulverized and split into four parts. A 10-gram sample was then obtained from each part.

[2] The chloride contents (% Cl^- in concrete) reported by Battelle Memorial Institute were converted to lbs Cl^-/yd^3 using the relation 1.0 lbs Cl^-/yd^3=0.0255% Cl^- in concrete.

1 in.=25.4 mm

the maximum difference was 0.10 lb Cl^-/yd^3 (0.0026% Cl^-). This repeatability is satisfactory when the method is used to define the total chloride content of bridge deck concrete.

These repeatability tests were performed by one operator. To measure the effect of different operators on method reproductibility, two 50-gram specimens were split into four and six parts, respectively, and the samples were run by two operators. The findings (table 2) showed that for chloride contents near the corrosion threshold, the maximum difference for single results by the two operators was 0.09 lb Cl^-/yd^3 (0.0023% Cl^-).

Thus, the wet chemical analysis test method for total chloride in concrete is sufficiently accurate and repeatable to provide data useful to the engineer evaluating the condition of a concrete bridge deck.

Table 2.—Repeatability tests—different operators

Specimen No.	Sample No.	Chloride content Operator 1 (chemist)	Operator 2 (civil engr.)
		lbs Cl^-/yd^3	
1	1	1.55	
	2		1.56
	3	1.59	
	4		1.62
Maximum difference = 0.07 lbs Cl^-/yd^3			
2	1	1.53	
	2		1.44
	3	1.51	
	4		1.50
	5	1.50	
	6		1.48
Maximum difference = 0.09 lbs Cl^-/yd^3			

1 lb Cl^-/yd^3 = 0.0255% Cl^-

Obtaining PCC Samples for Chloride Analysis

Problems are involved in obtaining samples for chloride analysis. One is to define the location and number of samples to be taken from a given bridge deck. A second is the time and cost required to core, section, and pulverize the concrete or mortar prior to chloride analysis. This limits the number of samples which can be run, and the desire to perform the test.

Location and number of samples

It is difficult to establish a specific number of samples which should be used to document the condition of a concrete bridge deck prior to permanent repair. Each structure may be unique. The following interim guidelines[3] can be suggested.

Using chloride analyses as the sole evaluation technique to define the areas of concrete which must be removed for permanent repair is not recommended because of economics. Two other tools, a delamination detector (chain drag, hammer, sounding rod, or mechanical sensor) to locate undersurface fractures, and a corrosion detection device to define areas of active steel corrosion, are nondestructive, more rapid, and more economical. These nondestructive techniques will validly define a portion of the concrete that must be removed. Chloride analyses can then be made on the balance of the deck to fully define the contaminated areas.

[3] These guidelines represent the recommendations of the FHWA Office of Research. However, the discussion does incorporate guides which FHWA field offices have received and offers other guides discussed with the Washington operating offices.

A pachometer is needed to nondestructively document the depth of the reinforcing steel at each sampling site (see figure 9).

Visual surveys of the top and bottom surfaces of the deck reveal the probable overall deck condition. Measurement of the spalled area is of little value since each spall is only the product of a more extensive condition.

Detailed analysis procedures for decks in various conditions are presented in the FHWA report *(1)*. Briefly, in addition to a visual survey, the procedures include:

1. Perform an electrical potential survey to define the areas with potentials greater than 0.35 volts to the copper-

Manual device

Figure 9.—Pachometers for determination of reinforcing steel depth.

copper sulfate electrode. Designate these areas as concrete removal sites.

2. Perform a delamination survey on the area of the deck with potentials less than 0.35 and designate the unbonded areas as concrete removal sites.

3. Determine the chloride content of rebar level concrete in the areas of the deck not yet designated for removal. That concrete with rebar level chlorides above the corrosion threshold must also be removed.

The number of chloride determinations required will be dependent upon the findings of the visual, electrical potential, and delamination surveys. For badly deteriorated decks, the need for reconstruction may be obvious and no chloride analyses are required. For decks with few visual signs of deterioration but widespread delamination and/or corroding reinforcing steel, a few chloride analyses are suggested to confirm the previous findings prior to recommending reconstruction.

Rolling pachometer

Similarly, for decks with no delaminations, spalls, or areas of corroding rebar, only a limited number of chloride determinations are suggested during the initial survey to confirm that the Cl^- content is below the corrosion threshold. However, if a portion of these initial samples show high chlorides, additional Cl^- analyses will be required to define the extent of the contaminated concrete. This concrete should then be removed and replaced prior to the installation of a water proof membrane or other protective system. Decks whose condition falls between these two extremes will undoubtedly require massive determination of the rebar level chloride in the concrete and removal of high chloride (greater than corrosion threshold) concrete if permanent repair without reconstruction is to be realized.

Electrical potential surveys cannot be validly performed on some bridge decks because the rebars are not electrically continuous. A visual survey, delineation of the areas of delaminated concrete, and approximately 10 chloride determinations spaced across the deck in sound concrete can be used to provide initial insight into the deck's condition. On this basis, areas for additional coring and chloride analyses can be defined.

Sample acquisition

Conventionally, a water-cooled coring rig is required to core a concrete bridge deck. The diamond core bits are expensive. Two operators are usually required, and an average of 20 minutes per core is common. The additional effort to slice and pulverize the samples is considerable. To prepare a sample for chloride analysis may take 2 to 3 man-hours.

A rotary hammer technique will reduce the time required to obtain and prepare a PCC sample for chloride analysis to less than 15 minutes. The equipment is portable and will fit easily in the trunk of a car. The total cost is approximately $400. Coring (pulverizing) is done without water so no soluble chlorides are removed from the concrete.

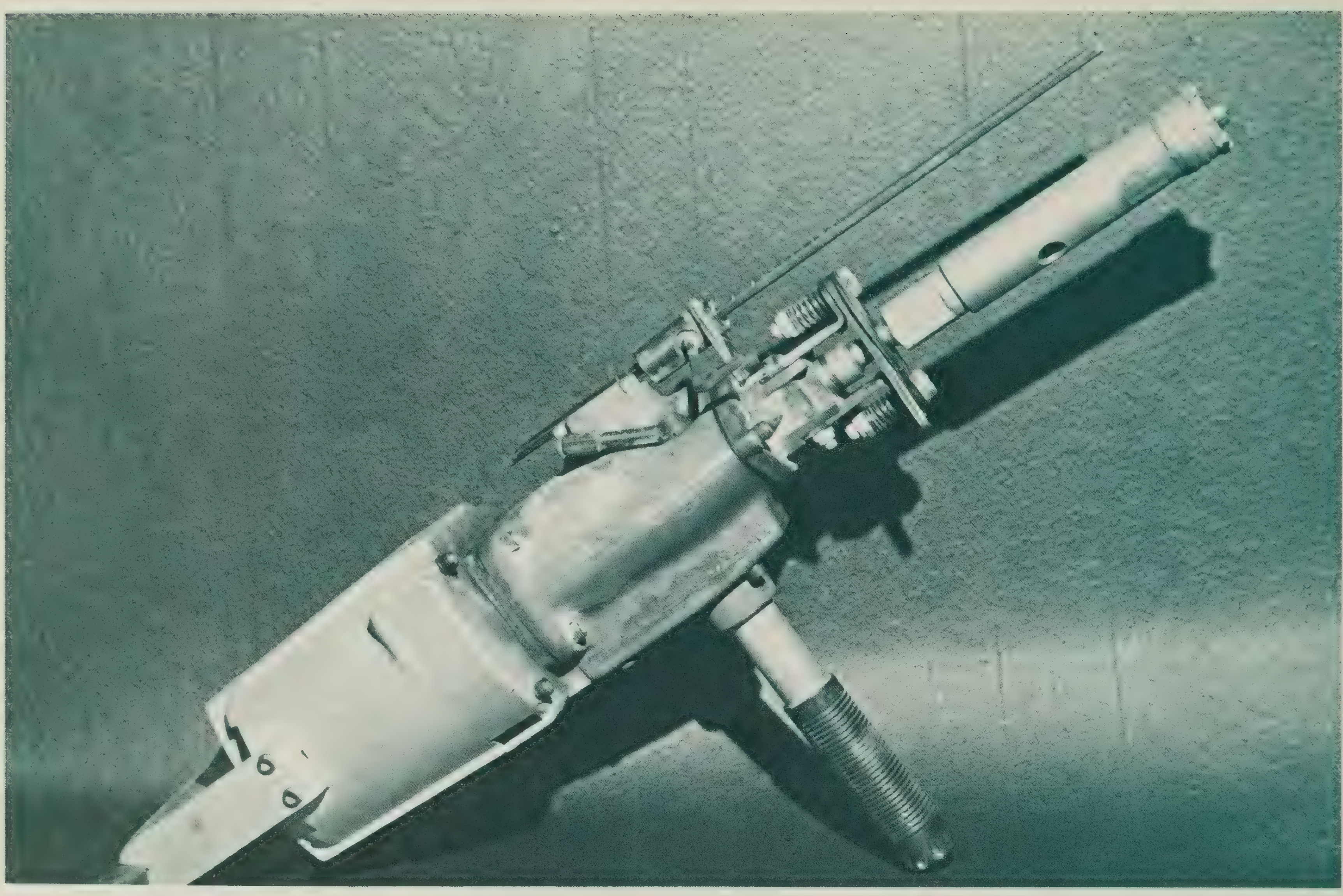


Figure 10.—Rotary hammer used to quickly obtain samples for chloride analysis.

The concrete is pulverized by the hammering and rotating actions of the core bit and a carbide-tipped starter bit positioned inside the core bit. Use of the core and starter bit results in the maximum degree of pulverizing and keeps the major portion of the pulverized material inside the core hole until the desired core depth is reached. The hammering action and bit rotation break up the concrete rapidly—a 1-inch (25 mm) depth in normal structural class concrete can be reached in 30 seconds.

The powdered concrete in the core hole is then collected and in some cases is ready for analysis. In other instances, a slight amount of additional pulverizing (5 minutes by hand) is required before the sample will completely pass a No. 50 (300 μm) mesh screen. An indicator attached to the hammer controls the coring depth, and a vacuum cleaner is used to remove unwanted pulverized material above the sampling depth. The hammer is shown in figure 10.

Summary

Corrosion of the reinforcing steel in portland cement concrete bridge decks due to the intrusion of deicer-borne chlorides has resulted in widespread bridge deck deterioration. When permanent repair of a deteriorated deck is desired, at present, the engineer's objective must be: Delineate and remove all chloride contaminated concrete and prevent future deicing salts from reaching the reinforcing steel. Use of a rotary hammer to quickly obtain samples and the wet chemical analysis for total chloride test method to accurately define the quantity of chloride in the deck will accomplish the delineation portion of this objective.

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Sawdust as Lightweight Fill Material

by David S. Nelson and William L. Allen, Jr.

Introduction

Unstable embankments and potential slide areas have always been an expensive maintenance problem. The Washington State Highway Department recently developed a unique solution to this problem by utilizing a new kind of fill material that is a lightweight waste product: ***sawdust.*** This lighter weight material reduces the driving weight of the slide mass and in some cases results in an appreciable unloading of the landslide *(1)*.[1]

Sawdust has been used for many years as a lightweight fill material for reclaiming peat and swamplands. The British Columbia Department of Highways used sawdust to traverse a peat deposit with their Burnaby Freeway *(2, 3)*. In Norway, sawdust was used instead of conventional embankment fill on a road built over 30 feet (9.1 m) of peat where severe settlements had occurred *(4)*. The City of Aberdeen, Wash., has used sawdust mixed with quarry rock as a base material for many of their city streets. In all cases, however, the sawdust remained below the water level after settlement and therefore was not subject to the decay process.

Experimental Repair Material

In March 1972 a major landslide occurred on U.S. 101 south of Cosmopolis, Wash., which forced closure of the highway (fig. 1). A 200-foot (61.0 m) section was lost as 1½ lanes of this two-lane highway slid downward 40 feet (12.2 m). A temporary detour was set up while the repair crews went to work. Because of the unusually heavy rainfall in this area—125 inches (3,175 mm) per year—it was decided that an all-weather material must be used to fill the hole left by the slide.

Here seemed to be the perfect opportunity to try out this relatively new type fill material. Since placement of sawdust is not altered by rainy weather conditions and since weight is such a critical factor in slides of this nature, sawdust appeared to fit the need for a well sought after repair material (fig. 2). It also served a second purpose: Timber companies got rid of a troublesome waste product that, because of environmental laws, could no longer be burned. Because this was the first time sawdust was used in a roadway section above a water table, the Washington State Highway Department developed a "wait and see" attitude.

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

Figure 1.—State Route 101 closed because of major slide—March 1972.

Figure 2.—Sawdust used for emergency repair of this highway.

Figure 3.—Preconstruction view of lightweight fill section. Crack line indicates sliding.

Figure 4.—Layered pavement sections (maintenance overlays).

Experimental Soil-Stabilizing Material

After a year's exposure, this experiment in slide repair still remained intact without any noticeable signs of deterioration or settlement. Thus it was decided to expand the use of sawdust as a lightweight fill material to other areas where the highways' constant slipping had been a continual maintenance problem. One potential slide area just south of the 1972 emergency repair site had been slowly slipping downward for quite a few years. Each year maintenance crews would come in and add asphalt surfacing to bring the riding surface back to grade. This additional dense surfacing compounded the problem by putting more driving weight on the slide mass thus inviting still further slippage (fig. 3).

