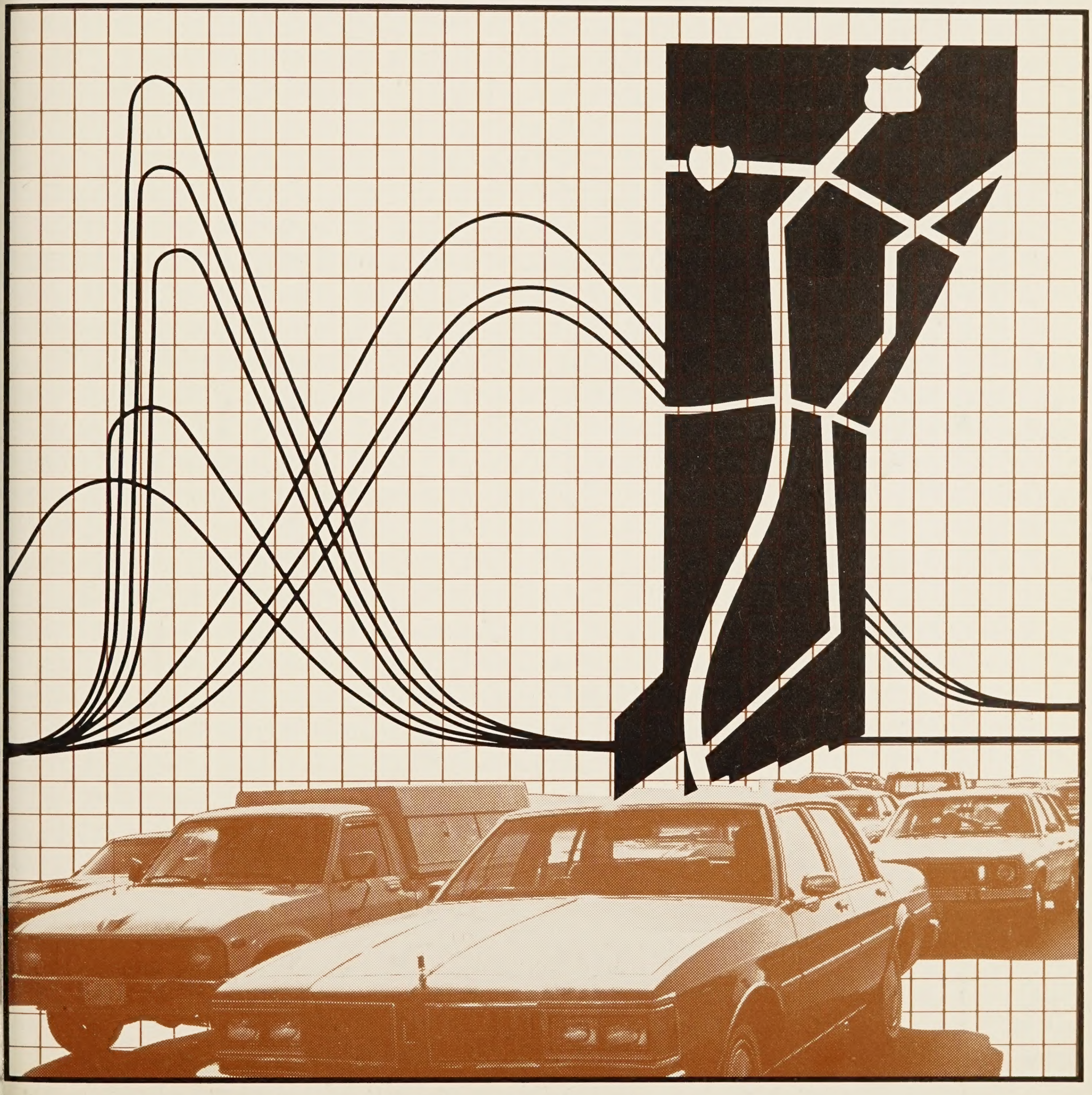
A JOURNAL OF HIGHWAY RESEARCH AND DEVELOPMENT
U.S. Department of Transportation
Federal Highway Administration

public roads

December 1981 Vol. 45, No. 3



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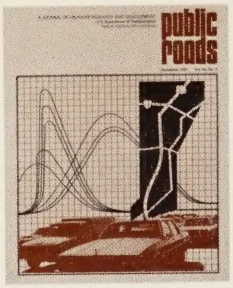

COVER:
Computerized traffic control leads to efficient use of the highway system.

IN THIS ISSUE

Articles

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U.S. Department of Transportation
Federal Highway Administration
Washington, D.C. 20590

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Washington, D.C. 20590

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Dallas Freeway Corridor Study

by

H. H. Bissell and B. T. Cima

Introduction

A primary goal of transportation systems management is efficient use of the highway system. One way to achieve this goal is through computerized traffic control. An early effort to control freeway corridor traffic took place in Dallas, Tex. This article reviews the history and the characteristics of the Dallas Freeway Corridor Project. The research results and the lessons learned from the project may be applicable to other jurisdictions contemplating implementing similar systems.

Corridor Location

The Dallas Freeway Corridor consists of a 78 km^2 (30 mi^2) corridor centered around the North Central Expressway (fig. 1). The corridor runs for approximately 19 km (12 miles) north from the Dallas central business district to the ring road, I–635. It has been the scene of dramatic office and residential development during the last decade.

The expressway itself was built according to older design standards with closely spaced interchanges and short exit and entrance ramps. The major arterial street, which roughly parallels the North Central Expressway, is Skillman Avenue. Skillman Avenue recently has been widened to three lanes in each direction.

Project Initiation

In the late 1960's, the city of Dallas began to experience operational problems on the North Central Expressway because of rising traffic levels. The city began to consider implementing the freeway surveillance and control techniques that were being demonstrated successfully in other cities such as Detroit, Mich., Chicago, Ill., and Houston, Tex. The city also realized that construction moneys to increase the capacity of the facility were limited. The Texas State Department of Highways and Public Transportation (SDHPT) concurred because construction of the remaining portions of the Dallas

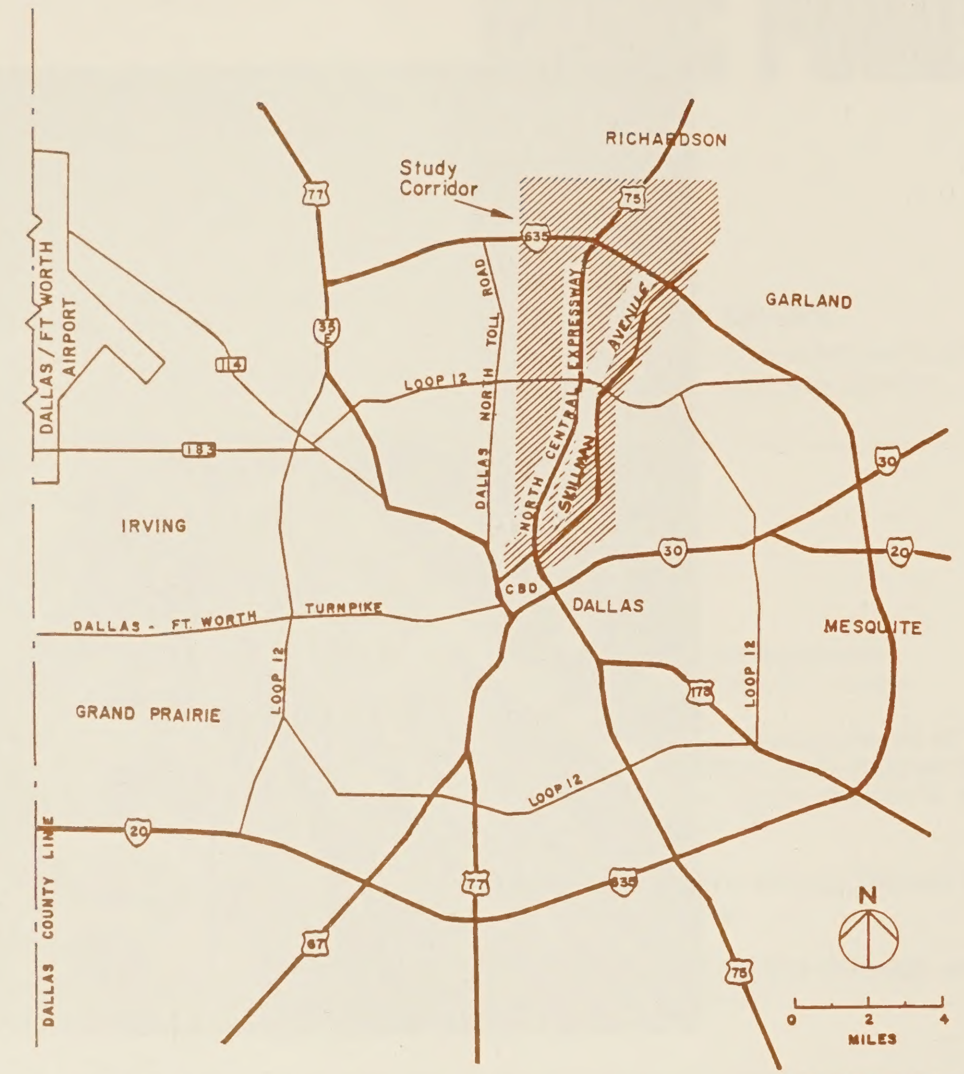

Figure 1.—Dallas Freeway Corridor System.

area freeway system, then underway, had higher priorities for the available construction funds.

During this same time, the Federal Highway Administration (FHWA) was involved in developing various computerized traffic control systems for both freeways and arterial streets. However, little research had been done to develop a corridor network into one integrated system. Texas Transportation Institute (TTI) was actively involved as a research contractor in these control system efforts, especially in Detroit and Houston.

In March 1968, representatives from FHWA, TTI, the city of Dallas, and SDHPT met and informally agreed that a research/implementation effort be carried out for the North Central Expressway Corridor. It was generally understood among the four organizations that:

- FHWA would support and fund the research effort that would include corridor control strategy development, system design, system programing, development of traffic control technique, and system evaluation.
- TTI, as the research agency, would carry out these functions, in addition to operating the control system until city personnel could assume that responsibility.
- SDHPT would provide operational hardware on the designated State highway system and designated urban system as funds became available.
- The city would provide operational hardware on the other sections of the city arterial system, in addition to operating and maintaining the hardware systems as the various research phases were completed.

Thus, the stage was set for the development, design, installation, and operation of a freeway corridor traffic control system.

The overall objective of the Dallas Freeway Corridor System Project was to develop and test traffic control and driver information systems and to optimize the flow of traffic in an intracity freeway corridor.

Freeway Corridor System Components

To meet this objective, an integrated system of seven components was developed. These include:

- Freeway ramp control—ramp metering.
- Freeway surveillance—closed-circuit television (CCTV).
- Frontage road intersection control—traffic signals.
- Arterial intersection control—traffic signals.
- Bus priority at signals.
- Driver information systems—rotating drum signs and telephone call-in.
- Control center—computers, display panel, and controls.

Following are a brief description and status report of the components installed in the Dallas Freeway Corridor to control traffic and provide driver information. The areas covered by the system components are shown in figure 2.

Freeway ramp control

Ramp meters were installed on 35 entrance ramps with an extensive network of 400 vehicle detectors (fig. 3). A textured surface treatment applied to the North Central Expressway destroyed many of the vehicle detectors embedded in the roadway. The ramp meters were operated on a fixed-time basis between September 1977 and December 1979 while the detectors were being replaced. The ramp meters operate on a demand responsive, gap sensing control logic that controls the throughput on the freeway by controlling the entry rate of the vehicles at onramps. Communications between the control center and the ramp meters and detectors are provided through leased telephone lines. The freeway control program initially was operated on an IBM 1800 computer and later was programed for a Data General NOVA Model 820.

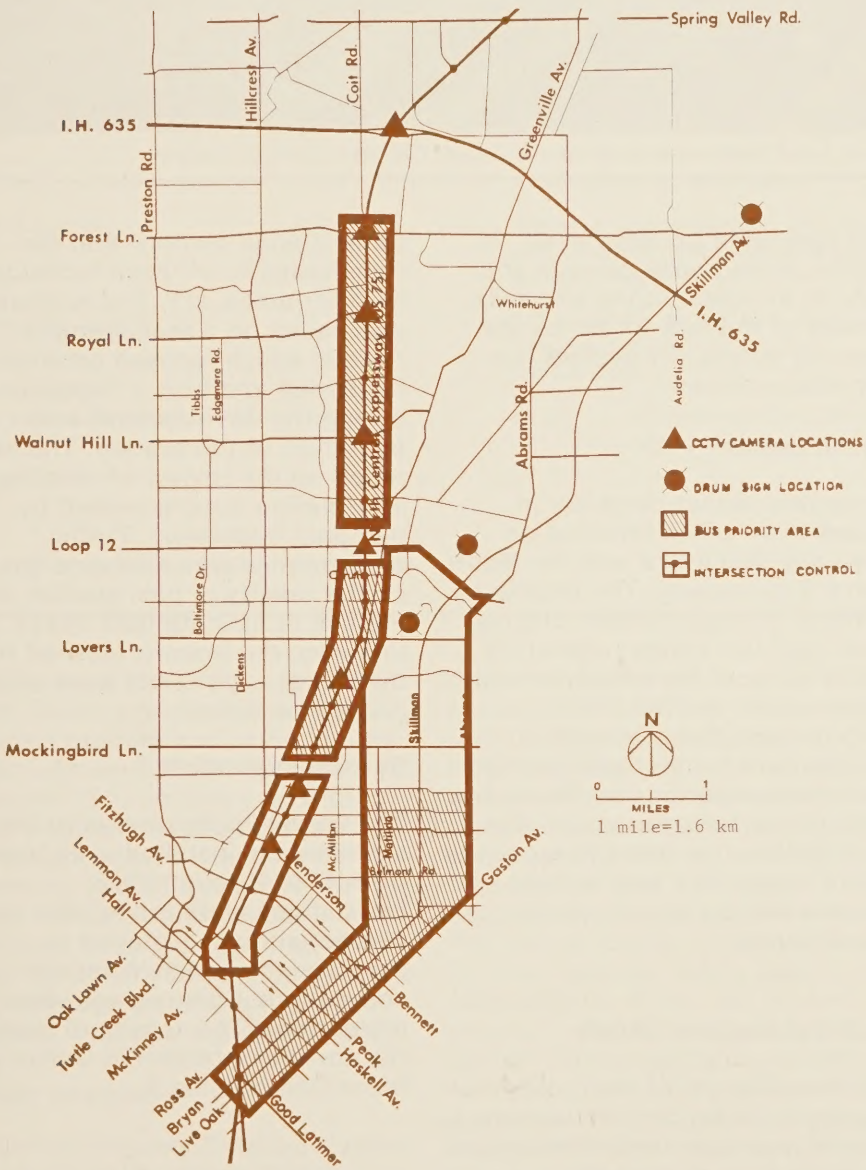

Figure 2.—Corridor area under control.

Freeway surveillance

Eight CCTV cameras were mounted on luminaire poles at intervals along the North Central Expressway, and one camera was positioned on top of the 10-story building where the control center was located. Monitors for each camera were located in the control center, with controls to pan, tilt, and zoom each camera. The eight cameras were connected to the control center by a coaxial cable laid directly in the median. The coaxial cable has now deteriorated, and currently, the one camera on top of the building is the only one working.

Frontage road intersection control

The frontage road was divided into three sections of interconnected signals controlling 15 diamond-type intersections. The three sections were required because the frontage road is broken by highway route Loop 12 and a railroad crossing. Approximately 150 vehicle detectors were installed to provide traffic flow data to the control center. Each intersection has a minicomputer in a field cabinet that samples the detectors, drives the signal lights, and communicates with the control center. The minicomputers also were programed to operate the intersections independently without control from the center. The minicomputers have experienced some hardware problems. Because replacements for the minicomputers are no longer available, selected intersections will be converted to fixed-time, and the minicomputer parts will be used to repair remaining installations as intersection hardware fails. The intersections are connected to the control center through leased telephone lines. The signal timing control patterns vary as traffic demands change. A three-phase or a four-phase signal operation is used depending on the best timing for current traffic. Progression and added green times are provided along the frontage road when an incident on the freeway forces traffic to use the frontage roads.

Arterial intersection control

Minicomputer controllers were installed at 13 intersections on and north of Mockingbird Lane. The control of these intersections is similar to the control of the frontage road signals, with timing and phasing changes related to measure traffic demands. The widening of Skillman Avenue has had some adverse impact on the effectiveness of this component, as well as the availability of Skillman as an alternate route within the corridor. Construction is now completed.

Bus priority at signals

A bus priority system was implemented to improve bus travel times in the corridor. The 63 intersections south of Mockingbird Lane were instrumented with approximately 100 bus detector loops and 50 magnetometer traffic detectors. About 110 buses were equipped with transmitters to activate the bus detectors. The 62 intersections are run on time-of-day signal patterns that are stored in one of the control center computers. Phase times are extended or shortened when buses are detected. Communications to the signal controllers from the control center are through leased telephone lines.

Driver information systems

Three large changeable message drum signs were erected along Skillman Avenue to inform drivers when the North Central Expressway was congested and to suggest alternate routes. The sign messages were selected from the control center by way of telephone lines by operators observing conditions on the CCTV. Two of the signs were removed during the widening of Skillman. Because most of the CCTV system is not operating, the third sign is not being used.

A pretrip dial-in telephone system was installed in the control center where recorded messages told the caller about traffic conditions on the North Central Expressway.

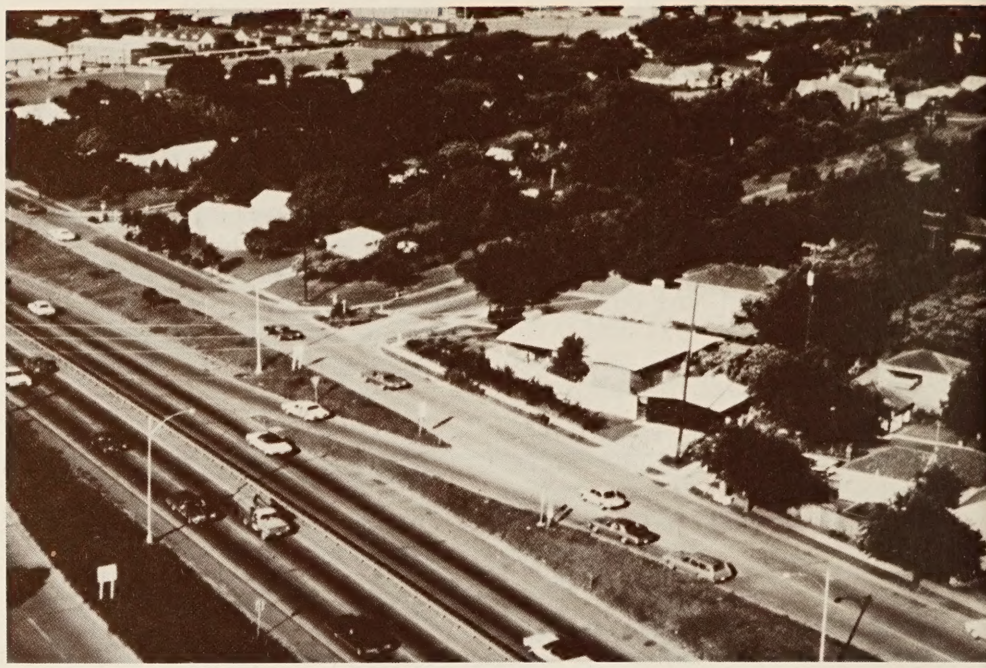

Figure 3.—A metered onramp on the Dallas Freeway Corridor System.

Messages were updated every 10 minutes during peak periods and every 30 minutes during offpeaks. Because of the lack of use by the motoring public, the system has been discontinued.

Control center

The control center currently is housed in an office building on University Boulevard and the North Central Expressway. The television monitors, a large corridor display board, and the system operators control console are located in the control room. An IBM 1800 computer and Data General minicomputers with teletypes and data communications hardware are located in an adjacent room. The city of Dallas now plans to move the control center to a new central business district signal system control center.

Final Evaluation Study

A final evaluation of the Dallas Freeway Corridor System was made to develop a clear understanding of the application of corridor surveillance and control techniques to the Dallas corridor and the transferability of these techniques to other areas. (*1*)[1] The evaluation was based on a management review, which focused on what happened and how it happened during the development and operation of the system. The study relied on the review of existing information supplemented by personal interviews. Traffic performance was assessed based on the results of past studies and the use of existing data bases. In this way, the lessons learned from the Dallas experience were captured and documented.

System evaluation

The various components of the system were installed over 10 years, from 1968 through 1978. Evaluations were made after each component was installed to measure the effects from the component. Cost records were maintained. The results of installing the corridor systems in Dallas are presented in table 1.

[1] Italic numbers in parentheses identify references on page 94.

Table 1.—Dallas Corridor System costs and effects (*1, 2*)

System component	Installation cost	Year installed	Major measured effects
	dollars		
Freeway ramp control (35 ramps)	580,509	1971	Freeway speeds increased from 1.2 to 26%. Rear end ramp accidents were reduced by 50%. Overall accidents in the control direction were reduced about 15%.
Frontage road signal control (15 intersections)	898,191	1974	Frontage road travel times reduced about 25%, while traffic volumes increased 7.5%.
Arterial intersection control (13 intersections)	518,728	1976	Arterial control (Mockingbird Study) showed traffic velocities increased 5% to 17% in peak direction without sacrificing opposing travel operations.
Bus priority system (62 intersections)	1,107,594	1978	Frontage road buses' travel times decreased by about 14% while automobiles' increased 16%. Other bus route improvements were less, with less effects on automobile traffic. Transit service and patronage increased.
Closed-circuit TV	80,000	1971	Improved incident detection verification and management. No quantitative measurements reported.
Rotating drum signs (3 signs)	25,000	1973	73% of drivers that had seen diversion messages said they changed their planned route.
Dial-in traffic information system	300	1975	Initially, calls numbered over 1,000 per day. Later, calls numbered only 50–100 per day.

In addition to the initial costs, the city of Dallas spends about $200,000 a year to operate and maintain the system. About $92,000 are spent for personnel (5½ person-years for a signal engineer, a programer, maintenance technicians, and operators) and $108,000 are for other costs including leased telephone lines ($30,000), electrical power ($44,000), control center rent ($17,000), computer maintenance ($10,000), and spare parts ($7,000).

System integration

The Dallas Freeway Corridor System was an early attempt to develop an integrated traffic management system in a freeway corridor. The computerized system never evolved into the automated system originally desired because the operator at the control center had to manually coordinate system components. This automated real-time control has not been accomplished at any other sites in the United States to date; Chicago, Ill., Los Angeles, Calif., and Minneapolis, Minn., all rely on manual intervention between system components. It is planned, however, that a fully automated system will be installed on Long Island, N.Y. The Integrated Motorist Information System currently being designed for the New York Department of Transportation and FHWA is scheduled to be installed during 1982 and 1983.

Spinoff Research Results

In addition to the direct research on freeway corridor systems obtained from the Dallas North Central Expressway studies, the following achievements were made as a result of the research conducted.[2]

1. PASSER II arterial traffic signal timing computer model was developed. It determines left turn pattern sequence and timing for arterial streets and plots the resulting time-space diagram.

2. PASSER III interchange and frontage road traffic signal timing computer model was developed to analyze diamond interchange phasing and to determine the interchange signal timing pattern for freeway frontage roads.

3. PASSER IV currently is being developed by TTI for SDHPT to obtain optimum movement of traffic within a freeway corridor. This model combines the two models mentioned above.

4. The concept of "stand alone satellite" systems, which are coordinated by a central computer, was demonstrated in Dallas and is now incorporated in the Flexible Advanced Computer Traffic Signal (FACTS) system being developed and implemented by SDHPT in Texas. FACTS also includes two intersection control concepts tested as part of the Dallas study.

5. It was learned from the Dallas study that no more than two signal cycles should be used to change an offset, and the offset should be changed by lengthening the cycle during transition if the new offset falls within 0 and 70 percent of the cycle length. The cycle should be

[2] Letter from M. G. Goode, Engineer-Director, Texas State Department of Highways and Public Transportation, Austin, Tex., Feb. 17, 1981.

shortened to reach the new offset if the offset falls within the last 30 percent (70–100 percent) of the cycle.

6. Three- and four-phase traffic signal controls for isolated freeway interchanges for full-activated operation were developed by SDHPT as a result of the Dallas study.

Major Lessons Learned

Major lessons were learned from the evaluation of the Dallas Freeway Corridor System. The evaluation demonstrated the effectiveness of computer traffic control. Both ramp and intersection control can be implemented successfully using computer technology. The following advice is offered to other jurisdictions contemplating implementing a sophisticated traffic control system:

- *Define resource requirements.* All parties involved in the project should be aware of the resources required to build as well as operate the system.
- *Obtain commitments.* Once the resource requirements for the project have been determined, formal agreements that define responsibilities and funding sources should be made between all parties involved.
- *Involve operating agency.* The agency that eventually will operate the system must be involved in the planning and implementation of the system to insure the operational effectiveness of the system.
- *Assign project coordinator.* One person should be designated as the project coordinator.
- *Balance research and operational needs.* In considering research and operational needs of a project, insure that the system can be economically and effectively operated after the research phases are completed.
- *Use off-the-shelf technology.* The use of tested off-the-shelf equipment generally results in an effective system that is easy to maintain.
- *Separate evaluation from implementation.* The implementation contractor should not also have responsibility for evaluating the system.

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(*1*) Bart Cima, Amin Hassam, Steve Hetruck, and Dave Henry, "Evaluation of the Dallas Freeway Corridor System," Report No. FHWA/RD–81/058, *Federal Highway Administration,* Washington, D.C., June 1981.

(*2*) James D. Carvell, Jr., "Dallas Corridor Study," Report No. FHWA–RD–77–15, *Federal Highway Administration,* Washington, D.C., August 1976. PB No. 270515.

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

Howard H. Bissell has been a research engineer with FHWA for the past 10 years. He was project manager for the research conducted on the North Central Expressway Corridor in Dallas, Tex. Before joining FHWA, Mr. Bissell was in private consulting with TRB and worked for city, State, and county governments.

Bartholomew T. Cima was the principal investigator on the research contract "Evaluation of the Dallas Freeway Corridor System" while he was a consultant with Peat, Marwick, Mitchell and Company. Before that he served as a study engineer on the Chicago Area Expressway Surveillance Project for the Illinois Department of Transportation. Currently, Mr. Cima is working in the policy planning area for the District of Columbia Department of Transportation.

Proposed Plans for Measurement of the Aerodynamic Behavior of the Luling, Louisiana Cable-Stayed Bridge

by
Hugh A. Thompson, Robert N. Bruce, Jr.,
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During the last decade, the Federal Highway Administration (FHWA) has sponsored and conducted research studies on wind and earthquake loading related to highway structures. Research under Project 5A, "Improved Protection Against Natural Hazards of Wind and Earthquake," in the Federally Coordinated Program of Highway Research and Development, has covered design procedures, computer modeling, field instrumentation systems, wind tunnel testing, retrofitting, and full-scale field testing.

A major effort within Project 5A has been to develop automated bridge instrumentation systems and to evaluate data collected by such systems to better characterize the wind environment near highway structures, particularly bridges. Data usually collected consist of wind parameters such as velocity components, direction, and turbulence intensity and associated bridge response parameters such as displacements, accelerations, and damping.

This article describes current plans to characterize the wind climate and associated structural response of the new cable-stayed girder bridge over the Mississippi River at Luling, La. The three major wind interactions of interest in the investigation—vibratory motion of stays, vibratory motion of the bridge deck, and extreme wind response—are discussed. The instrumentation and analytical techniques to be used in quantifying the wind climate and bridge reactions are described as are the preparatory tasks for this investigation.

Introduction

In 1970, FHWA installed its first field wind instrumentation system on the Newport, R.I., suspension bridge in cooperation with the Rhode Island Turnpike and Toll Bridge Authority. Sensors were located on outrigger booms and positioned symmetrically along the main span. While in operation, the system successfully recorded a portion of Hurricane Doria; these data have been analyzed and documented in recent reports. (*1*)[1]

In a cooperative effort with the Alaska Department of Transportation and Public Facilities, the system was moved to Sitka, Alaska, in late 1972 and installed on the Sitka Harbor cable-stayed bridge where data were obtained for 5 years. Vortex excitation of the free standing pylon towers of the bridge was recorded, evaluated, and documented. (*1*)

In 1977, the instrumentation was moved to Twin Falls, Idaho, to monitor column vibration on the Perrine Bridge over the Snake River Canyon. The instrumentation presently is monitoring wind and structural motion on the new Pasco-Kennewick Bridge in Washington State.

A new wind instrumentation system is being designed and developed for the Luling, La., cable-stayed bridge over the Mississippi River. The bridge, scheduled for completion in 1982, is a few miles north of New Orleans and connects Luling and Destrehan, La. This bridge has been the subject of an extensive wind tunnel section model testing program sponsored by the Louisiana Department of Transportation and Development and conducted by FHWA at the Fairbank Highway Research Station in McLean, Va. This program has provided valuable insight into the wind stability of various proposed cross sections and has resulted in the selection of a preferred superstructure section. In view of the extensive research experience with the Luling structure and the likelihood of hurricane wind forces at the bridge site, the bridge was selected for instrumentation.

[1] Italic numbers in parentheses identify references on page 100.

Bridge Description

The Luling bridge crosses the Mississippi River upstream from New Orleans in essentially an east-west direction. River flow at the crossing point is north-south. Principal features of the superstructure are shown in figure 1.