Soil surveys have shown that the slippage in this area is due primarily to water-susceptible clay shales on top of tilted sedimentary bedding surfaces. As the heavy rainfalls permeate these surfaces, the water acts as a lubricant and makes the whole highway section unstable. Attempts to remove the water from the highway section with horizontal drains have not been effective enough to remove all the water.

The lightweight fill construction was included in a Washington State contract for repairing and resurfacing State Route 101 from Cosmopolis to State Route 107. This entire highway was closed to traffic for 1 week (July 9–14, 1973) to allow complete excavation and backfill of the lightweight fill section. The contractor excavated the 250-foot (76.2 m) roadway section to a depth of 10 feet (3.0 m). The asphalt pavement section, layered from years of maintenance overlays, was observed during removal to be 4 feet (1.2 m) thick (fig. 4).

Figure 5.—Sawdust being dumped into the excavated site.

Figure 6.—Hog fuel.

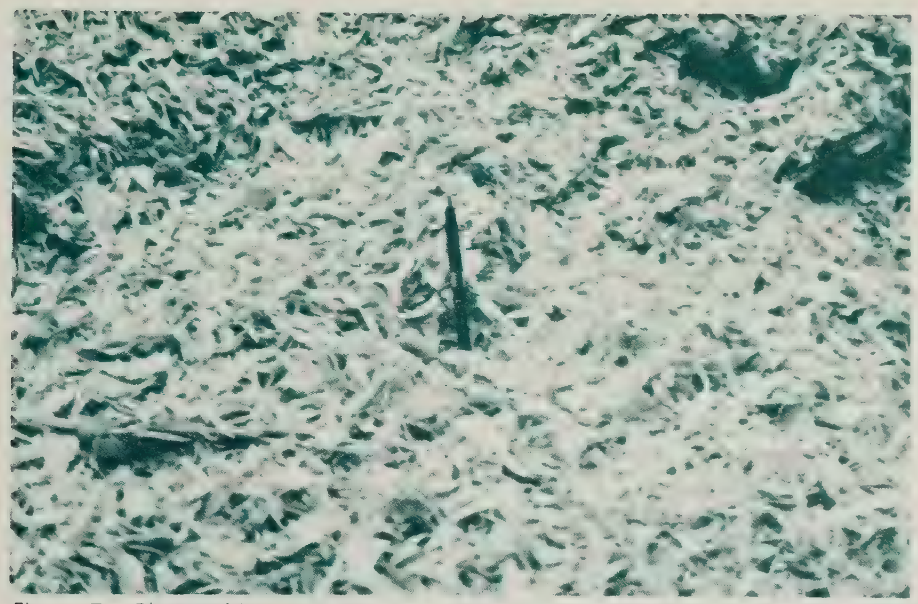

Figure 7.—Planer chips.

Figure 8.—Bark chips.

Figure 9.—Sawdust was placed in 1-foot lifts and was compacted with construction equipment in a predetermined pattern.

While the roadway section was being excavated, sawdust was delivered to a stockpiled site at the rate of 60 cubic yards (45.9 m^3) per hour. At the end of the second day of hauling, 70 percent of the required 3,650 cubic yards (2,790.6 m^3) of sawdust was at the construction site. On the third and fourth days, as the stockpiled material was being placed, additional trucks end-dumped the sawdust directly into the excavated site (fig. 5)

The only specification requirement for the lightweight material was that it be 100 percent wood fibers with no particle exceeding 6 inches (152 mm) in its maximum dimensions. The sawdust was of three varieties: hog fuel (fig. 6), planer chips (fig. 7), and bark chips (fig. 8). The hog fuel—pulverized bark that high-pressure water has stripped off logs—was a byproduct of a local pulp mill. The planer chips and bark chips were byproducts of a local sawmill.

The lightweight material, placed in 1-foot (0.3 m) lifts, was compacted by the action of construction equipment driving over it in a predetermined pattern (fig. 9). No water was required during placement and only minor effort was needed to attain maximum compaction. Because preliminary observations indicated the moisture-density curve very flat over most of the spectrum of the curve, no field compaction control tests were taken. As the material was compacted, it appeared quite resilient; that is, it would deflect 2 to 3 inches (51 to 76 mm) under the load of construction equipment and then spring back partially each time the load was released. However, as each additional lift was applied, the magnitude of deflection or resiliency under the compacting load remained constant: 2 to 3 inches (51 to 76 mm).

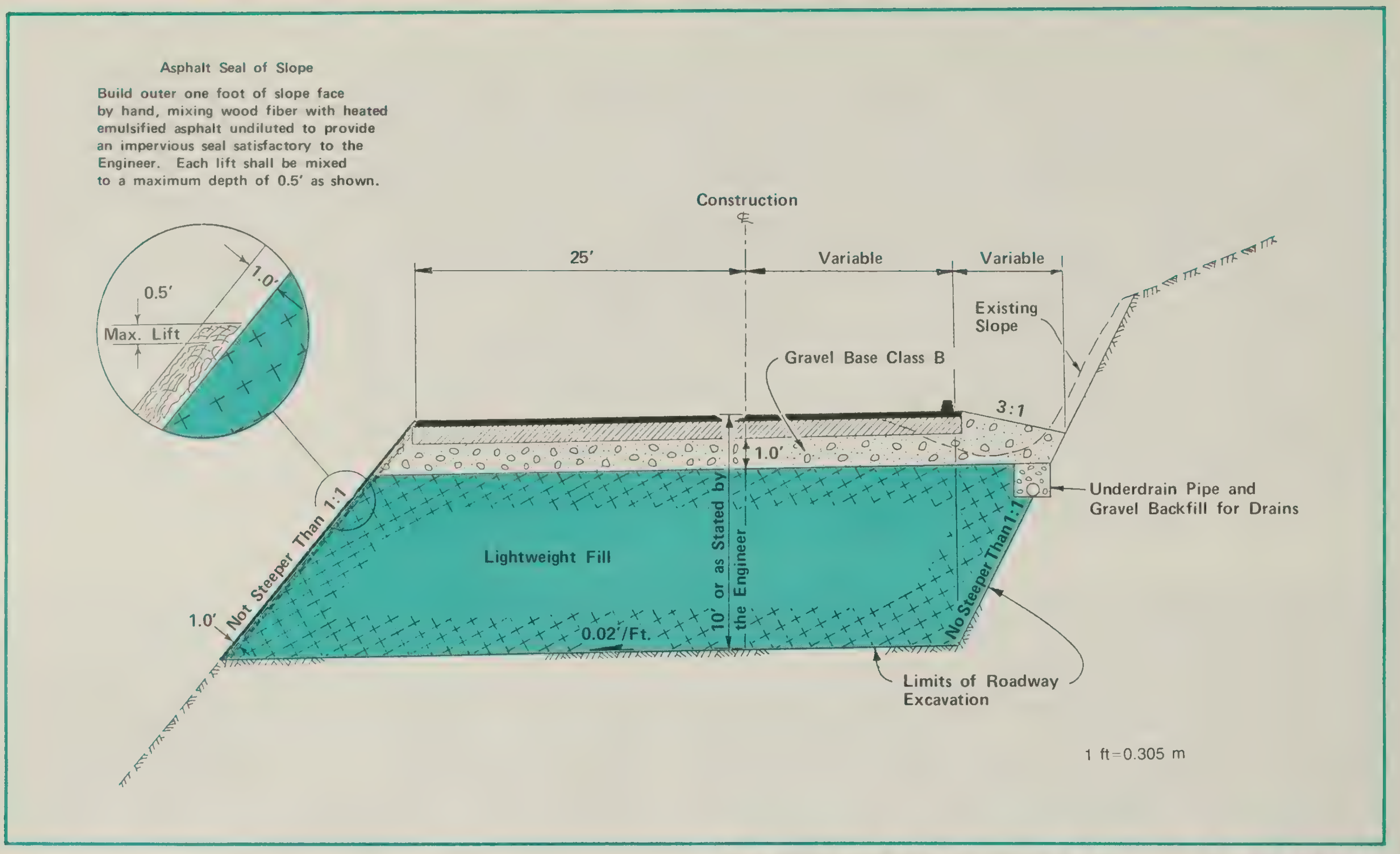

Figure 10.—Excavation and lightweight fill detail.

To keep air from deteriorating the sawdust, an emulsified asphalt sealer was applied on the 1:1 fill slope with each lift of sawdust (fig.10). The sealer was sprayed over the 2-foot (0.6 m) width at a rate of 1.3 gallons per square yard (5.9 l/m^2). Using pitchforks, laborers mixed the wood fibers with the heated asphalt along the slope to the required 1-foot (0.3 m) sealer widths (fig. 11). A small tractor then compacted the slope and spread more sawdust over it to repeat the process. The seal was designed to be somewhat permeable and allow trapped water to escape, thus reducing any hydrostatic pressures.

It appears the 125-inch (3,175 mm) annual rainfall has a dramatic effect on the water table which, in turn, has a significant effect on the embankment stability of the highway section. Slope stability calculations indicate that if the water table reached 10 feet (3.0 m) below the as-constructed pavement surface, sliding will occur and after years of maintenance repair overlays the water table need only reach 15 feet (4.6 m) below the pavement surface to effect sliding. Reconstructing the highway section with a 10-foot (3.0 m) layer of sawdust places the factor of safety of the embankment well above the sliding break point of 1.0.

Figure 11.—On the fill slope, an emulsified asphalt sealer was applied with each lift of sawdust while in the background laborers hand mix the wood fibers with the heated asphalt.

Sawdust Characteristics

Deterioration of the fill material is a problem of major concern, but sawdust piles over 20 years old in the western part of Washington have been observed to be still intact. Apparently the outer layer of the sawdust pile deteriorates and forms a *crust* that keeps air from reaching the interior of the pile. Similarly the asphalt-sealed slope will act as a crust that inhibits air from causing deterioration, and should give the material a life expectancy of at least 14 to 15 years as a stable fill. Norman D. Lea and Associates, who did some of the research in Canada's use of sawdust as a fill material, estimates sawdust above a water table to last at least 15 years.

The possible effects of leachates on the environment is an important consideration. If sawdust is buried too close to a flowing stream, removal of the nutritive elements of water—namely oxygen—will cause a pollution that is detrimental to underwater life. Thus an impermeable material must be used to seal off the harmful effects of sawdust when these conditions demand it.

A 1-cubic-foot (0.028 m^3) bucket was used to informally determine the density of the sawdust material. One cubic foot (0.028 m^3) in an uncompacted state weighed 23.7 pounds (10.8 kg). When compacted by techniques similar to the actual construction compaction, the material weighed 36.3 pounds (16.5 kg). If the average unit weight of the excavated material is assumed to be 140 pounds per cubic foot (2,242.6 kg/m^3), the excavated driving weight would approximate 6,899 tons (6,259 tonnes). The weight of the sawdust material put in its place would be 1,971 tons (1,788 tonnes). The comparison of the two weights leaves no doubt as to the distinct advantage of sawdust in fill areas: A reduction of 4,928 tons (4,471 tonnes) or 71 percent in the driving weight of a potential slide.

The laboratory moisture-density curve of hog fuel was very flat over most of the spectrum of the curve. This was to be expected and was the main reason compaction was not a critical factor in the placement of the fill material. Laboratory test results of the triaxial tests on the hog fuel established the angle of internal friction at 31° with a cohesion of 0 pounds per square inch. The permeability of the material was 1.0×10^{-3} cm/sec which is a reasonably good drainage property.

Paving Over Sawdust

Class B gravel base was placed on top of the compacted sawdust in two, 6-inch (152 mm) lifts. The material was graded level with conventional equipment and then compacted with a steel-wheel roller in a standard construction method. The base material behaved as if it were on a conventional fill material and showed no signs of excessive deflection under the roller load. The fibrous intertwining of the wood particles tends to disperse the load horizontally and thus improves the stability of the base material.

The remainder of the pavement section was placed by conventional means with a 0.5-foot (152 mm) asphalt-treated base placed in two lifts. A box spreader was used to place the material and a steel-wheel roller compacted each lift. The 0.15-foot (45.7 mm) Class B surfacing was placed on top of the asphalt-treated base for the wearing surface.

A total of 3,650 cubic yards (2,790.6 m^3) of sawdust was placed at a unit bid price of $2 per cubic yard ($2.62/m^3). This bid item was full compensation for placing and compacting the fill and asphalt sealing the slope. Because sawdust is a waste product, it was delivered to the construction site at no charge to the contractor. Excavation of the lightweight fill section was as *Roadway Excavation Including Haul* at a bid price of $2.50 per cubic yard ($3.27/m^3) and, of course, the base and surfacing were paid at their own unit costs.

The Federal Highway Administration (FHWA) Washington Division Office has documented the actual construction sequence in a 15-minute color narrated 16-mm film entitled "Sawdust as a Lightweight Fill Material." The movie is supplemented by a report *(1)* describing in detail the concept of using sawdust in fill areas. This report was a staff study by the FHWA Implementation Division in cooperation with the FHWA Washington Division Office. The movie is available from the FHWA regional offices (see page 87) or from the U.S. Department of Transportation, Federal Highway Administration, HHO–35, Washington, D.C. 20590.

Conclusions and Recommendations

In conclusion the following points should be emphasized:

- Sawdust can reduce the driving weight of a potential slide by as much as 71 percent.
- Sawdust needs very little compactive effort, only the action of construction equipment driving over it.
- From all indications sawdust fills can sustain roadway sections for 15 years or longer.
- The use of sawdust above the water table should be based on economics and availability, comparing the cost of rehabilitation after the lifetime of the material with alternate solutions.

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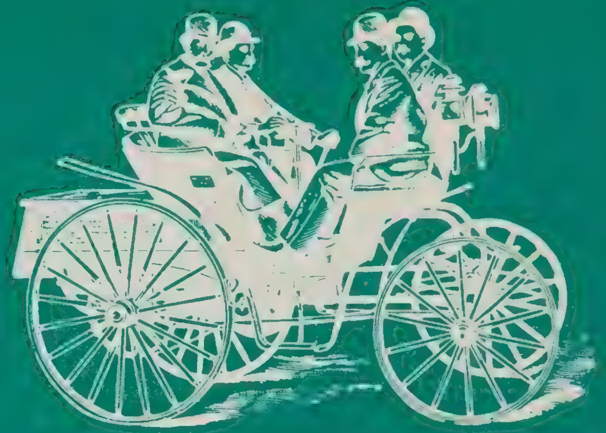
Highway Design for Motor Vehicles— A Historical Review

by[1] Frederick W. Cron



Part 3: The Interaction of the Driver, the Vehicle, and the Highway

This is the third in a series of eight historical articles tracing the evolution of present highway design practices and standards in the United States. The Introduction and Part 1: The Beginnings of Traffic Measurement were published in vol. 38, No. 3, December 1974. Part 2: The Beginnings of Traffic Research was published in vol. 38, No. 4, March 1975. The remaining parts, to be published in future issues, are 4: The Vehicle-Carrying Capacity of the Highway; 5: The Dynamics of Highway Curvature; 6: Development of a Rational System of Geometric Design; 7: The Evolution of Highway Grade Design; and 8: The Evolution of Highway Standards.

Width of Early Roads

Since ancient times most vehicular roads have been wide enough for two lines or lanes of traffic so that vehicles could pass each other without leaving the road. The Roman *via* was generally 12 Roman feet (11.7 English feet) wide when straight or 16 Roman feet wide when crooked. These widths were scaled to the widths of Roman vehicles which, according to the evidence of ruts worn in pavements still preserved in parts of the Empire, varied in gage from 3.8 to 4.5 Roman feet (*1*).[2] Thus, the 12-foot road width would permit two vehicles with their projecting hubs to pass with a center clearance of about 1 foot. On crooked roads the extra width of about 4 Roman feet was needed to provide room to maneuver heavy wagons, to which as many as 10 horses might be hitched in pairs ahead of the vehicles.

In 1915, 16 feet was considered an adequate width for a rural road in the United States. At that time freight wagons had a wheel gage of 60 inches and carriages a gage of 56 inches, and they were drawn by single horses or pairs of horses at speeds seldom exceeding 6 mph. For this traffic a 16-foot road was wide enough for two vehicles to meet without the need for either of them to run onto the shoulder.

The roads of this period were built with high crowns, and the amount and shape of the crown were subjects of considerable controversy among road engineers. High and steep crowns shed water quickly—which prevented softening of the road—but were uncomfortable to drive on. Consequently, most traffic ran in the middle, straddling the crown, except when meeting or passing other vehicles, and this caused the center to wear much more rapidly than the edges. In time, horsepaths and ruts might form in the middle of a broken stone road from wear; these would hold water and thus defeat the original purpose of the steep crown. When the crown was reduced to ½ inch per foot (4.2 percent) or less, a much better distribution of traffic resulted and wear was more uniform. Increasing the width of surfacing tended to distribute traffic better, and distribution also tended to be better on heavy traffic roads, because there was less opportunity to ride the middle (*2*).

The traffic-carrying capacity of roads and surface types was widely judged in terms of surface wear and maintenance expense. Thus, 400 vehicles per 10-hour day was regarded as the practical capacity of a 12-foot waterbound limestone macadam road; and 2,500 vehicles per day the capacity of an 18-foot waterbound traprock macadam road. By comparison, a 14-foot bituminous macadam surface would sustain "with satisfaction" 5,000 vehicles per average day (*3*).

After 1900, motor cars appeared in considerable numbers on the highways, running at speeds of 15 to 20 mph. People began to worry about the safety of vehicles passing each other at such speeds on the narrow roads. However, until about 1916, rapidly increasing volumes of motor traffic ran with reasonable safety on these roads, and it was not until motor trucks began to use the rural roads that narrowness became critical. This first became apparent in breakdown of roads at the edges.

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[1] Frederick W. Cron's biography appeared with part 1 of his article in vol. 38, No. 3, December 1974.

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

Roads — History
Farm, Frederick W.
History, Road
Motor vehicles — History

W. D. Sohier of Massachusetts remarked in 1919 that "It has been found everywhere there was heavy truck traffic that a 14 to 16 foot width of hardened surface was insufficient on main highways; that at least 18 feet was necessary or it sheared off on the edges"(*4*).

Trucks of this period with wheel gages of 69 inches and overall widths of 84 inches were considerably wider than either automobiles or freight wagons. Their speed—10 to 12 mph—was slow compared to autos, so they were frequently overtaken. Road users began to demand wider pavements for more comfortable passing and also legal control of the sizes of trucks.

In 1914 New York limited the overall width of vehicles to 90 inches. Michigan permitted no vehicles which exceeded 75 inches in wheel gage or 96 inches in overall width.

After World War I progressive States moved rapidly to a paved width of 18 feet with 5-foot shoulders, and by 1920 this was practically a standard for main roads in the United States. Less important roads continued to be built to the old narrow widths. The standard width for roads in the national forests, for example, was 18 feet shoulder-to-shoulder in easy terrain, or 12 to 13 feet in the mountains (*5*).

Lateral Distribution of Traffic on Road Surface

A novel study, possibly the first, of the distribution of traffic on a road surface was made in New York City in 1917. Certain streets were divided into 2-foot lanes by painted lines. The positions of moving vehicles with respect to these lanes were then observed and the heaviest-trafficked portions of the streets were noted. These observations showed that drivers tended to avoid the roughest portions of the street and use the smoother portions—even if these happened to lie between the streetcar tracks, as was often the case. Parked vehicles also had a marked effect on road use (*6*).

One of the earliest attempts to control the behavior of vehicle operators on the highway was the painting of a white centerline on curves in Marquette County, Mich., in 1920. These lines were handpainted with whitewash and lasted only a month on the road, but were very effective for keeping drivers in their own lanes (*7*). Two years later Wayne County, Mich., devised the first truck-mounted continuous line striper, a machine operated by two men, that could paint 6 to 7 miles of stripe per day. (Previously, it took four painters a full day to handpaint 1 mile of stripe.) (*8*).

In 1924, J. T. Pauls of the Bureau of Public Roads applied the New York City technique to determine the lateral positions of vehicles on rural roads of different widths (*9*). He divided the pavement into 1-foot strips and observed how many rear-wheel passages each strip sustained. This study produced some surprising discoveries:

- Drivers would not run with their outer wheels close to the edge of the pavement. Car drivers held back about 2.5 feet and truck drivers about 1.5 feet.
- As the width of pavement increased, drivers increased their clearance from the right edge, and for 24-foot pavements edge clearance was as much as 4 feet.
- Low or rough shoulders, ragged pavement edges, guard rails, curbs, culvert headwalls, or obstructions close to the pavement edge caused drivers to increase their edge clearance. In one case an underpass abutment at the edge of a 20-foot pavement narrowed the effective width to 18 feet.
- Narrow, 14- to 15-foot pavements tended to concentrate wheel loads in the center, often resulting in rutting and failure. Distribution was much better for widths of 18 feet or more.
- A painted centerline stripe or *guiding line* (at that time a novelty on highways) was very effective for keeping vehicles in their own lanes on curves.

This study was made at a time when passenger cars had a wheel gage of 58 inches and an overall width of 68 inches, and trucks had a gage of 65 inches and overall width of 88 inches. The observed speeds during the study were 10 to 12 mph for trucks and 20 to 25 mph for autos. Pauls concluded that 20 or even 18 feet of paving was adequate for safe clearance when overtaking other vehicles, but that 22 or 24 feet was excessive because the extra width was not fully utilized.

The Pauls method of studying transverse distribution of traffic was used in the Cleveland Regional Highway Planning Survey of 1927 with similar findings. One conclusion, supporting

Pauls', was that extra width above 20 feet adds little to the traffic capacity of a two-lane road. Another was that three-lane roads are unsatisfactory except in a few cases where there are pronounced peak periods of traffic in alternate directions at different periods of the day. Three-lane roads require lane marking and careful traffic control (*10*).

Motion Picture Placement Studies

The BPR lateral placement studies of 1924 and 1927 did not actually measure the behavior of drivers at the instant of passing each other on the highway, especially when they approached from opposite directions. These studies were made at fixed points on the highway and it would be only by coincidence that two vehicles would pass each other at the precise location of an observation station. In 1933 to obtain better measurements, Thompson of Johns Hopkins University and Hebden of the BPR mounted a motion picture camera on a car and then photographed events in motion by trailing other vehicles on the highways. Their most important findings were:

- When being overtaken the driver of a truck or other vehicle tried to center his own vehicle in his own lane, keeping his right wheel about 2 feet from the right edge of the pavement.
- The driver of the overtaking vehicle tended to swing wide to maintain the greatest possible clearance between his vehicle and the passed vehicle, even when this meant giving himself a lesser clearance from the left edge of the pavement. The average center clearance between vehicles varied from about 3.2 feet for 18-foot pavements to 4.9 feet for 22-foot pavements, and the average clearance of the overtaking vehicle's left wheel from the left edge varied from about 1.3 feet for 18-foot pavements to 3.6 feet for 22-foot pavements.
- When passing from opposite directions drivers kept their right wheels about 2 feet from the edge of the pavement maintaining a center clearance of about 4 feet for 18-foot roads and 6 feet for 22-foot roads.
- In a few instances passenger vehicles overtaking trucks on 18-foot roads ran with their left wheels on the shoulder. This did not happen with wider pavements.

Thompson and Hebden concluded that "Pavements of 18-foot width are too narrow for modern passenger cars alone or for modern mixed traffic. Pavements of 20-foot width are reasonably adequate for light-traffic roads used infrequently by wide trucks but are inadequate for heavy mixed traffic. Pavements of 22-foot width are entirely adequate for modern mixed traffic" (*11*).

–courtesy, California Division of Highways.

Road Speed Studies

Bureau of Public Roads and California Highway Commission observers made the first organized statewide speed studies in 1921 and 1922, by noting with a stopwatch the time it took for vehicles to traverse a measured distance. About 1930, Professor C. J. Tilden of Yale University improved on the California Technique with his invention of the Enoscope. This was an open-ended, L-shaped box with a mirror set at an angle of 45 degrees in the corner. The box was mounted on a tripod beside the road in such position that an observer 176 feet away could look into the open end and see across the road at that point. The observer noted the time on a stopwatch when a vehicle passed in front of him on the road and again when its image flashed in the mirror. If the time between these observations was 2 seconds, the speed was 88 feet per second or 60 mph; 3 seconds indicated 40 mph and so on. For some installations two Enoscopes were used with the observer stationed midway between them (*12*).

In 1933 the Maryland State Roads Commission and the University of Maryland observed traffic speeds with the Enoscope at 54 regular traffic census stations where the Commission had been counting traffic for many years. At most of these stations, hourly volumes averaged 200 vehicles or less, but a few were over 500 vehicles per hour. The aim of the study was to time approximately 500 cars in each direction at each station. In all, 52,704 observations were made, most of them at places where the posted legal speed limit was 40 mph. Of the total cars timed, less than two-tenths of 1 percent were traveling over 60 mph and 45 percent were within 5 mph of the legal limit. The average speed of all traffic in 40-mile speed zones was 37 mph, and 87 percent were traveling less than 45 mph (*13*).

At three stations the observations were made the same day as the county fairs. Curiously, the average speed of vehicles headed toward the fairs was 48 mph, but for those traveling in the opposite direction it was only 39 mph.

From the speed study the Commission concluded that 40 to 45 mph was a reasonable regulation and that its observance by the motoring public was good. No correlation could be found between average speed and pavement width—motorists seemed to drive on 16-, 17-, 18-, 20-, and 22-foot pavements at about the same speed (*13*).


PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

VOL. 19, NO. 3 MAY 1938

AUTOMATIC TRAFFIC RECORDER INSTALLED ON U S 240 IN MARYLAND

For sale by the Superintendent of Documents, Washington, D. C. See page 3 of cover for prices

About the same time as the Maryland speed studies the Michigan Highway Patrol was making a survey of road speeds in that State. (At this time there was no speed limit in Michigan.) To get a random sample the officers paced the first 10 vehicles they encountered each day for 2 weeks; if possible without the knowledge of the driver. The results fell into a pattern similar to Maryland's with 69 percent of the drivers traveling between 35 and 50 mph. Average speeds on paved highways were 43.3 mph in daylight and 41.5 mph at night. On gravel highways the corresponding speeds were 39.8 mph and 34.0 mph (*13*).