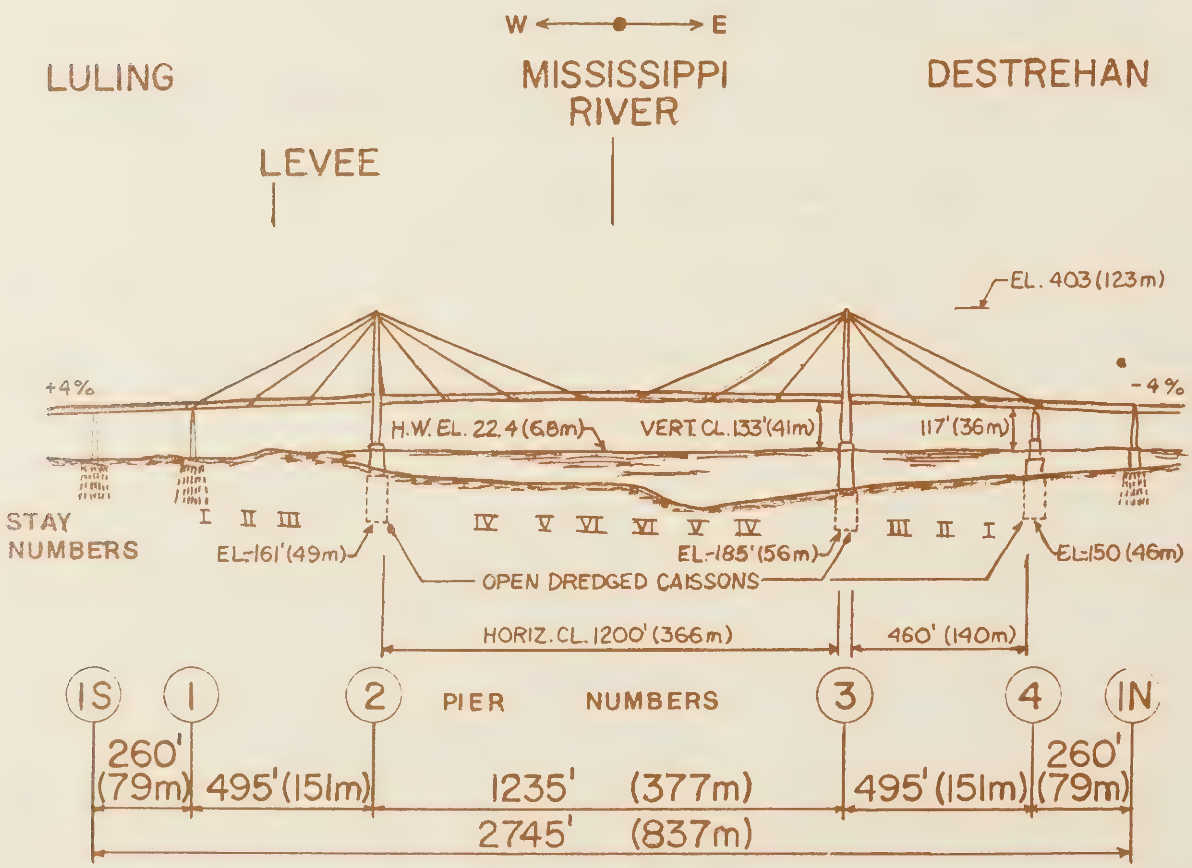

Figure 1.—Superstructure of the Luling bridge, La.

The Luling bridge is the largest cable-stayed bridge ever built in an area prone to hurricane winds. The five-span bridge has an overall length of 837 m (2,745 ft). The center span is 376 m (1,235 ft) long, adjacent side spans are 151 m (495 ft) long, and end spans are 79 m (260 ft) long. The center and adjacent side spans are cable-stayed from pylon towers that are 107 m (350 ft) high. The center span provides a minimum navigational clearance of 41 m (133 ft). The deck is approximately 4 m (14 ft) deep, and the six-lane roadway with shoulder has a maximum width of about 28 m (93 ft).

Objectives of the Luling Bridge Measurements

Three aerodynamic aspects of the Luling bridge have been selected for measurement and investigation—wind-induced vibrations of the stays, vortex-induced motion of the bridge deck, and the wind climate and dynamic response of the structure under hurricane wind conditions.

Wind-induced vibration of stays

Motion of the cable stays, the first phase of this investigation, has been assigned the highest priority for measurement. Such motions may arise through two distinct mechanisms—Aeolian or vortex shedding and wake effects.

As the air flow divides and recombines about the circular cross section of a cable, the boundary layer separates on the aft side, forming two counterrotating vortexes in the wake. These vortexes are swept into the mean flow, alternately from one side of the cylinder to the other. The regular pattern lying in the wake of such a circular cylinder is sometimes called a von Karman vortex street, although this designation is strictly correct only with low Reynolds numbers. With the departure of each vortex, a small periodic force perpendicular to the mean flow is generated on the stay. If the frequency of these periodic forces coincides with a natural frequency of the cable, then resonant vibrations of maximum amplitude are generated. When there are sufficiently low levels of mechanical damping or insufficient energy dissipation mechanisms within the cable system, these vibrations can induce fatigue damage at points of high strain, thereby shortening the useful life of structural members.

At several locations on the Luling bridge, the stays consist of four circular cables arranged in a square pattern. Horizontal wind blowing from various directions on the bridge causes one of the individual cables to lie downwind, or in the wake, of another. In these cases the leeward cable will not be subjected to a uniform flow field. Rather, the wake of the windward cable will contain a region of reduced windspeed where the turbulence intensity is quite high. Moreover, the frequency of this turbulence is dominated by vortexes shed by the windward cable. This combination of factors may generate resonant vibrations of large amplitude on leeward cables, even if the windward cable remains still.

Measurement of wind-induced motion of the cable stays has been assigned the highest priority in this investigation because of recent fatigue problems with the stays of the Brotonne Bridge over the Seine River in Rouen, France. Moreover, both Aeolian and wake-induced oscillations are common structural problems. For example, Aeolian problems are common on the conductors of transmission lines in the electric utility industry, on river crossings of pipelines, on stacks and chimneys, on heat exchanger tubes, and on pilings in river currents. With increasing demand for moving large quantities of energy at high voltage, bundled conductors recently have been developed for use by the electric utility industry. The configurations of bundled conductors are similar to those of the stays, and they commonly produce wake-induced vibrations called subconductor oscillations.

Aeolian and wake-induced motion of cable stays will be monitored by nine accelerometer stations. Each station will consist of a pair of accelerometers with their sensitive axes mutually perpendicular and also perpendicular to the axis of the cable. To measure Aeolian vibration, each of the six upriver stays at pier 2 will be instrumented with an accelerometer station on an outboard cable. To detect wake-induced motion, three inboard cables in stays I, IV, and V at pier 2 also will be instrumented with accelerometer stations.

The wind climate of the bridge is to be defined using five anemometers. These devices can measure the wind velocity components in three mutually perpendicular directions. The anemometers are to be located at the center and quarter points of the main span and at the centers of the two adjacent spans. Each of the anemometers will be pole-mounted on the centerline barrier of the highway 12.1 m (39.8 ft) above the parapet. To minimize interruptions in data collection from vehicular collisions, conduit and cable leads from the anemometers on the barrier will be routed beneath the bridge deck to the data collection point. In addition, provisions will be made for the possible installation of an anemometer on top of pier 2. An anemometer can measure instantaneous windspeed and direction, but not components of the wind. Installation provisions include four steel eyes, one located at each corner of the tower top, and a base plate predrilled to accommodate 28.4 mm (1.12 in) anchor bolts.

Cable leads for these instruments are to be run to a data acquisition system located in an air-conditioned enclosure beneath the bridge deck at pier 2. The enclosure will be equipped with 120 volts a.c. power as well as battery storage capability to insure data acquisition during power outages. This system stores sampled digital data on magnetic tape for subsequent reduction and analysis.

This data acquisition system will record cable accelerations in two mutually perpendicular directions. Through software, these time domain signals will be resolved into the elliptical displacement trajectory of the longitudinal axis of a cable at the accelerometer station. Using Fourier techniques, the spectrum of the major axis of this displacement ellipse will be developed, providing amplitude and associated frequency data. The mode shape of each frequency may be identified through analytical techniques. These data will be combined either with estimates of the flexural rigidity of the cable or experimental measurements of the rigidity to infer the magnitude of bending strains at the end point of the stays.

Anemometer measurements will provide values of the mean windspeed, the direction of the mean wind, and the relation of these parameters to bending strains induced in the stays. This combination of data will allow a probabilistic estimate of fatigue damage at the cable anchorage or, equivalently, an estimate of the useful life of the stays. These estimates may be compared with design predictions of fatigue effects.

Vortex-induced motion of the bridge deck

A bridge deck also is subject to mechanical excitation through vortex shedding, as well as to other aeroelastic phenomena. Because the cross section of a bridge departs so substantially from the circular, the vertical modes of vibration and the torsional modes of vibration of the bridge deck are of interest in the second phase of this study. Excitation of these vibratory modes has, in some bridges, caused structural failure. In less severe cases where structural damage is not a concern, motions perceptible to the public have caused adverse publicity. These problems have been addressed during the design phase of bridge construction by a combination of analytical and experimental techniques involving wind tunnel studies of proposed bridge cross sections. (*2*) Based on results of these studies, fairings were added on the cross section of the Luling bridge to obtain an aerodynamic shape having minimal sensitivity to vertical and torsional excitation. (*3*)

Because of the potentially disastrous consequences of these aeroelastic phenomena, the combination of analytical and experimental techniques is usually applied conservatively, perhaps adding significantly to the construction cost of some bridges. The degree of conservatism in the design of the Luling bridge is of principal interest in this phase of the research.

Bridge deck motion will be monitored by seven accelerometer stations located at the midspans of each of the four side spans and at the center and quarter points along the main span. Each station will consist of two accelerometers with vertical sensitive axes, located equidistant from the bridge centerline. Each accelerometer will be attached to the bottom flange of the floorbeam inside the box girder approximately 8.5 m (28 ft) from the bridge centerline and will be ventilated by an opening in the box girder to prevent operating temperatures from exceeding 71° C (160° F). Data from these accelerometers, together with output from the anemometers, will be recorded with the data acquisition system.

Analysis of data from these accelerometers will provide spectra of the vertical and torsional displacements of the bridge deck. From these spectra, displacement amplitudes, mode shapes, and combinations of vertical and torsional motion may be inferred. The torsional acceleration of the deck may be obtained by computing the difference between signals from the two accelerometers at a particular station. By averaging the signals, the vertical acceleration of the deck may be obtained. Both averaged and differenced signals must be integrated twice to obtain displacement and transformed into the frequency domain to obtain spectra. In this phase of the work, wind measurements provide values of the mean velocity and the intensity of turbulence associated with each level of deck motion. The Gill anemometry will provide measurements of the wind components in three mutually perpendicular directions—perpendicular to the bridge span, parallel to the span, and vertical to the span. These instantaneous measurements will be averaged over time to produce mean values of the wind components, which then may be combined vectorially to provide mean windspeed and direction. The turbulence intensity can be obtained by subtracting mean values from the instantaneous measurements of each component and computing the variance of fluctuation in component windspeeds.

Finally, the mechanical damping of the bridge must be measured. Although the details of these measurements are not yet complete, it is felt that the value of the mechanical damping is crucial to any complete understanding of bridge deck stability and motion.

Comparisons of actual bridge motion under known wind conditions with results from design studies based on wind tunnel data will complete this second phase of the investigation. The role of turbulence in these comparisons is of special interest. It has been suggested that atmospheric turbulence is a significant parameter in bridge stability. However, until quite recently, wind tunnel studies of bridge models have not included any simulation of turbulence.

Extreme wind climate and bridge response

The third phase of this study involves measuring the characteristics of extreme winds at the bridge site and the dynamic response of the bridge to such winds. Beginning in the early 1960's, the design of structures against extreme wind loads departed from the traditional view of a steady wind producing a static structural response. The wind began to be perceived as a steady flow with periodic velocity fluctuations caused by turbulence induced mechanically by the roughness of the upwind terrain. The response of a structure to this combination of air flows consisted of a steady response to the mean wind and a dynamic, vibratory response to the fluctuating components of turbulence. By the mid-1970's, this view of the design process had gained such widespread acceptance that it was incorporated into an extensively revised version of the American National Standards Institute (ANSI) code. (*4*) In this phase of the Luling bridge investigation, the validity of this new approach will be examined in the case of a very large structure exposed to extreme hurricane winds. Anemometer and accelerometer measurements on the bridge deck and acceleration measurements at the top and intermediate height of one leg of pier 2 will be used in this phase.

A hurricane wind will be defined as the mean velocity, spectra of velocity fluctuations in three mutually perpendicular directions at each anemometer, and correlation coefficients relating fluctuations of like turbulent components between various anemometer stations. In the measurements of correlation coefficients, the component fluctuating in the direction of the mean wind will be emphasized.

Two different presentations of wind parameters will be developed. One will use the bridge to define principal directions—perpendicular to the span, parallel to the span, and vertical to the span. These directions are of principal interest to bridge designers because they form the basis for many different analyses of bridge performance. It is unlikely, however, that a major hurricane will strike the bridge with precisely this orientation, thus providing these data. The second presentation is dictated by meteorology, independent of bridge orientation. This system uses principal directions that are parallel to the mean wind, transverse to the mean wind, and vertical to the mean wind.

Accelerometer data will provide displacement spectra from each station as well as the phase relation of various stations, so that mode shapes and frequencies of the bridge may be obtained and compared with analytical projections.

This phase of the investigation provides an opportunity to compare the dynamic design techniques proposed by others for the treatment of extreme winds with actual performance data from the structure. It also provides an opportunity to compare the characteristics of hurricane winds with spectra obtained from weather fronts and thunderstorms. Although these comparisons cannot be made until a severe hurricane occurs, the capability to collect and analyze these data should be developed early so that such information can be acquired in an early storm.

Preparatory Work

The preceding discussion outlined the three main objectives and phases of the Luling bridge study. The following discussion describes the preparatory work proposed for the study. This work consists of validating the instrumentation package and developing software for processing instrumentation data.

Instrumentation for this project is in the form of components, whose calibration and compatibility will be checked and verified in the laboratory. System compatibility will be examined by assembling the entire package and processing analog signals having known characteristics. In addition, because some environmental stresses associated with bridge installation are equal to or more severe than those specified by the manufacturer of the components, the system will be checked for degradation of signal quality in this severe environment using a chamber with controlled temperature and humidity.

The instrumentation contemplated for the Luling bridge produces data so rapidly that computer reduction will be required. The software necessary to process these data will be prepared, documented, and verified. Digital information recorded on magnetic tape by the data acquisition system will be translated into displacement spectra, phase spectra, turbulence spectra, mean wind velocities, and correlation coefficients. Computer programs that already produce the desired quantities from sampled data systems will be modified to be consistent with the format of the acquisition system and later will be verified. Where programs have not yet been developed, they must be written, documented, and tested.

After the electrical system is checked and software capability is developed, the entire system will be tested. Analog signals of known characteristics will be processed through the data acquisition system and reduced by the software to spectra of various kinds. These test signals also will be processed by both analytical and established analog techniques to produce spectra to compare with the results of the system proposed for use on the Luling bridge. A favorable comparison between these techniques will validate the procedures proposed for the Luling bridge.

Earlier efforts to monitor long-span bridge motion have been hampered by instrumentation difficulties and the need to develop extensive software capability for data processing. The proposed effort seeks to avoid or minimize these difficulties.