In the decade between 1930 and 1940 a number of ingenious methods were used to measure highway speeds. Johnson in Maryland photographed highways from an airplane and measured the change in position of the vehicles in the overlap between successive photos (*14*). Greenshields set up a motion picture camera about 300 feet to one side of the road, and measured the progress of particular vehicles from one photo frame to the next (*15*). Dana at Washington State College positioned two parallel light beams 24 inches apart so that they focused on photo-electric cells in a receiver on the other side of the highway. When the beams were cut by a vehicle, the time interval between the interruption of the two parallel beams was measured by an electrical circuit and converted to speed on a recording meter. This instrument could simultaneously count traffic, show in which direction each vehicle was traveling, and record its speed (*16*). Other devices depended on complex electronic circuits actuated by photo-electric cells, road tube pneumatic switches, or electrical sensors on the pavement (*12*).

These methods of speed measurement were cumbersome or imprecise or else they measured only *spot speeds,* that is the vehicle's velocity at a particular instant. More sophisticated equipment was needed before complex maneuvers involving acceleration and deceleration, such as overtaking, could be measured and analyzed.

The Overtaking Maneuver

The high speeds of motor cars—20 to 30 mph—introduced an element of danger into highway travel. By 1913 a consensus had evolved among highway engineers and administrators that there should be enough clear sight distance around all curves and over hill crests to permit a motorist to bring his machine safely to a stop from a speed of 20 to 25 mph. Inquiry of motor clubs throughout the United States disclosed that most of them thought this distance should be 200 to 300 feet; and in the period before World War I, it became general practice to provide this much sight distance by lengthening curves, or by flattening slopes and clearing away vegetation on the inside of curves (*17*).

By the 1920's a number of States by law or regulation prohibited motorists from overtaking and passing unless clear distances were available for the maneuver at the time it was undertaken. These clear distances varied from 150 feet in Arizona to 1,000 feet in Wisconsin (*18*). At this time there was no factual information available on the sight distances needed for safe passing.

In 1934, H. C. Dickinson of the National Bureau of Standards measured the time required for selected drivers to complete a passing maneuver on a nearly level highway during relatively light traffic. The drivers were instructed to follow behind the car to be passed at about the same speed, then accelerate past the other car and return to the right lane as rapidly as practicable, as if another car were approaching. An observer in the overtaking car timed the maneuver with a stopwatch. A considerable number of runs were made with different drivers and different cars and at speeds from 10 to 50 mph. The time to complete the maneuver always ranged between 5 and 7 seconds, and was independent of the traffic speed (*19*). From these tests Dickinson concluded that at least 900 feet of clear road ahead was required to safely pass a vehicle traveling 40 mph (*19*).

The same year, 1934, the Massachusetts Highway Accident Survey determined that if the passed car is traveling 40 mph and the overtaking car accelerates at 2 feet/sec^2, 818 feet will be required to complete the pass. However, this determination applied only to passings where there was no car coming from the opposite direction. If an opposing car were approaching at 40 mph, an additional 613 feet would be required to safely complete the pass (*19*).

CARS PASSING ON A NARROW ROAD

A third study of passing distances was made in 1934 by B. D. Greenshields of the Ohio State Highway Department, using a motion picture camera. He set up the camera at right angles to the highway and 300 feet away which gave a field wide enough that the photographed vehicles would appear in two or three successive frames. Knowing the time interval between exposures and the scale of the image, Greenshields could calculate the speed and acceleration of the vehicles and the interval between them.

For his passing study Greenshields observed 5,400 cars traveling on two-lane highways on which two-way traffic volume was 745 to 1,090 vehicles per hour. Since very few passings occurred directly in the camera's field, it was necessary to reconstruct the pass maneuvers by deductions made from the observed speed and spacings of the vehicles in the traffic stream. These deductions showed that at traffic speeds of 36 to 42 mph overtaking drivers used 9.5 to 13.2 seconds for the complete maneuver, with an average of 11.2 seconds, and that the distance required to accomplish the passing ranged from 1,050 feet to 1,500 feet with an average of 1,262 feet. As a general conclusion Greenshields recommended that passing should be prohibited where there is not at least 1,000 feet of clear sight distance (*19*).

In 1938 Matson and Forbes of the Yale Bureau of Street Traffic Research studied passing practices by placing themselves in the passed vehicle and observing the behavior of overtaking drivers with the aid of a camera, stopwatch, and the test vehicle's odometer and speedometer. The test car was driven at uniform speed in the traffic stream. As faster-traveling vehicles approaching from the rear turned out to pass, they were photographed from the rear window of the test car and again through the windshield after completing the pass. The distance separating the two vehicles at these instants could be calculated by stadia methods by scaling the tread dimensions of the overtaking vehicle in the photos. The 795 observations made were distributed geographically from New England to the West Coast (*18*).

The investigators found that there were two general types of passings:

- In *flying passes* the overtaking car made the entire maneuver without slowing down or accelerating.
- For *accelerative passes* the overtaking car slowed to about the same speed as the passed vehicle while the driver evaluated his opportunity to pass; then with the coast clear he accelerated and completed the pass.
- For both types of passes the time required for completion was affected by oncoming vehicles in the opposing lane. If there were none in sight or if they were far down the road, a leisurely or *voluntary return* was usual; but if the approaching vehicle was close, the overtaker would make a shorter or *forced return*.

Matson and Forbes assumed that the minimum safe time clearance between an overtaking vehicle and an approaching vehicle should be not less than 1 second; that is, the overtaker should have that much time to vacate the opposing lane and get back into his own. They found that 80 percent of drivers who attempted to pass in the face of oncoming traffic allowed themselves at least 1 second clearance; only 10 percent cut the clearance so short as to crowd the opposing driver to the outside of his lane or to the shoulder.

The time required for the passing maneuver varied from 8.0 to 10.5 seconds for 30-mph passes to 9.5 to 12.0 seconds for 50-mph passes, depending in each case on whether the return was forced or voluntary.

Differences in Individual Performance

The researchers were impressed by the large individual differences in performance between drivers. For example, some began accelerative passes with only 10 feet clearance between themselves and the vehicle ahead, while others had as much as 90

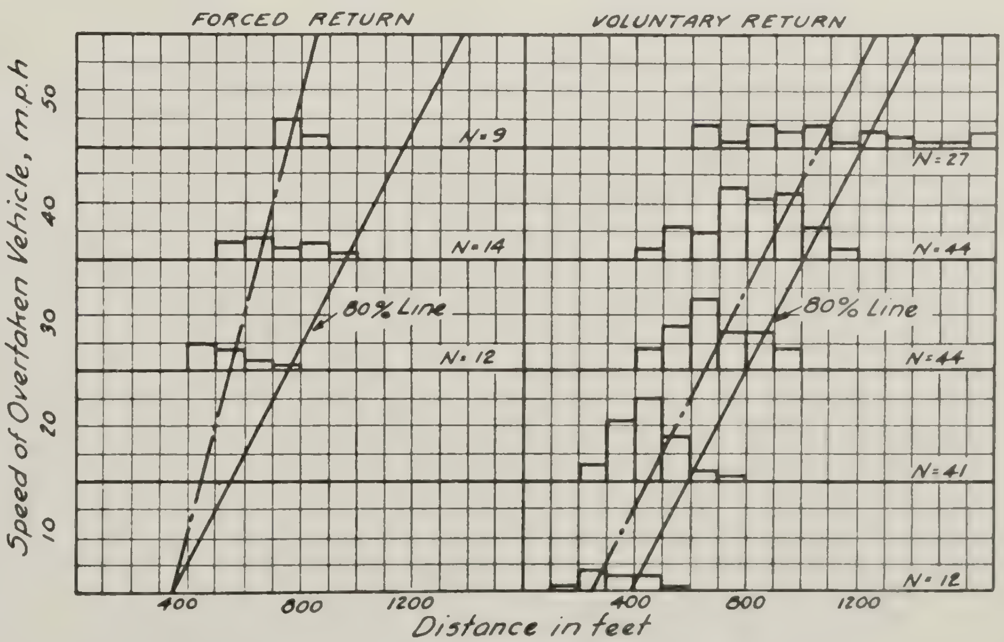

Figure 1.—Distance required to overtake and pass, flying type, all areas, all drivers (n =number of observations) (18).

feet clearance. There were similar large variations in the rear clearance after passing was completed, ranging from 50 feet up to 210 feet. Midwestern drivers were apparently more cautious and took more time to pass than eastern drivers. Women were more cautious than men.

The distribution of distances required for overtaking and passing is shown in figures 1 and 2. Median distances required to accomplish flying passes and accelerative passes appear in table 1.

Matson and Forbes recommended that the distance required for 80 percent of drivers to make the accelerative-voluntary return type of pass be adopted as the passing sight distance for design. They found this

Table 1.—Median values for overall distance required for overtaking and passing *(18)*[1]

Speed	Flying passes		Accelerative passes	
	Forced return	Voluntary return	Forced return	Voluntary return
Mph	*Feet*	*Feet*	*Feet*	*Feet*
30	550	650	450	600
50	750	1,051	750	1,000

[1] All drivers, all areas. Passenger vehicles only.

80 percentile distance to be 750 feet when the passed vehicle was traveling 30 mph; 950 feet for 40 mph; and 1,150 feet for 50 mph *(18)*.

The passing distances measured by Matson and Forbes were those required after the overtaking driver had made up his mind to pass and had begun the maneuver. They noticed, however, that for the accelerative or following type of pass the overtaking driver always spent a few seconds sizing up the situation ahead and satisfying himself that there was sufficient space available before pulling out of his lane and starting to overtake the car ahead. The distance traveled by the overtaking vehicle during this *perception time* was logically a part of the total passing distance.

In a later study Forbes undertook to measure this perception time, first by photographing traffic from an airplane with a moving picture camera, and later by stopwatch timing of the overtaking driver's reactions by a passenger-observer *(20)*. He found the time required by the 80 percentile of all observed drivers to be 3.5 seconds, regardless of the speed of the overtaken vehicle. For a speed of 50 mph this would add 258 feet to the total distance required to complete the pass, making the total distance about 1,300 feet.

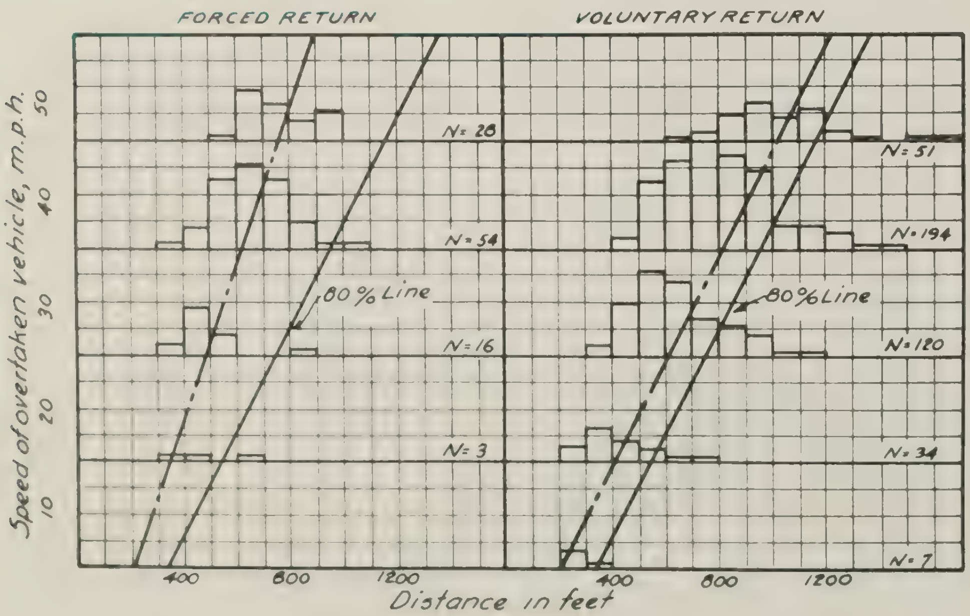

Figure 2.—Distance required to overtake and pass, accelerative type, all areas, all drivers (n = number of observations) (18).

The BPR–State Passing Studies

While Matson and Forbes were engaged in their passing studies of 1938 the Bureau of Public Roads was approaching the problem from still another angle. The BPR engineers hoped to measure and automatically record a large number of overtakings and passings on the highway, and then by analyzing the records determine how these maneuvers were actually made on the road. Essential to such a program was some means of knowing where all vehicles involved might be throughout the complicated maneuver. The BPR instrument laboratory met this need by developing an ingenious *road switch* actuated by a pulse of air from a rubber tube stretched across the road. Each pair of wheels passing over the rubber tube caused an air pressure impulse that moved a rubber diaphragm. The movement was only 0.025 inch, but this was sufficient to close an electrical contact and actuate a counting mechanism. Laboratory tests showed that the switch and counter would respond to impulses spaced 0.072 second apart—a rate which would enable the device to register both axles of a 114-inch-wheelbase vehicle traveling at 90 mph. In actual field trials the instrument counted axles with an error of less than 2 percent *(21)*.

In 1938 the Bureau of Public Roads observed and measured 1,635 passing maneuvers on Maryland and Virginia highways. For these observations the BPR engineers selected four straight stretches of road that were at least ½ mile long. For each test section they used 108 road switch detectors connected in pairs to rubber tubes stretched across the road at 50-foot intervals. A dead section at the center of each tube insured that the air pulse would travel in only one direction; thus each tube detected traffic separately in each direction of travel. All of the detectors were connected electrically to synchronized graphic pen recorders driven by clockwork at constant speed, producing a continuous record on paper charts. Since the detector tubes were uniformly spaced and the recording charts were driven at a uniform rate, the observers could determine the speeds of all vehicles involved in a passing maneuver by scaling the distance between successive pen marks and converting to velocity. An accuracy within 2 mph of true speed at a velocity of 60 mph was possible. The recording charts also showed the positions of all vehicles involved in the maneuver, including vehicles that might be approaching in the opposing lane *(22)*.