For example, instrumentation components that either fail to function properly or are incompatible with system requirements may be replaced at less expense if their shortcomings are demonstrated in the laboratory before installation on the bridge. Once the instrumentation package is installed on the bridge, extensive downtimes to diagnose malfunctions and acquire, deliver, and install replacement components must be avoided so that the opportunity to measure bridge response to a severe hurricane is not lost. In the laboratory, diagnostic procedures may be developed to rapidly isolate malfunctioning components. The instrumentation and analysis capability should be established independently so that errors in different components can be isolated.

Conclusion

The Luling, La., cable-stayed bridge is an excellent opportunity to advance our understanding of long-span bridge stability. The cable-stayed bridge design concept is relatively new in the United States, and the Luling bridge is located in a hurricane and tornado region. Field measurements on the completed structure should provide greater insight into the actual performance of long-span, cable-stayed bridges inservice. Accurate field data will enable researchers to access laboratory and analytical procedures and ultimately provide bridge designers with improved guidelines for sound, realistic designs for wind stability.

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

Rapid Measurement of the Chloride Permeability of Concrete[1]

by
David Whiting

This article describes the development of a device and test procedure that can assess rapidly the permeability of various kinds of concretes to chloride ions. The method consists of monitoring the amount of electrical current passed through a concrete specimen when a potential difference of 60 to 80 volts d.c. is maintained across the specimen for 6 hours. Chloride ions are forced to migrate out of a sodium chloride (NaCl) solution subjected to a negative charge through the concrete toward an electrode maintained at a positive potential. Concrete either can be tested in the laboratory in the form of 102 mm (4 in) diameter core specimens or in the field nondestructively in reinforced horizontal surfaces such as bridge decks. In addition to ordinary portland cement concretes, the procedure can be used with latex-modified, internally sealed, low water-cement ratio (Iowa), polymer-impregnated, and polymer concretes.

Introduction

Deterioration of concrete bridge decks built before the general use of deck protective systems in the early 1970's in areas where deicing salts are used is a serious problem for many State and Federal highway agencies. An estimated $6.3 billion are needed to restore the Nation's Federal-aid highway system bridge decks. (*1*)[2]

The Federal Highway Administration (FHWA) and other agencies have developed techniques to prevent the corrosion of reinforcing steel by deicing salts and the resultant delamination and spalling of the riding surface. Techniques commonly used fall into three categories—coated reinforcing steel, waterproof

[1] This article is a condensation of "Rapid Determination of the Chloride Permeability of Concrete," Report No. FHWA/RD–81/119, *Federal Highway Administration,* Washington, D.C., August 1981.

[2] Italic numbers in parentheses identify references on page 112.

membranes, and low permeability concretes. Low permeability materials that help keep chloride ions (Cl^-) from permeating the concrete are:

- Polymer-impregnated concrete (PIC).
- Latex-modified concrete.
- Dense portland cement concrete ("Iowa Method").
- Internally sealed concrete.
- Polymer concrete (PC).

Figure 1 is a graphic representation of the relative effectiveness of several such materials in halting chloride ion penetration. (*2*) Also included are conventional concretes with moderate (water-cement ratio w/c=0.4) to high (w/c=0.6) permeabilities.

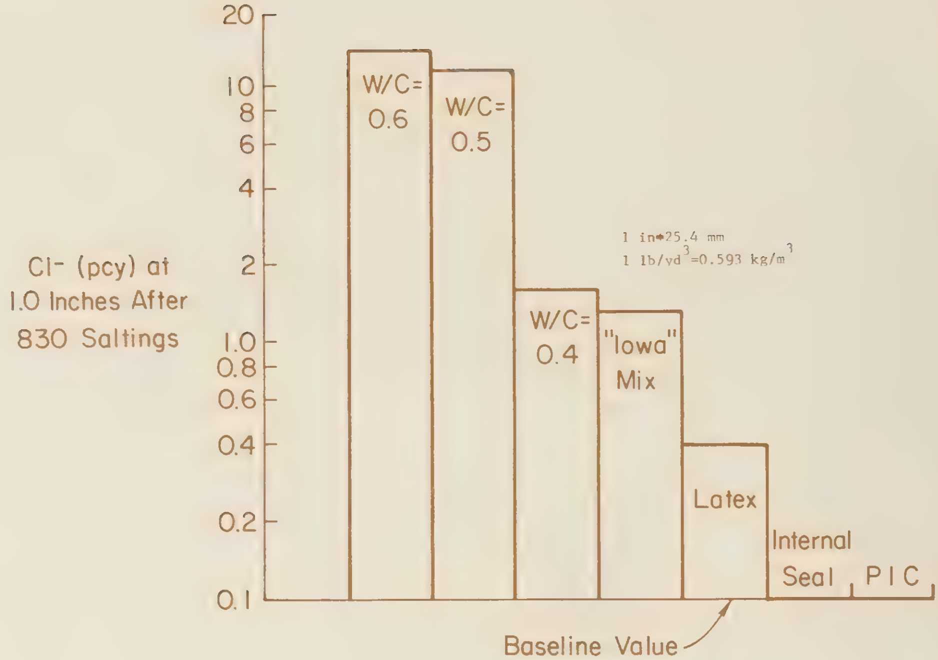


Figure 1.—Chloride permeabilities of various concretes. (2)

Rapid test procedures are needed to judge the field effectiveness of these materials and to evaluate proposed new materials or treatments. FHWA therefore sponsored research to develop a test apparatus and procedure to determine chloride permeability both inplace and in the laboratory. The two major objectives of this project were to develop and test a rapid, destructive permeability device that could be used in the laboratory on sections of concrete (such as cores) taken from structures and to develop and test a rapid, nondestructive permeability device that could determine the permeability of rigid concrete members (such as bridge decks) inplace in field installations.

The first phase of the study was a comprehensive literature review. Although many accepted techniques are available to determine the permeability of concrete to high waterheads, water vapor, and air, the review showed that only long term ponding methods currently are available to determine chloride permeability.

Although an empirical correlation between water and air permeability and chloride ion permeabilities may be established, the differences in transport mechanisms preclude the use of data gathered by conventional techniques in predicting long term chloride permeability without extensive cross correlations. Because permeability is an operationally defined parameter, there are no fundamental theoretical relationships between it and basic physical properties. Thus, the measurement of such properties as density, porosity, wave velocity, or other physical properties held little promise.

A more promising technique was suggested by previous research that had demonstrated that it was possible to remove significant amounts of chloride ions from concrete by applying an electric field between the surface of the concrete and the reinforcing steel. (*3, 4*) This technique could be used as a chloride permeability method if the polarity were reversed, that is, by making the reinforcing steel anodic (+). Chloride ion, having a negative charge, would migrate *into* the concrete. As the electrical resistivity of concrete decreases with increasing chloride ion concentration, a measure of the increase in current with time could be correlated with the amount of chloride entering the concrete. Additionally, at the end of the test, samples could be taken and analyzed with wet chemicals to verify that chloride had permeated the concrete.

The Applied Voltage Technique

Basic principles

The applied voltage technique is based on the principle that charged ions, such as chloride, will accelerate in an electric field toward the pole of opposite charge. The ions will reach terminal velocity when the frictional resistance of the surrounding media equals the accelerating force. This is the basis of "electrophoresis" (*5*) used in many chemical and biological studies. Before any actual instrumentation could be developed to apply this technique to concrete,

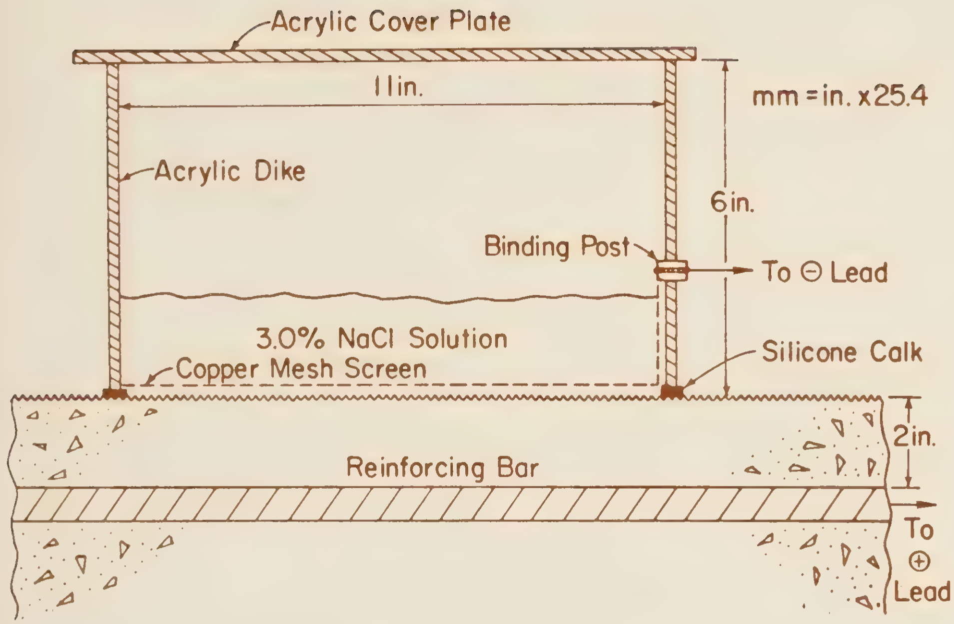


Figure 2.—Applied voltage apparatus used on slab specimens.

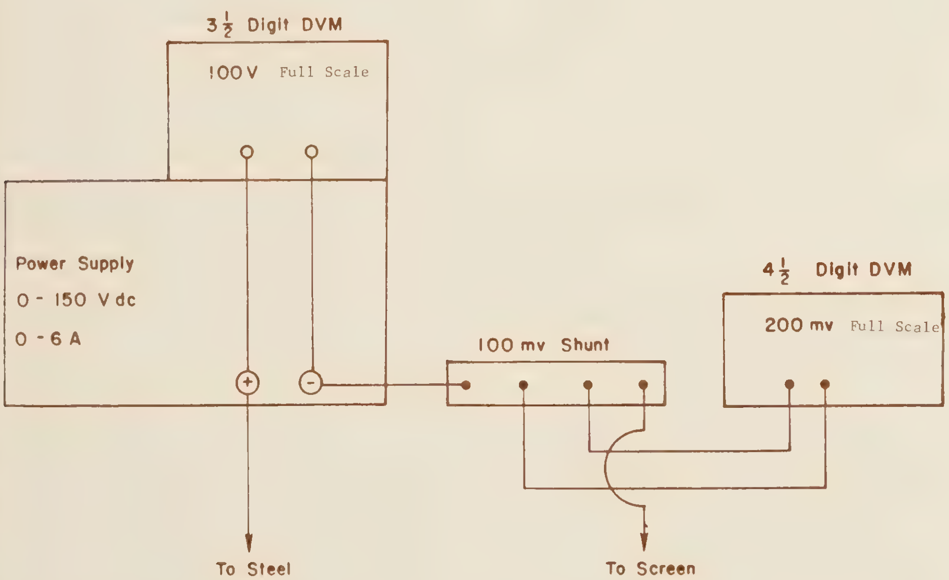


Figure 3.—Electrical block diagram of applied voltage apparatus.

however, the relationships between applied voltage, time, and amount of chloride ion flow into the various materials had to be evaluated in the laboratory. Once these relationships were established, the optimum set of conditions (voltage and time) could be chosen so a prototype could be constructed. In addition, preliminary laboratory studies had to indicate whether the concrete would need to be conditioned before actual testing.

Development of the technique

A drawing of the apparatus used in these preliminary studies is shown in figure 2. An acrylic dike was attached to the concrete surface with silicone caulking compound. The silicone was air-cured overnight, and the following morning 1.5 L (0.4 gal) of sodium chloride solution (30 percent sodium chloride by weight was used in initial tests) was poured into the dike. A No. 4 mesh copper screen was used as the negative electrode. The electronic components used are shown in figure 3. A d.c. power supply applies a constant voltage between the copper screen and steel reinforcing mat. The current flow is monitored by placing a digital voltmeter (DVM) across a 100 mv shunt in series with the copper screen. This provides a sensitivity of 1 ma/0.01 mv, and thus current in milliamps can be read directly from the DVM.

Conventional concrete slabs were prepared at both high and low water-cement ratios (0.35 and 0.60 by weight). Testing was conducted from 10 to 120 volts d.c. for 1 to 8 hours. Parameters evaluated included currents generated during the test, chloride content of the concrete slabs at various depths (up to 50 mm [2 in]) after the test, chloride content of the solution after the test, and temperatures generated inside the slabs during the test. The following conclusions were reached:

- A test period of 6 hours at 80 volts d.c. offers the optimum test condition for distinguishing between concretes having high and low chloride permeabilities.
- Up to 2.6 A of current are passed through a highly permeable concrete slab during the test period, compared with 1.8 A for a low permeability slab.
- After 6 hours of test at 80 volts d.c., approximately one-half as much chloride ion is detected at 19 mm (0.75 in) below the surface of the low permeability slab as is

detected at the same depth in the high permeability slab.

- Use of a more dilute chloride solution (3 percent) allows the drop in solution concentration to be used as a nondestructive measure of the amount of chloride that penetrates the slab.
- Temperature rise in the slabs during the test (greatest for the high permeability concrete) is significant. However, in full-scale bridge decks this was not thought to be a problem because the decks provide a much larger heat sink than do the laboratory specimens (0.09 m^2 [1 ft^2]).

Evaluation on range of concretes

The promising initial results led to an expanded test program on a variety of concrete slabs prepared by FHWA. These slabs were prepared from various concrete types and included both full-depth and overlay designs (table 1). Slabs were tested in the as-received condition, after moist storage, and after forced-air drying at 54° C (130° F). A 6-hour test at 80 volts d.c. applied to a 3 percent sodium

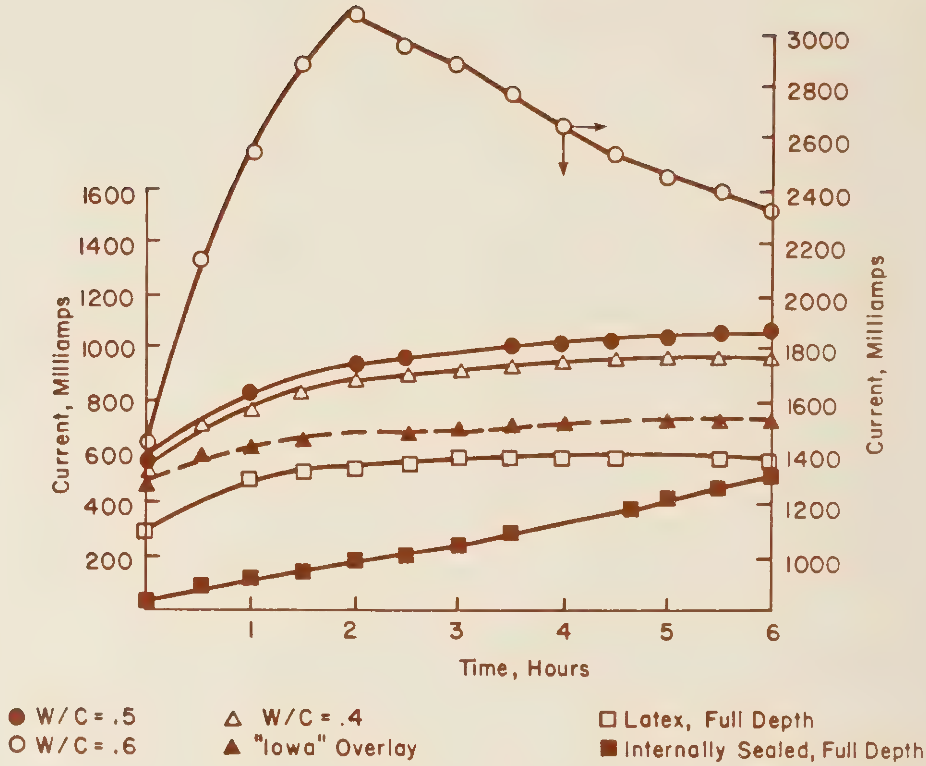


Figure 4.—Current versus time for various slabs tested at 80 volts d.c.