During 1939 and 1940 the BPR and the State highway departments used the new equipment for passing studies at 32 locations in Massachusetts, Illinois, Texas, California, and Oregon. In all, nearly 21,000 passing maneuvers were observed. These and the Maryland–Virginia studies showed that most of the overtaking drivers wanted to travel about 10 mph faster than the vehicles they were passing. More than half of all passings were *multiple maneuvers,* that is two or more vehicles were passed or did the passing. For accelerative-type passes most drivers made sure that they had at least 1,000 feet of clear road ahead before attempting to pass, and half of the drivers did not pass immediately upon perceiving that the road ahead was clear but waited a few seconds and did not fully utilize their passing opportunities *(23)*.

TRAFFIC DURING A STUDY OF VEHICLE PASSING PRACTICES (25)

Table 2.—Distances covered during the passing maneuver at various speeds *(24)*

Speed of passing vehicle . . . . .mph	30–39	40–49	50–59
Speed of passed vehicle . . . . .mph	20–29	30–39	40–49
Preliminary delay . . . . .feet	150	205	305
Occupation of left lane . . . . .feet	477	670	789
Interval for oncoming vehicles . . . .feet	602	756	900
Total . . . . .feet	1,229	1,631	1,994
Rounded distance . . . . .feet	1,200	1,600	2,000

In a later analysis of the BPR-State passing data, Prisk of the BPR found that although most passenger vehicles were easily capable of accelerating at a rate of 3 mph/second only half of the drivers accelerated as much as 1.3 mph/second *(24)*. He divided the typical passing maneuver into three *elements*:

- The *preliminary delay* during which the overtaking driver evaluates his opportunities to pass and accelerates to the point where his left wheel encroaches on the opposing lane. During this period the overtaking car may travel 150 feet for slow passes up to 305 feet for those made at 60 mph.
- The *occupation of the left lane,* which for the 80 percentile of drivers might range from 477 feet for a slow pass to 789 feet for a high-speed pass.
- The *interval for the oncoming vehicle,* during which the passing driver returns to his own lane, hopefully with a little distance to spare. This interval ranged from 602 feet for slow passes up to 900 feet for fast ones.

By combining the distances required for the three elements Prisk came up with safe distances for the passing maneuver (table 2).

Most of the observations for the 1938–40 passing studies were made at selected tangent locations where the grade was nearly level. However, one observation course in Illinois was on a 1,700-foot, 6-percent grade—a grade steep and long enough to make overloaded or underpowered trucks reduce speed to a 5-mph crawl. Lighter and faster vehicles that were easily able to surmount the grade at 25 to 35 mph had to queue up behind the slow vehicles in some cases for as long as 2 minutes before they could pass. This test was a convincing demonstration of the adverse effect of steep grades on highway capacity *(25)*.

Improved Equipment Developed by the BPR

We have seen that with the equipment developed by the BPR for passing studies it was possible to determine speeds indirectly by measuring the distance between pen marks on the recording tape and then converting the distance to velocity. This gave accurate results but was cumbersome and time-consuming. Needed was a means of measuring and automatically recording the speeds of individual vehicles in less than 1 second. The BPR instrument laboratory met this need by developing the *Speedmeter* in 1939. This device consisted of two pneumatic detector tubes spaced a known distance apart on the pavement (usually 24 feet), a timing unit, and an automatic pen recorder. For the timing unit the laboratory adapted the Strowger rotary switch, an electromechanical device borrowed from radio-telephony. The Speedmeter was a rugged device that could measure vehicle speeds with an accuracy of less than 1 mph (26).

While the BPR engineers were developing the speedmeter they were also working on a *placement detector* that would measure the transverse position of vehicles on the pavement. The resulting device consisted of two spring steel strips 24 feet long, separated from each other by a rubber arch. The bottom strip was solid, but the upper one was divided into segments each 1 foot long, and insulated from the others. When a vehicle wheel passed over one of the upper segments, it was pressed into contact with the bottom strip, completing an electric circuit to an automatic pen recorder (26).

The speedmeter and placement detector, teamed with the portable traffic recorder (another spin-off of the pneumatic road switch research), made a compact and efficient unit for research in traffic behavior. Trial runs proved that better data could be gathered with the new equipment for less than one third the field labor cost of the older methods and less than one fourth the previous cost for office analysis.

A number of units were assembled by the BPR and mounted in special vans. Two of these were placed in service the summer of 1939 and others in 1940 for use in cooperative studies of driver behavior. Before World War II ended the work, the Bureau of Public Roads and 10 State highway departments completed studies of driver behavior involving 95,589 vehicles at 47 locations. The 47 test sites were established at places where there was more than the minimum sight distance for a speed of 60 mph, where the shoulders were good, and there were no obstructions near the edge of the road. The test pavements were of concrete, 18, 20, 22 or 24 feet wide. At 28 of the locations studies were made during the day and also at night *(27)*.

The findings of these and previous studies have had an important influence on design policy in the United States. They showed that:

- Shoulder width in excess of 4 feet does not influence effective pavement width for moving vehicles when there are no vertical obstructions immediately adjacent to the shoulders. (Shoulders may however be wider than 4 feet for other reasons, such as safety.)
- Well-maintained grass shoulders have the same effect on transverse vehicle placement as well-maintained gravel shoulders.
- Bituminous treated shoulders 4 feet or more wide adjacent to 18-foot and 20-foot pavements increase the effective surface width approximately 2 feet.
- Lip curbs on 20-foot pavements reduce effective pavement width about 1 foot.
- Shoulder use increases rapidly as pavement width decreases below 22 feet. At 22 feet practically no traffic uses unpaved shoulders, but at 18 feet 5 percent of trucks do so when meeting oncoming traffic. If the shoulders are bituminous-paved 17 percent of trucks take to the shoulder when passing.
- With moderate volumes of mixed traffic on 18-foot pavements 11 percent of truck drivers and 5 percent of car drivers fail to keep their vehicles in their own proper lane, even when meeting opposing traffic.
- Drivers drive as fast on narrow pavements as on wide ones.
- When a passenger car meets a truck on a 22-foot pavement, the passenger car has the desired center and edge clearances, but the truck does not. Desired clearance for trucks requires a 24-foot width.

COLLECTING DATA ON SPEED AND PLACEMENT

VEHICLE POSITIONS ON A TYPICAL 22-FOOT HIGHWAY. (27)

One of the 1939 studies in Ohio was planned to determine the effect of fixed lighting on driver behavior. The test section was 1 mile long with a concrete pavement 20 feet wide and 10-foot shoulders, and was illuminated by overhead incandescent reflectors mounted 25 feet above the pavement and 125 feet apart. This study showed that the behavior of drivers at night under artificial overhead light was very similar to their behavior in daylight, both as to lateral placement on the pavement and as to the utilization of passing opportunities. But when the overhead lights were turned off and the drivers had to depend only on their own headlights, they tended to crowd closer to the center of the road by an average of 0.5 foot, and utilized only 38.5 percent of their passing opportunities, as compared to 55.6 percent under overhead lighting (*28*).

Bridge Deck Clearances Measured

In other significant lateral placement studies of this period, the observers noted that bridge curbs and parapets reduced the effective width of the bridge deck available for traffic. The drivers tended to shy away from these obstructions in order to preserve a clearance that was never less than 6.2 feet between the right wheel and the curb. The ideal situation was one where the driver did not change position toward the center of the road as he crossed the bridge, indicating adequate right wheel clearance. For this situation the observers determined that the following minimum bridge widths should be provided (*29*):

Width of approach pavement	Width of approach roadway	Width of deck between curbs
18 ft	24 ft	26 to 28 ft
20 or 22 ft	34 ft	28 to 30 ft

In yet another study, Green and Quimby of the Purdue University Joint Highway Research Project used an Army gunsight camera to photograph vehicles approaching narrow bridges. The camera was hidden from the view of the driver either in the overhead truss-work or behind a sign, and was aimed parallel to the curb. Scale was obtained by photographing a series of calibrated reference targets on the deck during an interval of no traffic (*30*). At this time (1945–47) there were a number of dangerous bridges on the Indiana State highways that were narrower than the approach pavements. These bridges had originally been built 19 feet wide for 18-foot approach pavements, and the pavements had later been widened to 22 feet without widening the through-truss structures. The widened road did not have a painted centerline.

At one particularly dangerous location the researchers tried various combinations of advance warning signs and diagonal striped panels, reflectorized and unreflectorized, to induce drivers to slow down and stay in their own lanes. Vehicle speeds were recorded with a Photo–Velaxometer, an accurate speed-measuring device developed at Purdue University. This study demonstrated that the centerline stripe was very effective for controlling vehicle placement, day or night. Reflectorized signs tended to cause drivers to move toward the center of the road at night, but had no visible effect in daylight. Apparently, the bridge itself was a big enough target to monopolize the driver's attention in the daytime (*31*).

In another study of driver behavior the Public Roads Administration (the former BPR) and seven State highway departments proved by speed and transverse position measurements what was already well-known in a general way—that a centerline stripe is a very efficient method for controlling driver behavior. The researchers selected 12 level and tangent locations where the pavement widths varied from 18 to 24 feet. For some of these, white or black centerlines were painted on the pavements, while others, comparable otherwise, had none. On these test sections measurements of speed and transverse position were made on 18,225 vehicles in 1940, 1941, and 1947. Traffic volumes ranged from 70 to 659 vehicles per hour.

Not unexpectedly, the observers found that a much higher proportion of vehicles traveled in their own lanes where the pavements were striped, the average placement being 0.4 to 1.4 feet farther to the right. At the same time, the tendency to ride the shoulder where pavements were narrow was not increased. Clearance between vehicles passing from opposite directions on roads with marked centerlines was 0.6 to 2.2 feet greater than on unmarked roads. Even on relatively wide roads with unmarked pavements 22 to 24 feet wide, 20 to 57 percent of passings were made with less than 3.0 feet clearance (considered the minimum for safety). By contrast, on centerlined roads less than 10 percent of the passes were made at less than 3 feet clearance. A somewhat unexpected finding was that the average speed of vehicles for all widths of pavement was about 4 mph faster on centerlined pavements than on unmarked pavements *(32)*.

Trip Speeds Measured

Before leaving the subject of driver behavior it would be desirable to take a look at studies of average road speeds of large groups of vehicles (as contrasted to spot speeds of individual vehicles). Without doubt, measurement of trip speeds by vehicles traveling in the traffic stream goes back to the earliest days when traffic congestion began to be noticed on streets and highways—probably in the period just before World War I. However, the first systematic use of trip speeds probably dates to the Cook County Transportation Survey of 1924. Trip speed studies were an important feature of the Cleveland Regional Highway Planning Survey of 1927.

In November 1932 the New Jersey State Highway Commission opened a 3.7-mile elevated highway to give high-speed access to the Holland Tunnel from the West. This viaduct, which cost $5,200,000 per mile was 0.7 mile shorter than the previously used surface route through Newark and Jersey City and eliminated two drawbridges. The Bureau of Public Roads and the Commission wanted to know accurately how much motorist time was saved by the new viaduct. They provided for a cooperative study to be made before and after the opening of the viaduct, using otherwise unemployed persons for observers. This was the first attempt to time large numbers of vehicles through a measured course *(33)*.

For the *before* condition it was necessary to establish observer stations at principal cross streets, but for the viaduct, observers were placed only at the ends. The observers were equipped with carefully synchronized watches. At each station they recorded the license number, the type of vehicle (passenger car, truck, or bus), and the time of passage. For the heavy traffic stations two passenger car observers and two truck and bus observers were needed.

The observers developed surprising speed in their nerve-racking jobs, consistently recording 20 vehicles per minute and sometimes exceeding 30. Even so, many vehicles were missed, which is not surprising considering that the average daily traffic on the old route was 37,400 vehicles on weekdays and 49,300 on Sundays. For a typical 18-hour count at the 8 stations, 99,000 license numbers were recorded out of a total of 128,000 vehicles passing these stations. Of the 99,000 recorded license numbers, 37,000 were matched, giving the trip time for 18,500 vehicles. On an average, 77 percent of passing vehicles were listed and 29 percent were timed over the route.

The same method was used for timing vehicles over the viaduct after it was opened to traffic; but since the number of vehicles was less, the success of recording and matching license numbers was higher (87 percent recorded and 49 percent matched.). After an initial adjustment period, viaduct traffic averaged 20,800 vehicles per day on weekdays and 38,700 on Sundays, which was fairly heavy for an undivided four-lane deck.

Analysis of the matched samples showed that trip time for the ground-level route varied from 9.1 minutes during the hours of lightest traffic (150 vehicles per hour) to 13.3 minutes during the 4:00 to 6:00 p.m. rush (1,400 vehicles per hour), provided there was no delay for an open drawbridge during the trip. (Such delay might add 10 minutes or more.) Trip time via the viaduct, by comparison, was only 5.5 minutes during light traffic, or 6.2 minutes during heavy traffic periods. The investigators estimated the total saving of vehicle-user time in the viaduct's first year of operation at 66,100,000 vehicle-minutes which at 2 cents per minute was estimated to be worth $1,320,000.