Table 1.—Concrete mix characteristics

Mix No.	Kind	Batch quantities: Cement	Water	Additive	Slump	Air	Overlay depth	Depth to steel mat
		lb/yd³	*lb/yd³*	*lb/yd³*	*inches*	*percent*	*inches*	*inches*
1	w/c =0.60	658	395	—	7.8	8.0	—	2.00
2	w/c =0.50	658	329	—	3.4	5.5	—	2.00
3	w/c =0.40	658	263	—	2.0	7.9	—	2.00
4	Latex-modified	658	158	Latex emulsion—198	7.2	7.7	—	2.00
5	Internally sealed	658	364	Wax beads—119	3.4	3.7	—	2.00
6	Polymer-impregnated concrete	658	329	PMMA 5 percent by weight of dry concrete	3.2	5.2	—	2.00
7	Latex-modified overlay	658	158	Latex emulsion—198	4.0	3.7	1.25	2.25
8	Internally sealed overlay	658	365	Wax beads—119	3.2	3.3	2.00	3.00
9	Iowa overlay	830	272	—	0.0	6.7	2.00	3.00
10	Polymer concrete overlay	19.9 percent "quick-deck" resin 0.4 percent accelerator 79.9 percent –8 sand			—	—	0.50	1.50

1 lb/yd³ =0.593 kg/m³
1 in =25.4 mm

chloride solution was used throughout this phase of the testing.

An example of the current flowing through the slabs during the test is shown in figure 4. In the high permeability concretes (w/c=0.6), there was a peak in the plots of current versus time, but for the other concretes current increased continuously with time at various rates. The magnitude of the current flow generally is proportional to the expected relative permeabilities of the various concretes. Additionally, the plots of current versus time were integrated to obtain the total amount of electric charge (in coulombs) passed through the slabs during the test. Likewise, after obtaining chloride analyses at 6.35 mm (0.25 in) increments after the test, the plots of chloride content versus depth were integrated to obtain a total slab chloride content. Finally, the solution remaining on the surface of the slab after the test was analyzed; the difference between the concentration of the remaining solution and the initial concentration was reported as chloride solution loss. Results for the as-received slabs are given in table 2.

The various concretes fell into four major groups. The first, conventional concretes, exhibited relatively high total chloride levels, solution losses, and charge passed. In the second group, latex-modified and Iowa concretes, values of total chloride, solution losses, and charge passed were about 20 to 30 percent lower than the "best" of the conventional concretes. The third group, internally sealed concretes, showed a dramatic drop in all parameters. When internally sealed concrete was not heat treated, its permeability was similar to that of conventional concretes. The final group, polymer-impregnated and polymer concretes, showed negligible permeability to chlorides.

Table 2.—Permeability characterization parameters

Description	Total chloride in concrete[1]	Chloride solution loss[2]	Charge passed	Chloride permeability
		percent	*coulombs*	
w/c=0.60	0.77	0.53	52,570	High
w/c=0.50	0.47	0.38	22,500	High
w/c=0.40	0.37	0.34	20,410	High
Latex overlay	0.37	0.38	16,950	Moderate
Latex full depth	0.27	0.18	8,670	Moderate
Iowa overlay	0.31	0.27	15,270	Moderate
Internally sealed full depth heated[3]	0.10	0.10	5,770	Low
Internally sealed full depth unheated	0.93	0.53	36,070	High
Internally sealed overlay heated[3]	0.09	0.03	3,020	Low
Internally sealed overlay unheated	0.47	0.28	22,418	High
Polymer impregnated	—	—	0	Very low
Polymer concrete overlay	—	—	0	Very low

[1] Total chloride in concrete=Integral of chloride profile (concentration versus depth); that is, area under percent chloride by weight of concrete versus depth curves.
[2] Solution loss=Drop from initial 1.8 percent chloride (3 percent NaCl) concentration.
[3] Slabs stored in moist room for 90 days before testing; all others tested at initial "as-constructed" conditions.

Correlation with 90-day ponding results

Currently, the only standardized test method for determining chloride permeability of concrete is that specified in American Association of State Highway and Transportation Officials test method T–259 80I (Resistance of Concrete to Chloride Ion Penetration). (*6*) In this method, a 3 percent sodium chloride solution is applied to the surface of a concrete slab continuously for 90 days, then the concrete is sampled at various depths and the chloride concentration of the samples is determined. This procedure was applied to companion specimens prepared with the specimens used in the evaluation of the rapid applied voltage test. Using linear regression techniques, results of the 90-day ponding were correlated with various combinations of the data sets shown in table 2.

The data indicated that the closest relationship was that between the charge passed (coulombs) in the rapid test and the total integral chloride in the 90-day test (a correlation coefficient of 0.92). Although the relationship was highly significant, the quantitative error estimate (percent standard error) was relatively high, about ±30 percent. Applying the 95 percent confidence limits to charge levels representative of high, moderate, low, and very low permeabilities, the data in table 3a were developed. For a test result of 30,000 coulombs passed, there is a 95 percent chance that the 90-day total integral chloride would be between 0.9 and 1.3 units. Table 3b indicates that this is indeed within the range of conventional concrete. For a test result of 15,000 coulombs, which we have selected as representing "moderately" permeable concrete, there is a 95

Table 3a.—Expected error in estimation of 90-day ponding results from charge passed data in applied voltage test

Charge passed	Permeability designation	Calculated 90-day total integral chloride in concrete[1]	95 percent confidence level	+ Total integral Cl^- in concrete	− Total integral Cl^- in concrete
coulombs					
30,000	High	1.08	0.19	1.27	0.89
15,000	Moderate	0.65	0.15	0.80	0.50
5,000	Low	0.37	0.19	0.56	0.18
0	Very low	0.24	0.22	0.46	0.02

[1] Total chloride in concrete=Integral of chloride profile (concentration versus depth); that is, area under percent chloride by weight of concrete versus depth curves.

Table 3b.—Comparison data—90-day ponding

Type of concrete	Total integral chloride in concrete—90-day ponding[1]
Conventional	1.0–1.5
Iowa, Latex	0.4–0.6
Internally sealed, PIC	0.1–0.4
PC	0.1

[1] Total chloride in concrete=Integral of chloride profile (concentration versus depth); that is, area under percent chloride by weight of concrete versus depth curves.

percent chance that the 90-day total integral chloride would be between 0.5 and 0.8 units. This is close to the range expected for Iowa and latex concretes. For low values of charge passed (5,000 coulombs), the confidence limits span the 0.18 to 0.56 range. This encompasses (but exceeds somewhat on the high side) the range expected for high quality internally sealed and polymer-impregnated concretes. However, for very low values of charge passed (such as for the polymer concretes), the upper confidence limit is high (0.46 units). This can be attributed to the fan shape of the confidence band at the extremities, where estimates based on the regression line are not advisable.

Preconditioning of specimens

When the applied voltage technique was tested on a set of slabs subjected to forced air-drying at 54° C (130° F), serious problems were encountered. The total chloride, solution loss, and charge passed were much lower than recorded on companion slabs tested in the as-received or moist conditions. It was obvious that drying reduced the moisture content of the concrete so much that the amount of current that could be passed through the slabs was limited by the abnormally high electrical resistance caused by the drying. A method was needed for preconditioning the test area to obtain a moisture content close to that of the original concrete.

The apparatus (fig. 2) was modified so that a vacuum of up to −98 kPa (−29 in Hg) could be drawn on the surface of the slab. Various resaturation schemes using alkali solutions, limewater, and heating were evaluated. The following approach was found to be most effective:

1. A vacuum is generated over the test area for 1 hour. Limewater is then let into the chamber, and the vacuum is maintained for another hour.

2. The vacuum is then broken and the limewater, still covering the concrete surface, is heated to 60° C (140° F) for approximately 18 hours.

3. The limewater is then removed, the 3 percent sodium chloride solution is poured into the chamber, and the test is continued as previously described.

This preconditioning procedure was incorporated into the design of the prototype and followed in all subsequent work.

Development and Evaluation of a Prototype Device

A device incorporating the features of the laboratory components into a single compact package suitable for both field and laboratory use was designed and constructed. The field instrumentation consists of three separate modules. The first (fig. 5) houses the electronics package (this also can be used in the laboratory), the second (fig. 6) allows for vacuum saturation before the test, and the third (fig. 7) applies voltage to the salt solution permeant. All instrumentation was designed for portability, and the entire field set (including auxiliary generator, tools, and chemicals) can be transported in a van-type vehicle. Safety features include overtemperature and overcurrent cutouts, time runout, and battery backup for data storage in case line power is lost.

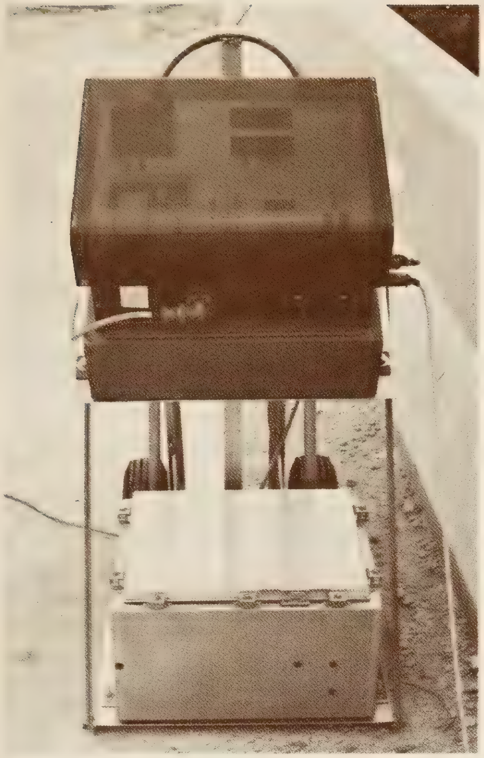


Figure 5.—Electronics module.

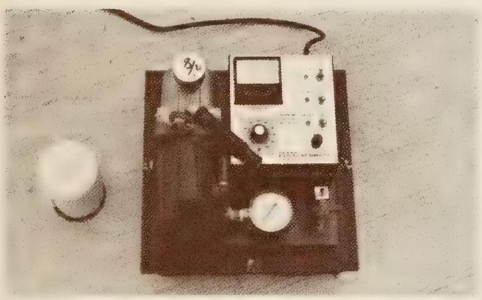


Figure 6.—Vacuum saturation module.

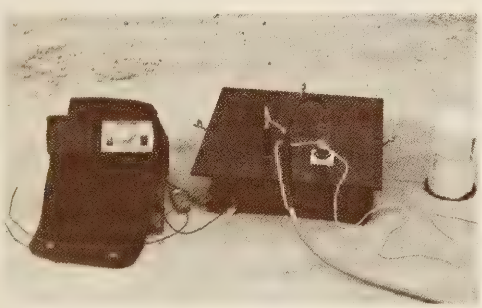


Figure 7.—Applied voltage module.

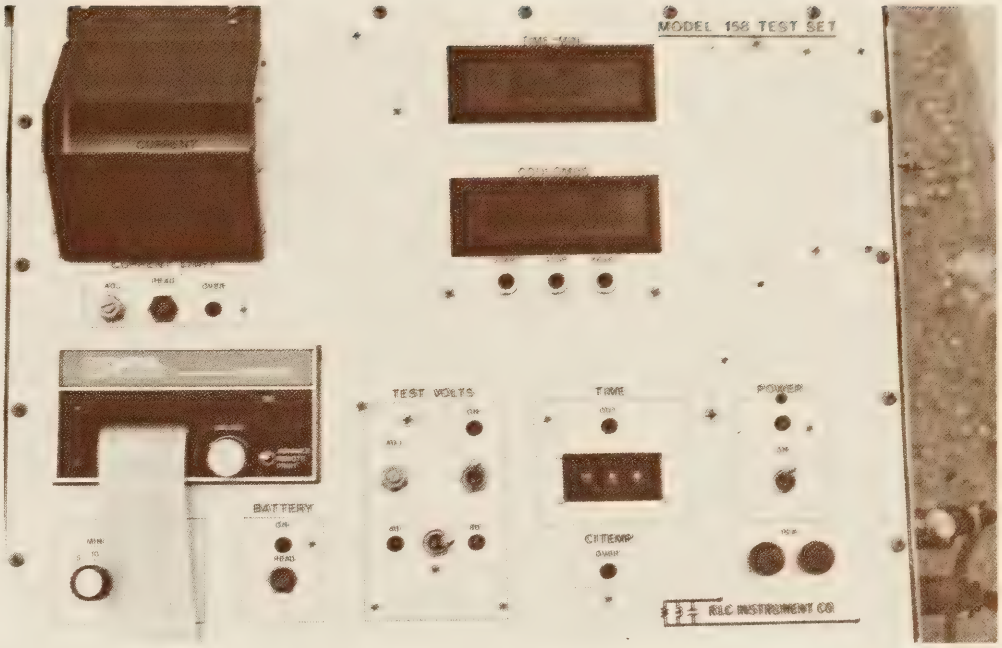


Figure 8.—Control panel (electronics module).

Details of construction

The complete electronics module consists of two weatherproof enclosures mounted on a lightweight pneumatic-tired aluminum dolly. Overall dimensions are 1 245 × 737 × 610 mm (49 × 29 × 24 in) and total weight is 75 kg (166 lb). The lower enclosure houses the power supplies that can supply up to 7 A at 80 volts d.c. A cooling fan prevents overheating of the unit. The upper enclosure houses all electronics, operating controls, and the control panel (fig. 8). The elapsed time and coulomb meters are light emitting diodes (LED's) constructed to be visible even in strong sunlight. Because the digital panel meters, used for the current and voltage meters, have LED's not visible in sunlight, a sunshield was constructed. A rainshield can be placed over the entire panel, as shown in figure 5.

The vacuum saturation module uses a compact single-stage vacuum pump mounted onto a 6.35 mm (0.25 in) thick anodized aluminum plate along with a dial vacuum gage. Also included are a thermocouple-based temperature controller, a resistance heater, a pyrometer, a magnetically coupled stirrer, and a fluid entry valve. The dike is constructed from 6.35 mm (0.25 in) thick anodized aluminum plate having a Teflon coating. Wing nuts tightly seal the dike to the vacuum saturation cover. The dike contains both the limewater solution used in the saturation phase and the chloride solution used during the actual test.