The Bureau of Public Roads subsequently developed automatic equipment, previously described, which greatly reduced the labor and cost of average speed studies.

By 1939 the driver behavior studies described above had produced sufficient fruit that the Bureau of Public Roads and the State highway departments were able to mount an assault on the principal problem still facing highway designers—how to measure the vehicle carrying capacity of roads. In part 4 of this series we will follow the research that eventually culminated in the first highway capacity manual.

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

The following are brief descriptions of selected items which have been recently completed by State and Federal highway units in cooperation with the Implementation Division, Offices of Research and Development, Federal Highway Administration (FHWA). Some items by others are included when they have a special interest to highway agencies. These items will be available from the Implementation Division unless otherwise indicated. Those placed in the National Technical Information Service (NTIS) will be announced in this department after an NTIS accession number is assigned.

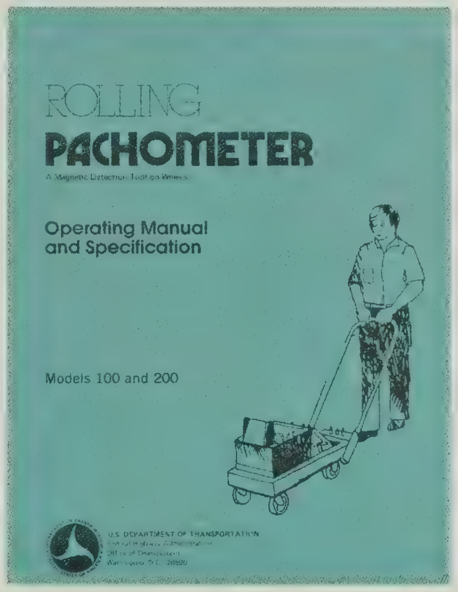
U.S. Department of Transportation
Federal Highway Administration
Office of Development
Implementation Division, HDV-20
Washington, D.C. 20590

Rolling Pachometer

by FHWA Implementation and Engineering Services Divisions

The Rolling Pachometer is a dynamic bridge inspection instrument which incorporates a modified version of the commercially available hand portable pachometer. It is designed to rapidly monitor and record the depth of concrete cover over the reinforcing steel in bridge decks. It graphically records bar depth automatically while traveling at 88 feet per minute. With adequate calibration data suitably adjusted, the Rolling Pachometer has demonstrated the capability of identifying rebar cover depth within a tolerance of ±¼ inch.

A brochure, operating manual, and specifications for the Rolling Pachometer are available from the Implementation Division.

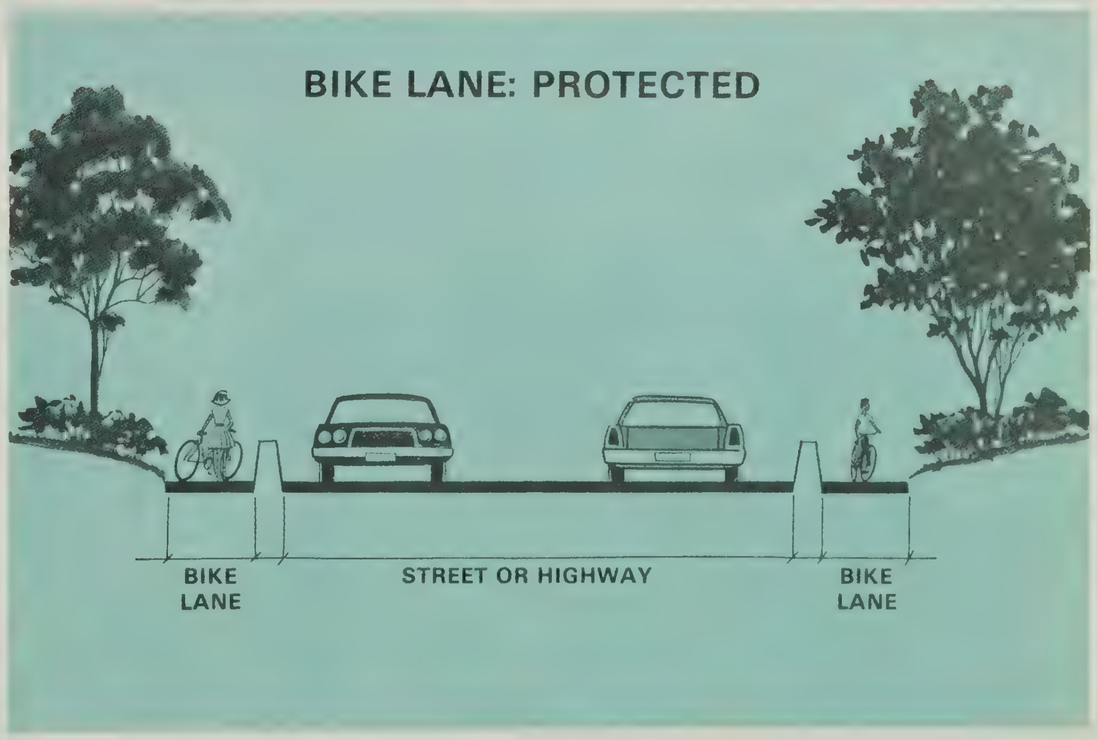


Bikeways—Let's Get Serious

by Maryland Department of Transportation

The Maryland Department of Transportation, Division of Highways, has developed a movie that explains their bicycle program and describes the steps they took in program development, planning, and implementation. The movie also shows the various classes of bikeways and how they may be used in the development of a bikeway system. The information presented would be of value to jurisdictions wanting to develop their own bicycle program.

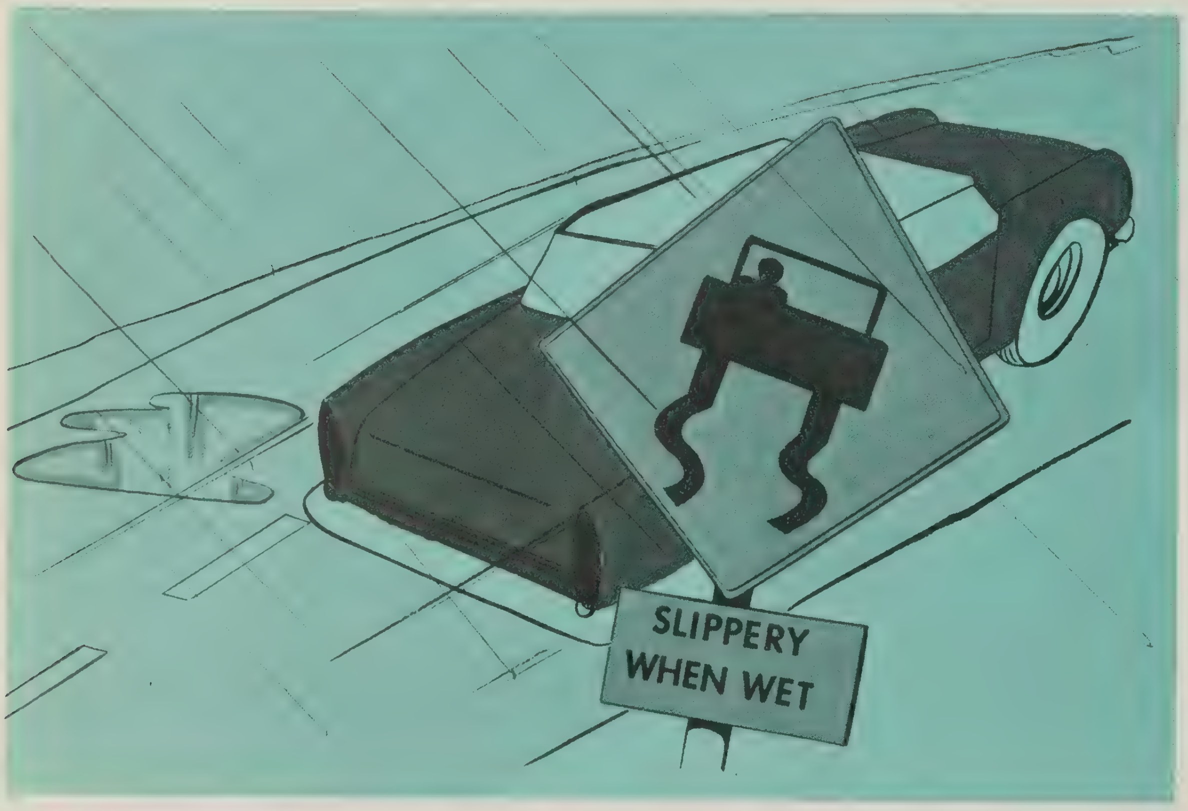
Information on availability of the movie may be obtained from the Implementation Division.



Texturing of New Concrete Pavements

by Georgia Department of Transportation

The Georgia Department of Transportation is using transverse plastic grooving for portland cement concrete pavements. The plastic grooving is accomplished by using a steel tine comb attached to a concrete finishing machine. The resulting surface is such an improvement over that obtained by other conventional finishing methods (burlap drag, nylon broom, etc.) that Georgia currently specifies grooving of plastic concrete on the mainline by use of special provisions on a project-to-project basis.



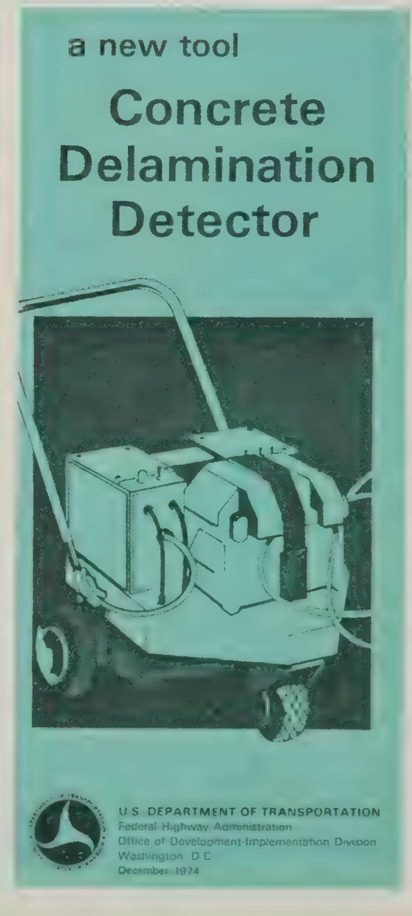
A package containing a slide presentation with narrative tapes and copies of the Georgia Special Provisions is available on a loan basis from the Implementation Division.

Concrete Delamination Detector

by Texas Highway Department and Texas Transportation Institute

The concrete delamination detector is a mechanical-electronic device that sonically identifies sub-surface fracture areas of concrete bridge decks by selectively filtering the frequency response of the deteriorated concrete. Decks covered with 2 or more inches of bituminous concrete are no problem to the concrete delamination detector in locating delamination in the underlying concrete. It also responds to unbonded overlay, but cannot distinguish the difference between unbonded overlay and concrete delamination. The two signal-receiving hydrophones are positioned 12 inches apart and, thus, can straddle and not record delaminated areas less than 12 inches in diameter or width. The detector will identify any unsound area that both the tapper wheel and hydrophone pass over.

A brochure and tape-narrated color slides are available from the Implementation Division.

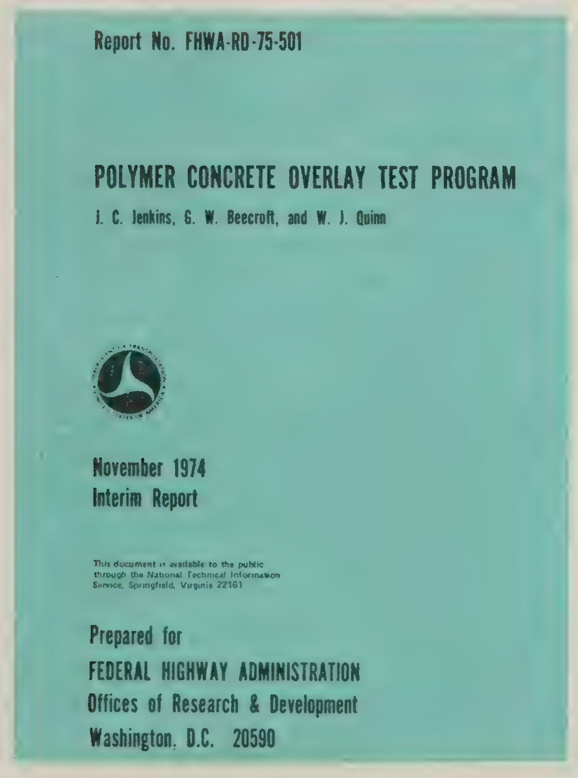


Polymer Concrete Overlay Test Program

by Oregon State Highway Division

This is an interim report outlining the work done by the Oregon State Highway Division in developing polymer concrete mixes suitable for overlaying portland cement concrete. Oregon's primary interest in the polymer concrete overlay is as a fully bonded structural element for deck strengthening, and secondarily as an impermeable element for deicer protection. Various combinations of polymer concrete mixes are presented with test results showing their physical properties. Also presented is a polymer concrete formulation which research findings indicate would be suitable for bridge deck or pavement patching. Methods for handling the chemicals used are recommended.

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 240624).