The screen assembly used to apply voltage to the saline solution is fabricated from No. 4 mesh stainless steel fabric reinforced along its outer edge with a 3.3 mm (⅛ in) stainless steel rod. The screen is bolted to Teflon standoffs and sits approximately 1 mm (0.04 in) above the concrete surface during the test. The negative electrode is a 12.7 mm (½ in) stainless steel rod bolted to the screen at one end; a female "banana" jack is fastened to the other end. A high limit thermostatic switch is housed in a 15.9 mm (⅝ in) diameter stainless steel well that extends into the chloride solution and terminates on the cover in an amphenol connector. The electronics module and vacuum saturation/heating module are powered by a portable 3.5 kW gas generator. An auxiliary 22.7 L (6 gal) gas tank allows the unit to run unattended for at least 14 hours.

Test procedure

The field test procedure can be separated into four stages—location of reinforcing steel and bonding of test dike, vacuum saturation and heating of test area, applied voltage test, and chloride sampling, which is optional.

Each test takes 30 hours; using two dikes, four tests can be completed in one work week. The second stage requires overnight operations; however, the unit may be left unattended during this period. A condensed version of the full test procedure follows.

- The topmost steel within the test area to be covered by the dike is located and outlined on the surface. The dike is placed symmetrically over the area, with one bar running through the center of the area. The dike is bonded to the concrete with a silicone caulk, which is then allowed to cure for 4 hours.

- A hole is drilled to the top steel about 127 mm (5 in) from the dike, and a pin is pressure fitted into the rebar.

- The vacuum lid is attached to the dike and a vacuum of −94.5 to −101 kPa (−28 to −30 in Hg) is pulled for 1 hour. Limewater is then let into the chamber and the vacuum is continued for an additional hour. The vacuum is then broken and the heater is activated to 60° C (140° F).

- The following morning the limewater is removed and 1.5 L (0.4 gal) of 3 percent NaCl is placed into the dike. The applied voltage screen is placed on the dike and the proper connections are made to the electronics module. The module is set for 80 volts and a test time of 6 hours. The STOP and RESET buttons are depressed to zero all counters, then, the START button is depressed to activate the instrument. Elapsed time, current, and coulombs are printed automatically at 30-minute intervals. The instrument shuts down after 6 hours.

- *Optional Stage*—After the test, a 200 mL (0.053 gal) sample of the NaCl solution is taken. The remaining solution is discarded and the test area is flushed with freshwater. The test area is then surface-dried and duplicate samples for chloride are taken directly above the central rebar. FHWA guidelines for chloride sampling are followed (*7*), except that samples are taken at 6.35 mm (0.25 in) increments up to 25.4 mm (1 in), and a final 12.7 mm (0.50 in) increment up to 38.1 mm (1.50 in) for five samples per drill hole. The samples are returned to the laboratory, dried to constant weight, and analyzed for total chloride with a Gran plot titration. (*7*)

Effects of various test variables

Before actual field testing, the prototype had to be tested on slabs intermediate in size between the small specimens tested in the earlier phases of the project and full-scale bridge decks, to allow a more realistic heat sink and also to allow variables to be evaluated within a single slab. The variables that needed to be studied in more detail were: Concrete cover, ambient test temperature, geometry of reinforcing cages, and presence of chloride in concrete before the test.

All testing was done on 1 220 × 1 525 × 180 mm (4 ft × 5 ft × 7 in) slabs having reinforcing mat details typical of those used in reinforced concrete bridge deck slabs. One slab was cast with concrete containing 3 kg/m^3 (5 lb/yd^3) of chloride ion. The results of these tests can be summarized as follows:

- The amount of concrete cover affects test results; for example, if a 50 mm (2 in) cover is assumed for a test, results may vary as much as 25 percent if the actual cover varies ±25 mm (1 in) from that value.

- Reinforcing cage geometry has no significant effect within the limits of spacing typically found in plain reinforced decks.

- Charge passed (in coulombs) during the test increases with an increase in ambient temperature. Between 4° C (40° F) and 21° C (70° F) there is an increase of approximately 540 coulombs/°C (300 coulombs/°F) and between 21° C (70° F) and 38° C (100° F) there is an increase of approximately 162 coulombs/°C (90 coulombs/° F).

- Charge passed during the test is not affected if up to 3 kg/m^3 (5 lb/yd^3) of chloride ion is present in the concrete.

- No spalling or other obvious signs of concrete distress were observed in any of the tests, although the reinforcement was slightly corroded in the slab deliberately prepared with a large amount of chloride ion.

Field Trials

Field trials of the prototype instrument were conducted on two bridge decks in Wisconsin. At both sites construction was ongoing, thus providing "worst case" conditions for ruggedness testing. The test results were interpreted to find the permeability of the test sections, but the relatively unquantified influences, such as variable overlay depths, limited the validity of these findings. Finally, cores were obtained from the two bridges and one additional structure to evaluate in the laboratory using the applied voltage cell.

The first structure chosen was a four-span conventional poured concrete deck on newly constructed I-43. No deicing salt had been applied to this deck. The concrete was designed to have a 390 kg/m^3 (658 lb/yd^3) cement content, 27.6 MPa (4,000 psi) minimum compressive strength, and 5 to 7 percent air content. Calculated water-cement ratio was 0.45.

Tests were conducted at four locations—one location on each of

Table 4.—Field test results—conventional deck, I-43, Wisconsin

Location	Cover[1]	Air temperature range	Charge passed	Solution loss[2]	Total integral chloride in concrete[3 4]
	inches	*°Fahrenheit*	*coulombs*	*percent Cl⁻*	
1	2 ¼	72–87	31,990	0.30	0.66
2	2 ½	61–68	26,680	0.25	—
3	2 ¾	66–71	22,672	0.22	0.33
4	2 ⅜	54–69	33,646	0.35	—

[1] Solution loss=Drop from initial 1.8 percent chloride (3 percent NaCl) concentration.
[2] Measured approximately 127 mm (5 in) outside actual test area on transverse bar.
[3] Corrected for baseline values.
[4] Total chloride in concrete=Integral of chloride profile (concentration versus depth); that is, area under percent chloride by weight of concrete versus depth curves.

1 in=25.4 mm
°F=1.8°C + 32

Table 5.—Field test results—dense concrete overlay, I-94, Wisconsin

Location	Cover[1]	Air temperature range	Charge passed	Solution loss[2]	Total integral chloride in concrete[3 4]
	inches	*°Fahrenheit*	*coulombs*	*percent Cl⁻*	
1	4 ½	65–88	17,690	0.29	0.27
2	4 ⅛	67–79	9,250	0.10	0.22
3	3 ⅞	73–82	13,390	0.24	0.27
4	4 ¼	67–79	12,190	0.17	0.33
Average	4 3/16	N/A	13,130	0.20	0.27

[1] Solution loss=Drop from initial 1.8 percent chloride (3 percent NaCl) concentration.
[2] Measured approximately 127 mm (5 in) outside actual test area on transverse bar.
[3] Corrected for baseline values.
[4] Total chloride in concrete=Integral of chloride profile (concentration versus depth); that is, area under percent chloride by weight of concrete versus depth curves.

1 in=25.4 mm
°F=1.8°C + 32

the four spans—on the 3.05 m (10 ft) concrete shoulder. Cores were taken 0.3 m (1 ft) from each test area. Results of the field testing are shown in table 4. Values for charge passed and solution loss fall between previous laboratory values for concretes having water-cement ratios of 0.4 to 0.5. Although ambient temperature appeared to have little significant effect, the effect of clear cover was substantial. The field results appeared more sensitive to concrete cover than those from previous laboratory tests. The results did show that at all covers the results for the concrete bridge deck with a design water-cement ratio of 0.45 fell between the laboratory results for water-cement ratios of 0.4 and 0.5.

Analyses of "blank" chloride powder samples taken before testing at locations 1 and 3 indicated baseline chloride contents of 0.055 and 0.078 percent, respectively. This could explain the erratic results for total integral chloride obtained by integrating the chloride profiles and then subtracting the baseline values. Because this deck had not been salted, the chloride probably was from chloride bearing materials used in the concrete mixture. These results indicated that chloride sampling after the test is inappropriate where large amounts (greater than 0.01 percent) of chloride are detected in the blank samples.

The second bridge was a three-span conventional poured concrete deck on I-94. The initial structure was completed in 1965, but a dense concrete overlay had been applied to the deck in June 1980 as part of a highway rehabilitation program. No deicing salt had been applied to the overlay, and chloride contents had not been determined on the original deck. The overlay was a dense concrete mixture designed to have a 488 kg/m^3 (823 lb/yd^3) cement content and a 5 to 6 percent air content, with a calculated water-cement ratio of 0.25. The overlay was placed at a slump of 13 to 19 mm (0.50 to 0.75 in) using a vibrating screed.

Tests were conducted at four locations on the 1.68 m (5.5 ft) wide shoulder (table 5). Solution loss values correlate well with charge passed. Chloride values, however, showed poor correlation with charge passed. The cause of the high charge and solution loss values at location 1 was not readily apparent. Maximum air temperature was somewhat higher for this location, but cover for location 1 was *greater* than for any of the other locations. Additionally, a core taken 0.3 m (1 ft) from the test section showed an overlay thickness of 64 mm (2.5 in), which was greater than that of the other test sections. Results obtained on laboratory testing of cores helped explain some of these discrepancies.

Laboratory Testing of Core Specimens

Although the field test apparatus is rugged, accurate, and reliable, two major drawbacks limit both its practicality and the significance of the data obtained on field structures:

- A series of four tests requires that one lane of traffic be closed for 5 days. This is very expensive considering the amount of traffic control required. On new construction not yet opened to traffic, however, this would be no problem.
- Correction factors for depth of cover and ambient temperature are available only for conventional concrete. These corrections may differ for the other special kinds of concrete. In the present study the other kinds of concrete were tested at only one fixed cover and temperature, so field application will be limited until these data have been developed. This also applies to overlays, where various combinations of overlay/substrate thicknesses can occur; only one combination has been evaluated in the laboratory.

Because of these limitations on the field device, a laboratory version of the apparatus was developed to test accurately a variety of materials. The device, an applied voltage cell, is shown in figure 9. The cell is machined out of solid acrylic plastic; it provides two reservoirs of approximately 200 mL (0.053 gal) capacity that are fitted onto the opposite faces of a 102 mm (4 in) diameter core using a rapid-setting silicone compound. One reservoir is filled with 0.3M (1.2 percent) NaOH, the other with 3 percent NaCl. The core specimen (usually 50 mm [2 in] thick) is coated on its side with epoxy and vacuum saturated for 18 hours before testing. Photographs of the cell and a specimen ready for testing are shown in figure 10.

The equipment and procedures are similar to that used for the field test, except that the applied voltage cell is used in place of the dike. Appropriate adaptors convert the field cable connectors into "banana" jack connections to the cell. The electronics module is used the same way as in the field, except that the voltage is set at 60 volts d.c. rather than 80 volts d.c.

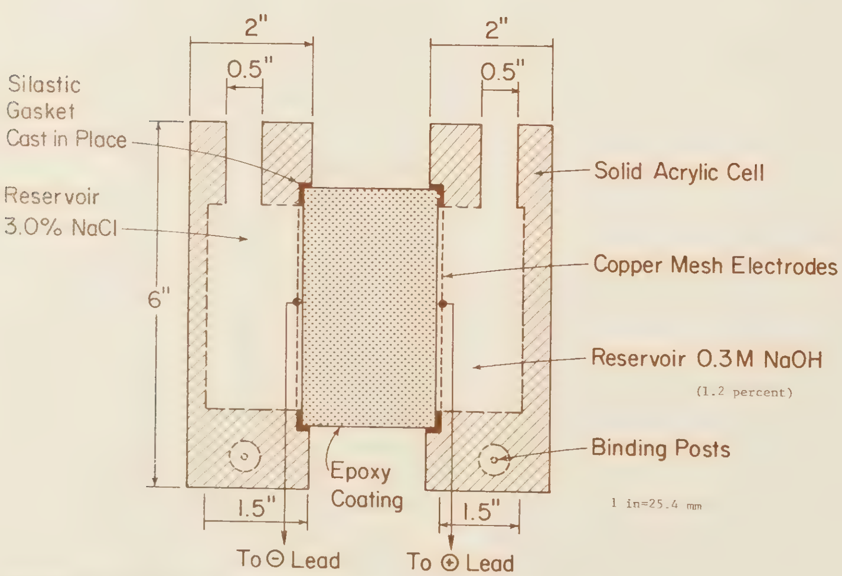
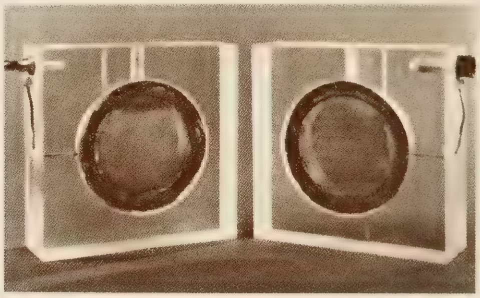
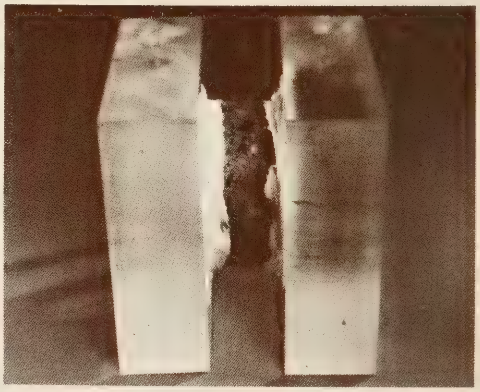


Figure 9.—Drawing of applied voltage cell for core specimens.

Figure 10.—The cell and specimen ready for test.



(a) Applied voltage cell, disassembled.



(b) Applied voltage cell, assembled with specimen.

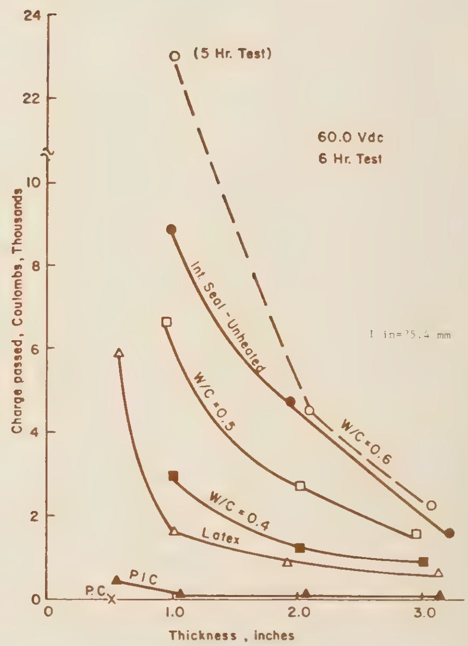


Figure 11.—Effect of core thickness on charge passed.