Report No. FHWA-RD-75-501

POLYMER CONCRETE OVERLAY TEST PROGRAM

J. C. Jenkins, G. W. Beecroft, and W. J. Quinn

November 1974
Interim Report

This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161

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

Keyed Riprap

by Oregon State Highway Division

Highway engineers traditionally have used heavy rock riprap to protect bridges and highway fills from erosion by water. The riprap is usually placed by end dumping with the results, quite often, being a loose mass of segregated rock. Projecting rocks are sometimes displaced by drag forces or floating drift during periods of high flow. The engineers' answer in the past has been to use larger rocks in greater quantities.

A new rock riprap placement technique known as *keyed* or *plated* riprap is now being used on highway projects in Oregon. With this technique a dense mass of interlocked stone with a smooth, uniform surface is produced. Keying is accomplished by pounding the surface with a heavy steel plate. The plating breaks high points off the larger pieces of stone and pushes others into the material beneath.

Keyed riprap is more attractive than conventional riprap and, in Oregon's experience, costs about the same because less material is used. It is also considered more effective in preventing erosion than conventional riprap and maintenance costs have been low.

A movie entitled "Keyed Riprap" has been sent to each FHWA regional office and is available from the regions on a loan basis (see page 87).


Before


After


After

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 Research)—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 1F: Energy Absorbing and Frangible Structures

Title: Video Tape Monitoring of Impact Energy Attenuators—Phase II. (FCP No. 41F2224)
Objective: Expand upon the findings of D–4–108 by using improved video tape equipment for monitoring attenuators. The study is aimed at devising means of decreasing frequency of attenuator impacts.
Performing Organization: California Department of Transportation, Sacramento, Calif. 95814
Expected Completion Date: May 1979
Estimated Cost: $130,000 (HP&R)

FCP Project 1H: Skid Accident Reduction

Title: Wet-Surface Accidents and Their Relation to Pavement Friction. (FCP No. 41H5232)
Objective: Determine critical friction values (SN and PSN), critical wet-surface accident rates, and ratios of wet to dry accidents for various highway types and conditions and then determine accident reductions resulting from improved pavement friction.
Performing Organization: Kentucky Department of Highways, Lexington, Ky. 40508
Expected Completion Date: May 1980
Estimated Cost: $96,000 (HP&R)

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

Title: Nontoxic Yellow Traffic Striping. (FCP No. 31I2162)
Objective: Find an acceptable nontoxic yellow pigment as an alternate to lead chromate for use in yellow highway lane striping.
Performing Organization: National Bureau of Standards, Washington, D.C. 20234
Expected Completion Date: May 1977
Estimated Cost: $90,000 (FHWA Administrative Contract)

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

FCP Project 2B: Development and Testing of Advanced Control Strategies in the Urban Traffic Control System

Title: Traffic Responsive Ramp Control Through Use of Micro Computers. (FCP No. 42B5013)
Objective: Evaluate the use of micro computers as traffic responsive ramp controllers. Develop hardware and software, considering cost and functional capability.
Performing Organization: California Department of Transportation, Sacramento, Calif. 95807
Expected Completion Date: December 1976
Estimated Cost: $122,000 (HP&R)

FCP Project 2C: Requirements for Alternate Routing to Distribute Traffic Between and Around Cities

Title: Development of Traffic Logic for Optimizing Traffic Flow in an Intercity Corridor. (FCP No. 32C1022).
Objective: Develop a traffic control algorithm which will select—both off-line and real time—values of control variables which will optimize traffic flow in a general intercity corridor.
Performing Organization: Sperry Systems Management Division, Great Neck, N.Y. 11020
Expected Completion Date: March 1977
Estimated Cost: $660,000 (FHWA Administrative Contract)

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

Title: Evaluation of Carpool and Bus Traffic Operational Incentives. (FCP No. 32D1524)
Objective: Provide technical assistance to several State and local agencies which are conducting evaluations of various operational incentive projects, conduct an overall evaluation of the projects, and develop a handbook for use by other agencies contemplating similar projects.
Performing Organization: JHK and Associates, Alexandria, Va. 22304
Expected Completion Date: January 1977
Estimated Cost: $92,000 (FHWA Administrative Contract)

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

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

Title: Effects and Evaluation of Water Quality Resulting from Highway Development and Operation. (FCP No. 33E2012)
Objective: Identify and quantify the constituents of highway runoff—physical, chemical, and biological characteristics—and identify the environmental impacts. Develop a predictive procedure for determining effects on water quality for use by highway departments.
Performing Organization: Envirex, Inc., Milwaukee, Wis. 53201
Expected Completion Date: March 1978
Estimated Cost: $496,000 (FHWA Administrative Contract)

FCP Project 3F: Pollution Reduction and Visual Enhancement

Title: Highway Location, Design, and Operation to Reduce Photochemical Smog Formation. (FCP No. 33F3092)
Objective: Evaluations of highway operations, locations, and designs as factors contributing to photochemical smog and formulation of methods to reduce effects of highway sources.
Performing Organization: Systems Applications, Inc., San Rafael, Calif. 94903
Expected Completion Date: June 1976
Estimated Cost: $172,000 (FHWA Administrative Contract)

FCP Category 4—Improved Materials Utilization and Durability

FCP Project 4G: Substitute and Improved Materials to Reduce Effects of Energy Problems in Highway Costs

Title: Evaluation of SA–1 Process for Reduction of Hydrocarbon Usage in Bituminous Pavement Construction. (FCP No. 24G1034)
Objective: Evaluate the potential of the SA–1 process to effect a reduction in asphalt cement requirements and to effect a reduction in required mixing temperature for bituminous paving mixtures.
Performing Organization: Federal Highway Administration, Washington, D.C. 20590
Expected Completion Date: June 1978
Estimated Cost: $130,000 (FHWA Staff Research)

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

FCP Project 5B: Tunneling Technology for Future Highways

Title: Evaluation of Aerial Remote Sensing Techniques for Defining Critical Geological Features Pertinent to Tunnel Location and Design. (FCP No. 35B2022)
Objective: Determine extent to which aerial remote sensing systems can be used to develop a three-dimensional geologic model to aid in tunnel location and design in mountainous areas.
Performing Organization: Earth Satellite Corporation, Washington, D.C. 20006
Expected Completion Date: August 1976
Estimated Cost: $237,000 (FHWA Administrative Contract)

Title: A New Sensing System for Pre-Excavation Subsurface Investigation for Tunnels in Rock Masses. (FCP No. 35B2152)
Objective: Develop the preliminary design of a complete optimized sensing system in rock masses using previously prepared long horizontal bore holes. The system will determine discontinuities (gouge zones, cavities) within 100 ft of the bore hole axis, joint patterns in the rock, the mechanical properties of the rock, and the location of changes in material.
Performing Organization: Ensco, Inc., Springfield, Va. 22151
Expected Completion Date: August 1976
Estimated Cost: $254,000 (FHWA Administrative Contract)

Title: Effects of Environmental Transitions on Highway Tunnel Operations and Safety. (FCP No. 35B4022)
Objective: Determine those aspects of tunnel transition zones which influence motorists performance and comfort; and develop design requirements which will maximize (1) the efficiency of tunnel operations, (2) safety, and (3) utilization of tunnels.
Performing Organization: Franklin Institute Research Labs, Philadelphia, Pa. 19103
Expected Completion Date: February 1977
Estimated Cost: $481,000 (FHWA Administrative Contract)

FCP Project 5C: New Methodology for Flexible Pavement Design

Title: A Study of Flexible Pavement Base Courses and Overlay Designs: Second Cycle of Research at the Pennsylvania Transportation Research Facility. (FCP No. 45C3252)
Objective: (1) Traffic existing sections to an unserviceable state and continue analysis; (2) examine effect of base course thickness on structural coefficients and pavement performance; (3) evaluate and refine VESYS II; and (4) validate the overlay design procedure now used in Pennsylvania.
Performing Organization: Pennsylvania State University, University Park, Pa. 16802
Funding Agency: Pennsylvania Department of Transportation
Expected Completion Date: June 1977
Estimated Cost: $543,000 (HP&R)

Title: A Statewide Study of Subgrade Soil Support Conditions. (FCP No. 45C3253)
Objective: Develop a statewide correlation between the most common soil series characteristics and modulus of resilience. Develop a correlation among soil series characteristics, AASHTO engineering soil classification, and rutting performance parameters and predict the rutting susceptibility of various soils. Determine the effect of environmental factors by using moisture shift function.
Performing Organization: Ohio State University, Columbus, Ohio 43215
Funding Agency: Ohio Department of Highways
Expected Completion Date: June 1978
Estimated Cost: $223,000 (HP&R)

FCP Project 5D: Structural Rehabilitation of Pavement Systems

Title: Second Generation Overlays. (FCP No. 45D2324)
Objective: Solve the following problems of second generation overlays of portland cement and bituminous pavements: (1) loss of clearance overhead; (2) introduction of undesirable cross slopes and loss of curb height; (3) correction of rutting; (4) raising of manholes; and (5) minimizing reflection cracking.
Performing Organization: New Jersey Department of Transportation, Trenton, N.J. 08625
Expected Completion Date: June 1977
Estimated Cost: $104,000 (HP&R)

Title: Development of a Rational Overlay Design Method for Pavements. (FCP No. 45D2332)
Objective: (1) An intensive analysis of test data to correlate deflection data with theory; (2) a literature review on overlay methods for concrete pavements; (3) development of design charts; (4) a review of load data; (5) performance surveys of asphalt overlays on PCC pavements; (6) field testing before and after overlay; and (7) modification of design charts.
Performing Organization: Kentucky Department of Transportation, Lexington, Ky. 40508
Expected Completion Date: June 1978
Estimated Cost: $131,000 (HP&R)

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

Title: Design and Tests of Precast Bridge Elements. (FCP No. 45F3092)
Objective: Gather and evaluate information from other States on their current use of precast segmental bridge deck elements. Design and test a new type of element for bridge decking.
Performing Organization: Louisiana State University, Baton Rouge, La. 70803
Funding Agency: Louisiana Department of Highways
Expected Completion Date: July 1976
Estimated Cost: $94,000 (HP&R)

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

Title: Bicycle-Safe Grate Inlets Study. (FCP No. 35H2032)
Objective: Develop new designs and improve certain existing designs of grate inlets on paved roads which maximize bicycle and pedestrian safety and hydraulic efficiency.
Performing Organization: Bureau of Reclamation, Denver, Colo. 80225
Expected Completion Date: June 1976
Estimated Cost: $175,000 (FHWA Administrative Contract)

Title: Assessment of National Small Rural Watershed Program (FCP No. 35H3092)
Objective: A comprehensive independent assessment will be made of the national small rural watershed program in keeping with long range goals of the Federal Highway Administration and State highway agencies.
Performing Organization: Meta Systems, Inc., Cambridge, Mass. 02138
Expected Completion Date: May 1976
Estimated Cost: $110,000 (FHWA Administrative Contract)

Non-FCP Category 0—Other New Studies

Title: Bearing Capacity and Characterization of Compressible Clayey and Silty Foundation Soils. (FCP No. 40M1522)
Objective: Determine character of compressible soils, develop graph to enable prediction of magnitude of settlement, and develop procedures for investigation of foundation soils.
Performing Organization: Oklahoma Department of Highways, Oklahoma City, Okla. 73105
Expected Completion Date: June 1978
Estimated Cost: $213,000 (HP&R)

Highway Research and Development Reports Available from the National Technical Information Service

The following highway research and development reports are for sale by the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161.

Other highway research and development reports available from the National Technical Information Service will be announced in future issues.

STRUCTURES

Stock No.	
PB 239537	The effect of Densification on the Engineering Characteristics of Organic Soils.
PB 239632	An Investigation of the Parameters Influencing Bond Behavior with a View Towards Establishing Design Criteria.
PB 239782	Automobile Tire Hydroplaning—A Study of Wheel Spin-Down and Other Variables.
PB 239839	Development of Procedures for Characterization of Untreated Granular Base Course and Asphalt-Treated Base Course Materials.
PB 239932	Anchorage Devices for Large Diameter Reinforcing Bars.
PB 239935	Field Testing a Reinforced Concrete Highway Bridge.
PB 240260	A Laboratory Evaluation of Full Size Elastomeric Bridge Bearing Pads.
PB 240261	Fatigue Data Bank and Data Analysis Investigation.
PB 240262	Influence of Water Depths of Friction Properties of Various Pavement Types.
PB 240269	Tire-Pavement Friction as a Function of Vehicle Maneuvers.
PB 240277	High-Strength Steel as Reinforcement for Concrete Bridges—Final Report.
PB 240308	Embankments With and Without Moisture Density Control.
PB 240328	The Behavior of an Axially Loaded Drilled Shaft Under Sustained Loading.
PB 240367	Fatigue Behavior of Welded Reinforcement in Reinforced Concrete Beams.
PB 240447	Deflection Behavior of Asphaltic Concrete Pavements.
PB 240625	Threshold Crack Growth in A36 Steel.
PB 240698	An Investigation of the Effectiveness of Existing Bridge Design Methodology in Providing Adequate Structural Resistance to Seismic Disturbances. Phase III: Analytical Investigations of Seismic Response of Short, Single, or Multiple-Span Highway Bridges.
PB 241241	Rainfall and Visibility—The View from Behind the Wheel.
PB 241291	Monitoring Field Installations of Impact Energy Attenuators by Videotape.
PB 241420	Performance Study of Continuously Reinforced Concrete Pavement on I–95.
PB 241506	Effectiveness of Attached Bridge Widenings.
PB 241512	Mechanical Strain Recorder on a Connecticut Bridge.
PB 241521	Highway Pavement Maintenance Costs and Pavement Type Selection.
PB 241634	The Inelastic Analysis of Reinforced Concrete Slabs.