The cores extracted from the slabs supplied by FHWA were examined thoroughly, and the effect of core thickness on charge passed is shown in figure 11. Based on this work, a standard thickness of 50 mm (2 in) was selected to optimize test sensitivity yet minimize the heating effects seen when testing thinner specimens. An interpretation of results for 50 mm (2 in) thick specimens is given in table 6.

The cores extracted from the two Wisconsin decks plus four cores taken from a polymer-concrete overlay in Oregon were tested with the applied voltage cell. Averages for each deck are given in table 7. Referring these averages to table 6 indicates that the conventional concrete is moderately permeable. This conforms to the calculated water-cement ratio of 0.45. The dense concrete overlay would be classified as having low permeability, although the value of charge passed (1,770 coulombs) is quantitatively closer to what might be expected from a water-cement ratio of 0.4 (good quality conventional concrete) than from the water-cement ratio of 0.25 obtained from the job records for this mix. Finally, the polymer concrete overlay exhibited zero charge passed, confirming its dielectric nature.

Table 6.—Applied voltage cell test—interpretation of results

Chloride permeability	Charge passed	Kind of concrete
	coulombs	
High	4,000	High water-cement ratios (0.6 or greater)
Moderate	2,000–4,000	Moderate water-cement ratios (0.4 to 0.5)
Low	1,000–2,000	Low water-cement ratios
		"Iowa" dense concrete
Very low	100–1,000	Latex-modified concrete
		Internally sealed concrete
Negligible	100	Polymer-impregnated concrete
		Polymer concrete

Table 7.—Applied voltage cell tests, field cores

Material	Charge passed	Solution loss[1]
	coulombs	*percent chloride*
Conventional concrete	2,830	0.30
Dense concrete overlay	1,770	0.10
Polymer concrete overlay	0	—

[1] Solution loss=Drop from initial 1.8 percent chloride (3 percent NaCl) concentration.

Conclusions and Recommendations

The applied voltage chloride permeability device is a versatile instrument that can be used both for inplace and laboratory testing. A prototype package is available for testing bridge decks and other horizontal surfaces. For laboratory testing, either the prototype or standard electronic components can be used to test cores in a specially constructed applied voltage cell.

Currently, implementation of the field device is limited by a lack of background data on the effects of clear cover and temperature on the test when applied to materials other than conventional portland cement concrete. In addition, the effects of varying overlay thickness and combined overlay/substrate covers need to be quantified.

The laboratory cell test offers the most reliable alternative to field testing. This requires that 102 mm (4 in) cores be extracted from the structure, sectioned, and vacuum saturated before testing. In this test the effects of cover and temperature are eliminated, allowing a more accurate assessment of the permeability. Because the test equipment can be set up easily from off-the-shelf components, it is recommended that highway agencies and other interested parties apply this technique to materials currently being evaluated in bridge deck construction and maintenance programs.

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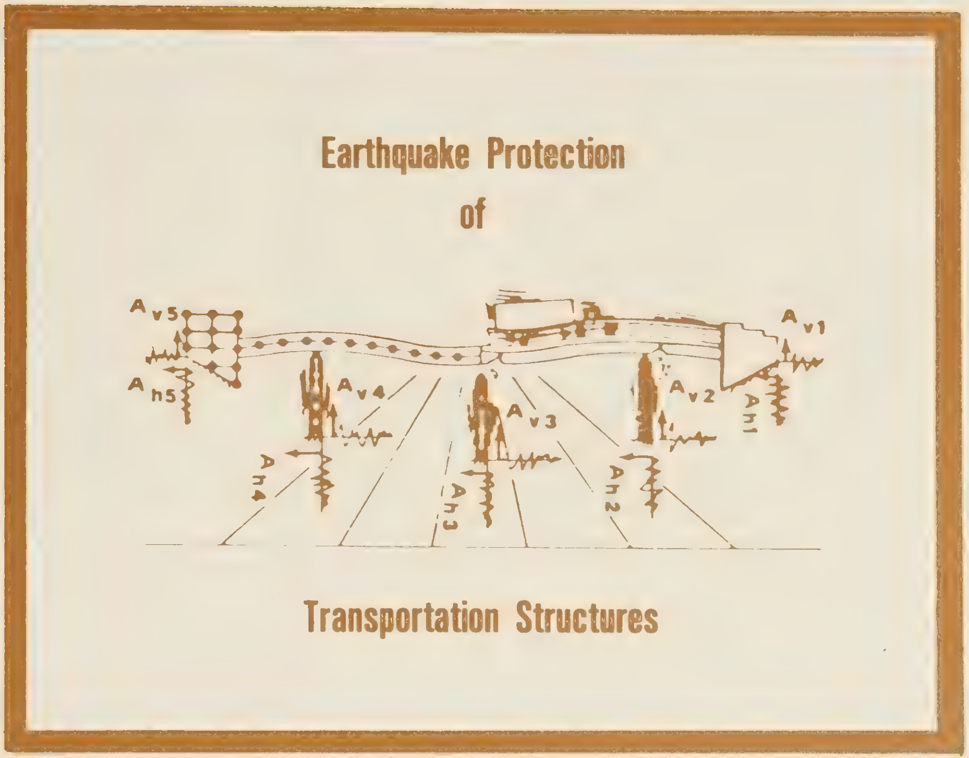

Mitigation of Earthquake Damage on Eastern Highway Systems

by
James D. Cooper

This article reviews highway bridge damage that would probably occur if there were a major earthquake in the Eastern United States. Past earthquake-induced failures to the highway network are reviewed to show the vulnerable components of the highway system. The damage documented in this article occurred in regions generally associated with significant earthquake activity; however, these kinds of damage would cause appreciable disruption to the Eastern highway system. Practical retrofit concepts are presented as well as new design techniques that can enhance the seismic performance of structures and mitigate earthquake effects on bridges located in less seismically active areas of the United States. Major obstructions in the implementation of existing technology in the United States is discussed along with research recommended to improve earthquake-resistant highway bridge design and construction.

Introduction

Earthquake engineering has been researched to a large extent in the past three decades. Structural analyses, designs, building codes, and specifications have been refined and updated to provide the engineer with the necessary tools to design and construct modern buildings to resist the forces developed during strong earthquakes. Unfortunately, these tools have not been modified or adapted for use outside regions of moderate to high seismic activity. Although research was being conducted to better understand building performance, little research to insure the satisfactory seismic performance of highway structures had been conducted in the United States before the 1971 San Fernando, Calif., earthquake. Also, little earthquake research had been addressed at improving the performance of the roadway system itself because of the great redundancy in the highway network and the relative ease at which the roadway system can be repaired.

Since 1971, however, research has been implemented on the west coast and in Alaska where highway bridges are vulnerable to seismic damage. To date, seismic design considerations for virtually all bridges east of the Rocky Mountains have been neglected. A comprehensive study has been conducted to develop design and construction guidelines for new and replacement bridges to resist the effects of earthquake motions. The guidelines specifically address structures in regions where there is a low probability of seismic occurrence yet significant potential exists for catastrophic damage.

Vulnerable Details and Modes of Failure

After earthquakes that cause structural damage and failure occur, sites are investigated to gain insight into failure modes of roads and bridges. Rational design procedures can be developed to insure the structural integrity of the highway system during periods of ground motion by adequately documenting such failures. Documentation of bridge damage and failures in the 1964 Alaska, 1971 San Fernando, 1976 Guatemala, and 1978 Japan earthquakes (*1–3*)[1] has pointed to critical weaknesses in bridge supports and substructures during strong seismic activity.

Roads

Generally, roadway systems have been damaged and subsequently closed from earthquake-induced embankment and slope failures and roadway cracking. Roadways on soft grounds or embankments have suffered severe cracking, settlement, and sloughing. Eastern roadways no doubt will suffer this kind of damage during earthquake activity. However, it does not appear economical to design and construct earthquake-resistant roadway systems that can survive an event without damage. In most cases, because of system redundancy, traffic can be rerouted quickly with only minor inconvenience to the user.

Bridges

Major damage has occurred to highway bridges subsequently requiring costly repair or replacement and disrupting highway use for long periods. Typical bridge damage includes collapsed spans, loss of bearing support, abutment and column or pier movement, joint movement, and column and foundation failure. Eastern bridges are particularly vulnerable to these kinds of damage.

[1] Italic numbers in parentheses identify references on page 123.

Alaska—1964

The primary damage to Alaskan bridges is attributed to span shortening, horizontal pier movement, and relative vertical movement between piers and abutments. Abutment fills settled, and in some cases, bridge abutments moved together toward the center of a stream causing expansion joint collapse and severe superstructure shortening. Ten bridges totally collapsed, 26 experienced partial collapse and were beyond repair, and 60 others experienced some form of repairable damage.

Soil liquefaction, evident wherever severe damage occurred, played a major role in foundation displacement and bridge damage. Liquefaction is thought to have occurred during seismic loading, thus accounting for most of the pier movement at river crossings. Damage to many bridge piers was caused when piers and pier caps displaced longitudinally to the point where spans fell off bearing supports. Longitudinal pier tilting, caused when foundations moved toward the streambed while pier tops were restrained by decks, is shown in figure 1. Bearing devices were severely damaged as shown in figure 2.

Those bridges founded on piles that were driven through sands and silts experienced severe damage, those bridges having foundations supported directly on bedrock suffered little damage, and those bridges founded on piles in gravelly sands and gravel performed relatively well.

Numerous river crossings that are vulnerable to this kind of damage exist in the Eastern United States.

Figure 1.—Pier or column tilting.

Figure 2.—Bearing damage.

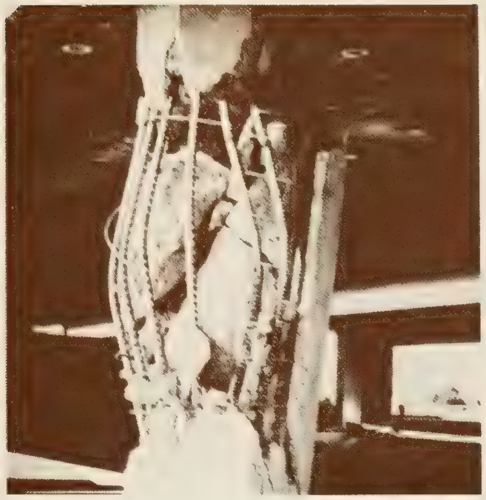

Figure 3.—Failed concrete column core.

Figure 4.—Column/foundation connection failure.

San Fernando—1971

Several major bridges collapsed during the San Fernando earthquake. These spectacular collapses highlighted the weaknesses in the design detail of columns and connections. A unique feature of the earthquake was the significant amount of vertical acceleration, averaging approximately two-thirds that of the horizontal. Seven bridges either totally collapsed or were so badly damaged that they had to be replaced. Sixty additional structures experienced moderate to extensive, but repairable, damage.

The enormity of the vertical component of force is shown in figure 3 where the reinforced concrete column burst in the middle as though it had been failed in a huge testing machine. This kind of failure makes a strong case for providing more substantial spiral ties that can retain the crushed concrete and provide continued support to the structure.

The integrity of footings was severely jeopardized by the kind of forces produced by this earthquake. Figure 4 illustrates what can happen to the connection between the base of a column and the cap of a pile footing if the connection is not adequately designed. The grinding back and forth and the pounding completely reduced the pile cap to nothing more than rock, dust, and rubble. Main column reinforcement, typically number 18 bars, pulled out completely because of inadequate connection details. Because of this kind of failure, additional top steel is now added to the footing and tied into the column. Anchorage details have since been improved.

Although the East may not experience a San Fernando type of earthquake, improved connection and concrete confinement details can reduce significantly, at little additional cost, the potential for catastrophic failure.

Guatemala—1976

The Guatemala earthquake was of particular interest because it vividly demonstrated the importance of using hinge restraining devices to tie the superstructure together. The superstructure of one major bridge collapsed and that of another rotated, in plan, and fell off its bearings. A third structure that had restrainer bars designed into the superstructure suffered no appreciable damage. Eleven other bridges suffered varying degrees of repairable damage. Typical damage included lateral displacement of the superstructure, cracked abutments caused by deck impact, and tipped or fallen rocker bearings.

The most severely damaged bridge was a five-span, simply supported steel plate girder structure. The earthquake caused the three center spans of the superstructure to fall off the bearing supports (fig. 5). The abutments and column bents were undamaged with the exception of extremely minor hairline cracks around the base of one of the columns. The bridge collapsed principally because of the bearing details used and the lack of longitudinal and transverse superstructure restraint at the supports. Figure 5 shows the fixed bearings in place atop the free standing piers. An expansion rocker can be seen resting on the tie beam in the foreground. The north girder appears to have walked laterally approximately 127 mm (5 in) on the expansion bearing.

Figure 5.—Collapsed spans.

Figure 6.—Tipped bearings.

Figure 7.—Undamaged bridge with joint restraining devices provided.

Figure 6 shows the north girder fixed bearing detail at the west abutment. The bearing support is inclined approximately 8 degrees. The anchor bolt is extended and there is space between the abutment bearing ledge and bearing plate. If appropriate hinge restrainers had been in place (thus tying the structure together), the bridge probably would not have collapsed.

The bridge that performed so well (fig. 7) was constructed in 1972. The bridge is of interest because it incorporates seismic hinge and abutment restrainers, and although minor damage occurred, the bridge remained inservice following the earthquake. The performance of the eight-span concrete bridge indicated that incorporating new seismic design techniques in new construction can provide adequate structural resistance against major earthquakes. The generally good performance of the bridge is attributed to the use of seismic restraint mechanisms, for example, hinge restrainers that tied the prestressed I girders together (fig. 8). The only restrainer failure occurred across the west abutment where all 12 cables failed. The failure resulted in the walking of bearing pads at the abutment.

Japan—1978

Approximately two dozen bridges were damaged significantly during this earthquake. Massive reinforced concrete piers were badly cracked and damaged. They responded essentially as nonductile, rigid bodies to the earthquake motion. In spite of the major damage to these elements, the designs proved satisfactory because the bridges did not collapse but remained operational for emergency use. Although this kind of construction proved successful, it is too expensive for use in the United States where the philosophy is to use more ductile, energy-absorbing designs. Pier tilting and abutment movement were evident at several sites and probably caused extensive damage to bearing devices. This kind of movement is exceptionally difficult to control and indicates the need to perform indepth geotechnical investigations, particularly in areas with high water tables.

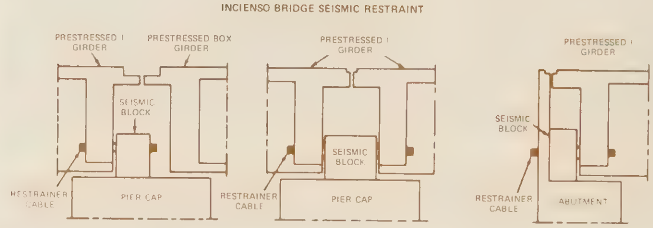

Figure 8.—Joint restraining devices.