Federal Highway Administration Regional Offices:

No. 1. 4 Normanskill Blvd., Delmar, N.Y. 12054.
Connecticut, Maine, Massachusetts, New Hampshire, New Jersey, New York, Puerto Rico, Rhode Island, Vermont, Virgin Islands.

No. 3. 1633 Federal Bldg., 31 Hopkins Plaza, Baltimore, Md. 21201.
Delaware, District of Columbia, Maryland, Pennsylvania, Virginia, West Virginia.

No. 4. Suite 200, 1720 Peachtree Rd., NW., Atlanta, Ga. 30309.
Alabama, Florida, Georgia, Kentucky, Mississippi, North Carolina, South Carolina, Tennessee.

No. 5. 18209 Dixie Highway, Homewood, Ill. 60430.
Illinois, Indiana, Michigan, Minnesota, Ohio, Wisconsin.

No. 6. 819 Taylor St., Fort Worth, Tex. 76102.
Arkansas, Louisiana, New Mexico, Oklahoma, Texas.

No. 7. P.O. Box 19715, Kansas City, Mo. 64131.
Iowa, Kansas, Missouri, Nebraska.

No. 8. P.O. Box 25246, Bldg. 40, Denver Federal Center, Denver, Colo. 80225
Colorado, Montana, North Dakota, South Dakota, Utah, Wyoming.

No. 9. 450 Golden Gate Ave., Box 36096, San Francisco, Calif. 94102.
Arizona, California, Hawaii, Nevada.

No. 10. Room 412, Mohawk Bldg., 222 SW. Morrison St., Portland, Oreg. 97204.
Alaska, Idaho, Oregon, Washington.

No. 15. 1000 North Glebe Rd., Arlington, Va. 22201.
Eastern Federal Highway Projects.

No. 19. Drawer J, Balboa Heights, Canal Zone.
Canal Zone, Colombia, Costa Rica, Panama.

PB 241790 Water Drainage from Highway Fills.
PB 241794 An Integrated Pavement Design Processor.

MATERIALS

Stock No.
PB 238157 Use of the Freezing Soil Heave Stress System to Evaluate the Frost Susceptibility of Soils.
PB 239043 D-Cracking in PCC Pavements—Cause and Prevention.
PB 239633 Instrumental Colorimetry of Retroreflective Sign Material.
PB 241077 Mixture Design Concepts, Laboratory Tests, and Construction Guides for Open-Graded Bituminous Overlays.
PB 241226 Polymer Pavement Concrete for Arizona Study I.
PB 241255 Behavior of Shrinkage-Compensating Concretes Suitable for Use in Bridge Decks—Interim Report, Phase 2.
PB 241294 Corrosion Testing of Bridge Decks.
PB 241440 Field Evaluation of a Direct Transmission Type Nuclear Moisture-Density Gauge.

TRAFFIC

Stock No.
PB 239916 Right-Turn-On-Red: Current Practices and State of the Art.
PB 240246 Study and Development of Highway Advisory Information Radio.
PB 240408 A State-of-the-Art Literature Review on Statewide Traffic Models—Interim Report.
PB 241578 Study of Local Ramp Control at Culebra Entrance Ramp on the Southbound IH–10 Freeway in San Antonio.
PB 241582 A Report on the User's Manual for Progression Analysis and Signal System Evaluation Routine—PASSER II.
PB 241606 Actuator Program for Frontage Road Control.
PB 241612 Frontage Control Strategy for the North Central Expressway.
PB 241613 Freeway Operations Manual—Dallas Corridor Study.
PB 241630 Data Collection and Analysis for the Eastshore Freeway (I–80) Corridor.
PB 241636 Evaluation of a Prototype Safety Warning System on the Gulf Freeway.
PB 241716 Variable Cycle Signal Timing Program (complete set).
PB 241717 Phase 1: Vol. 1—Development of Third-Generation Control Policies for the UTCS.
PB 241718 Phase 1: Vol. 2—Optimization of Traffic Signals in Networks by Mixed Integer Linear Programing.
PB 241719 Phase 1: Vol. 3—CYRANO (Cycle-Free Responsive Algorithms for Network Optimization)—Moderate, Congested, and Light Flow Regimes.
PB 241720 Phase 1: Vol. 4—Prediction Algorithms, Software and Hardware Requirements, and Logical Flow Diagrams.

ENVIRONMENT

Stock No.
PB 240167 A Sample Comparison of the Geomorphic Character of Two River Basins as Related to Susceptibility to Bridge Failure.
PB 240435 Roadside Establishment of Green Ash as Affected by Mulches, Herbicides, Slow-Release Fertilizers, and Sizes of Plant Stock.
PB 240436 Effects of Slow-Release Fertilizers on the Performance of Blueleaf Honeysuckle Grown in Nutrient Deficient Soil.
PB 240437 Comparison of Different Methods of Establishing Bur Oak on the Roadside.
PB 240585 Fixed Illumination for Pedestrian Protection—Phase I.
PB 240623 Snow and Ice Removal from Pavements Using Stored Earth Energy.
PB 241418 Impact Performance and a Selection Criterion for Texas Median Barriers.
PB 241421 HVOSM User's Manual.
PB 241422 Hydraulic Design Curves for Kansas Standard Depressed Curb Inlets.
PB 241455 Evaluation of High Intensity Sheeting for Overhead Highway Signs.
PB 241532 Field Evaluation of a Recharge Basin.

IMPLEMENTATION

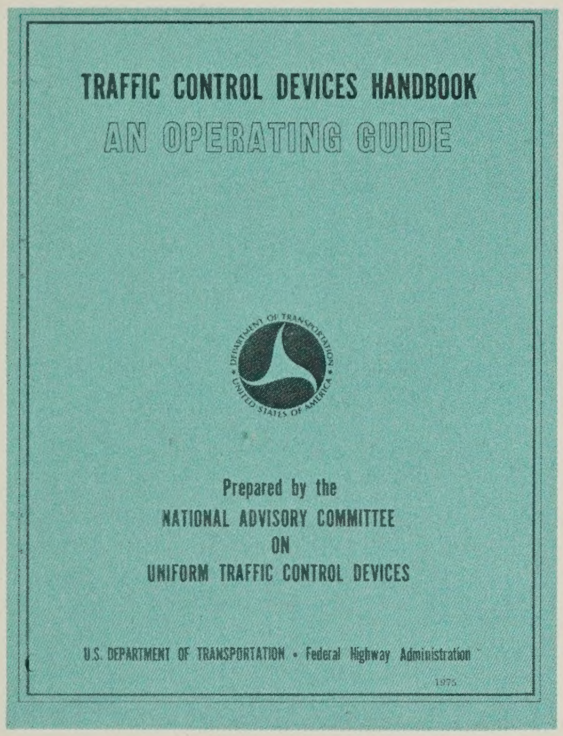
Stock No.
PB 239534 Development of Maintenance Methods and Cost Codes.
PB 239684 Fixed-Time Traffic Control for Signalized Diamond Interchange Complexes.
PB 240624 Polymer Concrete Overlay Test Program—Interim Report.
PB 240626 Debris Removal from Concrete Bridge Deck Joints—Final Report.
PB 240627 Presplitting—Final Report.
PB 240690 User's Manual for Membrane Encapsulated Pavement Sections (MEPS)—Final Report.
PB 241053 A Manual for Planning Pedestrian Facilities.
PB 241222 Development of a New Collapsing Ring Bridge Rail System.
PB 241425 Evaluation of Asphalt Concrete Produced by the Dryer-Drum Mixing Process—Interim Report.
PB 241497 A Correlation of Various Smoothness Measuring Systems for Asphaltic Concrete Surfaces—Final Report.
PB 241503 Vibratory Roller Evaluation Study—Interim Report 1.
PB 241573 Modern Concepts for Gradation Control. Phase IV: Granular Base Courses—Final Report.
PB 241691 Introduction to Concrete Polymer Materials.
PB 241743 A Portable Automated Data Acquisition System for Material Haul Documentation—Final Report.

PLANNING

Stock No.
PB 240361 Urban Corridor Demonstration Program Manhattan CBD—North Jersey Corridor: CBD Bus Reroute.
PB 240446 Feasibility Study for Use of Photographic Road Inventory Procedures.
PB 241823 Carpool Incentives and Opportunities.

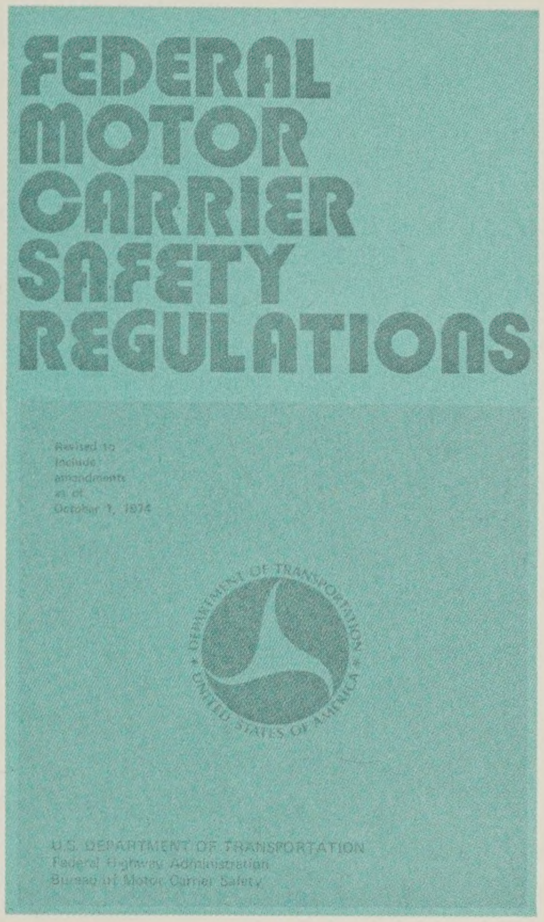
☆ U.S. Government Printing Office: 1975—620-833/5

New Publications



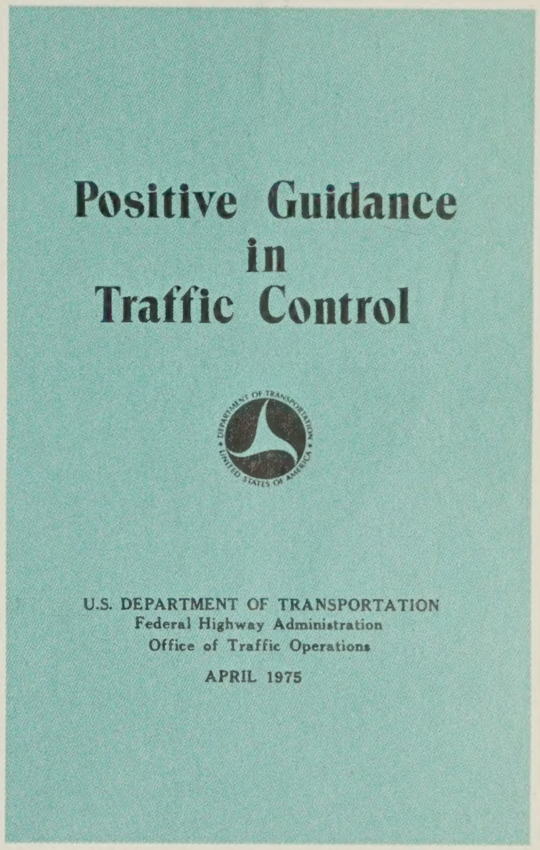
Traffic Control Devices Handbook—An Operating Guide is intended to supplement the provisions of the Manual on Uniform Traffic Control Devices and serve as a guide for those in State, city, county, or local jurisdictions who, because of their occasional contact with the details of installing needed traffic control devices, can benefit from additional reference material. It presents typical values or ranges of values used for implementing traffic control measures and provides examples of specifications, contract plan sheets, and work orders for those who are not familiar with the preparation of these documents. It is divided into three parts: signs, pavement markings, and signals.

The handbook may be purchased for $2.80 from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Stock No. 050–001–000938).



Federal Motor Carrier Safety Regulations provides, in one publication, the applicable motor carrier safety regulations for motor carriers operating in interstate or foreign commerce. This is a revised issue of the regulations, including amendments through October 1, 1974, parts 390 through 397. There are sections on qualifications of drivers; driving of motor vehicles; parts and accessories necessary for safe operation; notification, reporting, and recording of accidents; hours of service of drivers; inspection and maintenance; driving and parking rules when transporting hazardous materials.

This publication may be purchased for $1.55 from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Stock No. 050–004–00011).



Positive Guidance in Traffic Control presents an approach designed to provide high-payoff, short-range solutions to safety and operational problems at relatively low cost. The approach is based on the premise that a driver can be given sufficient information to avoid accidents at hazardous locations. Since few hazardous locations are identical, each must be individually analyzed to develop appropriate solutions. Positive guidance is a tool for both the analysis and the solution development. The pamphlet presents a procedure in outline form which illustrates implementation of the positive guidance approach.

This pamphlet may be purchased for 85 cents from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Stock No. 050–001–00094).

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