The most spectacular damage occurred to a 574 m (1,885 ft) 15-span structure constructed in 1956. It suffered major damage to the superstructure, bearings, and abutment. One drop-in plate girder span fell off its bearings and dropped to the ground (fig. 9). The girders at the adjacent pier displaced longitudinally 533 mm (21 in), allowing the unrestrained suspended span to drop off the 457 mm (18 in) hinge seats (fig. 10). Extensive bearing and abutment damage emphasized the extensive relative motion that occurred between the super- and substructure (fig. 11).

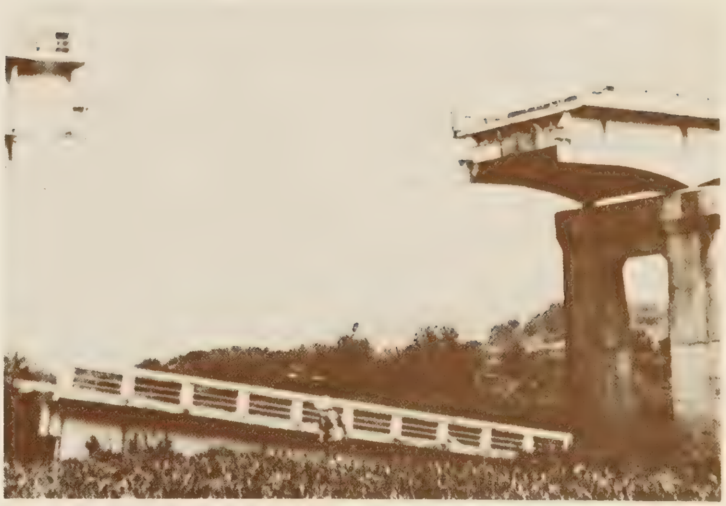

Figure 9.—Collapsed span.

Lessons learned

Damage to bridges from past earthquakes reinforces the need to consider an aggressive program to strengthen those structures designed and built under older seismic design criteria. The most vulnerable components requiring attention are bearings and columns, including connections into the foundation. Modest investments can significantly reduce future damage and help to avoid collapse of older, major structures. Clearly, Eastern structures should be evaluated to determine their importance to the areas they serve. Post-earthquake investigations have shown that the kinds of bridges in the East are extremely vulnerable to seismically induced ground motion.

Figure 10.—Longitudinally displaced girder.

Improved Highway System Response

Two approaches can be taken to improve the seismic resistance of highway networks. The first approach to upgrading seismic resistance is time consuming but economically feasible and is being pursued. Design guidelines can be upgraded as more knowledge is gained about the response of specialized transportation structures to seismic activity. New guidelines can be applied as older bridges are removed from service because they are either structurally unsound or functionally obsolete.

The second approach includes the strengthening or retrofitting of existing bridges that are important to the network and are susceptible to damage or collapse in the event of an earthquake. This approach may prove costly and consequently be economically infeasible.

What is required is a balanced approach to strengthen the highway system against seismic attack. This can be accomplished by retrofitting those structures that form critically vital links in the network and are exceptionally

Figure 11.—Abutment movement and damage.

vulnerable to damage while at the same time imposing new seismic design guidelines that are appropriate for various geographic regions on bridges being replaced.

Following is a discussion on the background, development, and application of newly developed seismic design guidelines for highway bridges (*4*) and a discussion on currently available retrofitting techniques for enhancing the seismic resistance of existing structures.

New seismic design guidelines

Before the 1971 San Fernando earthquake, the American Association of State Highway and Transportation Officials (AASHTO) specifications for the seismic design of bridges were based in part on the lateral force requirements for buildings developed by the Structural Engineers Association of California. (*5*) In 1973 the California Department of Transportation (CalTrans) introduced new seismic design criteria for bridges that included the relationship of the site to active faults, the seismic response of the soils at the site, and the dynamic response characteristics of the bridge. (*6*) In 1975 AASHTO adopted Interim Specifications, which were a slightly modified version of the 1973 CalTrans provisions, and made them applicable to all regions of the United States.

In addition to these code changes, the 1971 San Fernando earthquake also stimulated research activity on seismic problems related to bridges. By 1977, significant earthquake engineering research studies relating to highway bridges had been completed and the Federal Highway Administration (FHWA) contracted a study to develop recommended bridge seismic design guidelines and evaluate their impact on design, construction, and cost.

A basic premise in developing the bridge seismic design guidelines was that they apply to all parts of the United States because the seismic risk varies across the country. Therefore, design guidelines were developed for four bridge seismic performance categories based on the importance classification of the bridge and the seismicity of the area in which the bridge is located.

Bridges are classified according to their relative importance. Essential bridges, determined by their social/survival and security/defense classification, must function during and after an earthquake. Seismic Performance Category (SPC), another aspect of classification, covers the differing degrees of complexity and sophistication of seismic analysis and design that are required. SPC–D bridges include those designed for the highest level of seismic performance, with particular attention to methods of analysis, design, and quality assurance. SPC–C bridges include those where a slightly lower level of seismic performance is required but the potential damage is slightly greater than SPC–D. SPC–B bridges include those where a lesser level of seismic performance is required and a minimum level of analysis and specific attention to support design details are provided. SPC–A bridges include those where no seismic analysis is required, but attention to certain design details for superstructure support is provided. Most Eastern bridges fall into SPC–A and SPC–B classifications.

In assessing bridge failures of past earthquakes in Alaska, California, and Japan many of the "loss of span" failures are attributed partly to relative displacement effects. Relative displacements are caused by out-of-phase motion of different parts of a bridge, lateral displacement and rotation of the foundations, and differential displacements of abutments. Therefore, in developing the new seismic design guidelines, the design displacements were considered to be as important as design forces. For higher performance bridges, requirements for ties between noncontinuous segments of a bridge are specified in addition to minimum bearing support lengths at abutments, columns, and hinge seats.

The methodology used in the guidelines addresses the relative displacement problem. (*7, 8*) The methodology varies in complexity as the SPC increases from A to D.

Minimum requirements are specified for bearing support lengths of girders at abutments, columns, and hinge seats to account for some of the important relative displacement effects that cannot be calculated by state-of-the-art methods. A somewhat similar requirement is included in the latest Japanese bridge code. Member design forces are calculated to account for the directional uncertainty of earthquake motions and the simultaneous occurrence of two perpendicular horizontal earthquake forces. Also, design requirements and forces for foundations are intended to minimize damage because most damage that might occur will not be readily detectable.

For SPC–A bridges the only design requirement is to provide minimal bearing support lengths for girders at abutments, columns, and expansion joints. Even though the level of seismic risk of these bridges is very small, superstructure collapse must be prevented and hence the requirements. Design for the level of seismic forces in these regions is not necessary.

Elastic member forces for SPC–B bridges are determined by a single mode spectral approach. Design forces for each component are obtained by dividing the elastic forces by a reduction factor (R). For connections at abutments, columns, and expansion joints, the R-factor is either 0.8 or 1.0 and therefore they are designed for the expected or greater than expected elastic forces. Foundations are also designed for the elastic forces. For columns and piers the R-factor varies between 2 and 6 and therefore they are designed for forces lower than that expected from an elastic analysis and are assumed to yield when subjected to the forces of the design earthquake. Design requirements to insure reasonable ductile capacity of columns in SPC–B are not specified, whereas they are for bridges in higher seismic performance categories.

For SPC–C and SPC–D bridges the general approach is similar to SPC–B; however, several additional requirements are included. For columns, additional requirements are included to insure that they are capable of developing reasonable ductile capacity. For connections and foundations, alternate design forces to those determined by the procedures of SPC–B are also permitted. These are based on the maximum shears and moments that can be developed by column yielding when the bridge is subjected to the design earthquake forces. Horizontal linkage and tie-down requirements at connections also are provided. For SPC–D bridges, settlement slabs are required to reduce the chance of abutment backfill settlement.

Ground motion intensities to be used with the seismic design provisions were selected carefully. Seismic risk maps and associated design spectra were developed for the "Tentative Provisions for the Development of Seismic Regulations for Buildings" (ATC–3–06). (*9*) The ATC–3–06 maps are based on an appraisal of expected ground motion intensities. The probability that the design ground shaking will be exceeded is approximately the same in all parts of the United States and includes the frequency of occurrence of earthquakes in various regions of the United States. It is possible that the design earthquake ground shaking might be exceeded, although the probability is small.

Ground motion is characterized by two parameters—Effective Peak Acceleration (EPA), A_a, and Effective Peak Velocity-Related Acceleration (EPV), A_v. Although these parameters are not precisely defined in physical terms, they are considered as normalizing factors for the construction of smoothed elastic response spectra for ground motions of normal duration. (*10*) The EPA is proportional to spectral ordinates for periods of 0.1 to 0.5 seconds, and the EPV is proportional to spectral ordinates at a period of about 1 second. (*11*) The constant of proportionality (for a 5 percent damped spectrum) is standard at 2.5. Thus when the ordinates of a smoothed spectrum between the periods mentioned above are divided by 2.5, the EPA and EPV are obtained. The EPA and EPV are related to peak ground acceleration and peak ground velocity but are not necessarily the same or even proportional to peak acceleration and velocity.

The new seismic design guidelines for highway bridges provide for two methods of analysis that vary according to the refinement in the mathematical idealization. They are the single mode spectral analysis method and the multimode spectral analysis method. Both methods assume simultaneous support excitation. The single mode spectral method is recommended for bridges near New Madrid, Mo., although no special analysis (SPC–A) is recommended for Eastern bridges outside this region. Thus, use of simple recommended details will suffice for most Eastern bridges.

The single mode spectral method requires a calculation of a period. The method is based on the premise that the mode shape of the vibrating structure can be assumed and represented by a shape function that can be expressed mathematically in terms of a single generalized coordinate taken as the amplitude at the point of maximum displacement. It is designed to approximate the dynamic character of the bridge. The method consists basically of the following steps:

1. Determine the period for the assumed mode.

2. Determine the corresponding seismic coefficient as a function of A_v, structure period, and soil type.

3. Determine the maximum displacement from the seismic loading.

4. Determine the component forces corresponding to the maximum displacement.

Application of the guidelines for new construction will undoubtedly reduce the vulnerability of Eastern bridges to potential damage but will require many decades to completely implement. There are many Eastern bridges that require immediate rehabilitation. Relatively simple, applicable retrofitting techniques have been developed in recent years for use on these bridges.

Retrofitting techniques

Before taking any steps to retrofit an existing bridge, it is necessary to decide whether the bridge actually needs retrofitting and if so what kind or kinds of retrofit measures to use. To determine the need for retrofitting, a structural seismic analysis should be performed. A simplified structural analysis, which adequately considers the principal modes of bridge response when subjected to the probable site-dependent seismic loading, can be made. Should the results prove marginal as far as probable failure, then a more detailed structural analysis may be necessary. If the analyses indicate that failure will occur that is so extensive that the bridge could not remain in emergency use, a retrofit measure can be designed based on the mode and extent of failure predicted by the analysis. It is, however, important to consider that strengthening a component that is susceptible to a particular mode of seismic damage may actually lead to a different mode of failure, or possibly to failure of another component.

Observed failure modes for conventional bridge structures subjected to seismic loading are grouped into two categories—substructure failures (column, pier, or abutment) that lead to loss of support capacity and superstructure collapse from excessive relative motion at supports. Structural failure and damage to bridges also may be caused by inadequate foundation strength or load bearing degradation during seismic loading. Soil liquefaction is an example of a cause of this failure mode.

The following five bridge retrofit techniques have been identified and several measures have been developed for each. (*12, 13*)

- Superstructure horizontal motion restrainers for hinges, expansion joints, and bearings.
- Vertical bearing support restrainers.
- Bearing area widening techniques.
- Column or pier strengthening.
- Footing strengthening.

Specific retrofit measures must be cost effective. It is important to emphasize that seismic and structural considerations are not the only considerations in the bridge retrofit decision process. Other decisions are the importance of the bridge to the given locality based on the kind of highway, traffic volume, and accessibility of other crossings and the replacement or repair costs because of estimated damage and lost time.

A brief summary of some retrofit measures are given below:

1. Restricting longitudinal, vertical, and lateral relative displacements of the superstructure at expansion joints and bearing seats by means of cables, tie bars, shear keys, extra anchor bolts, or metal stoppers.

2. Restricting rigid body motion of the superstructure by connecting it with high strength steel cables to a supporting or an adjacent foundation or pier cap, enlarging bearing areas, or placing stoppers at edges of bearing areas.

3. Reducing induced vibrations by installing energy-absorbing devices such as elastomeric bearing pads at bearing seats or adapting a "shock absorber" kind of damper that allows slow movement such as displacement from creep, shrinkage, and temperature change with negligible resistance but develops a large resistance in the event of a rapid displacement.

4. Strengthening substructure elements such as increasing the strength of an existing column by adding longitudinal and spiral reinforcement to the exterior of the column then bonding the added reinforcement with a new layer of high-strength concrete using pressure grouting procedures and Gunite. The additional longitudinal reinforcement also could be extended into the cap and the footing, thus increasing the flexural strength of the column-to-cap and column-to-footing connections.

It is likely that there are many possibilities; an effort must be made to identify those that are cost effective for Eastern highway bridges.

The major obstruction to implementing the above measures East of the Rocky Mountains is the general attitude that damaging earthquakes do not occur in this part of the United States. However, the effects of a big Eastern earthquake would be felt over large areas. Implementation of design guidelines in new construction is an economical way of improving the seismic resistance of the highway system. Many major existing river crossings in the East should be considered as candidate structures for retrofit. A modest investment in tying structures together, restraining superstructure motion, and stabilizing bearings can pay large dividends when the next large earthquake occurs. Available retrofit technology must be implemented into the existing highway system if future disaster is to be avoided.

Research Needs

Two workshops were held to discuss and identify research needs to improve the seismic vulnerability of the highway system. The first, sponsored by the American Society of Civil Engineers Technical Council on Lifeline Earthquake Engineering, was held on Sept. 11–12, 1978. Participants of the workshop developed research needs statements for each of the major lifeline areas. The second workshop, sponsored by the National Science Foundation and conducted by the Applied Technology Council, was held Jan. 29–31, 1979. It specifically addressed the earthquake resistance of highway bridges. In addition to research needs, the participants discussed the state of the art in bridge design and research. Following are some of the recommendations resulting from these workshops.

Analysis

Simplified computer-oriented analytical techniques are essential to determine the seismic response of typical kinds of bridges. Currently available general analysis programs tend to be too cumbersome for the periodic use required by the bridge designer. Programs developed should include linear dynamic analysis capabilities and be tailored specifically to the analysis and design of bridges. They should provide incremented processing capabilities with an accessible data base to define and modify input data, examine intermediate results, and store final output results for postprocessing. Preprocessing should be included to generate models with the characteristics unique to bridge structures. The development of such a program or programs presupposes that a library of appropriate loading conditions has been developed and can be retrieved for use in the analysis.

Once simplified programs are developed, parametric studies should be conducted to determine and classify the seismic response of typical kinds of bridges according to general features. Effects of parameters such as span length, column height and stiffness, curvature, skew, material, and restraint conditions on gross structural response should be determined. Then, properties and conditions that tend to be more resistant to seismically induced loading can be optimized.

The accuracy of computer programs developed should be verified. Assumptions or approximations made in the mathematical idealization can be checked if ambient as well as forced vibration studies are conducted on existing bridges and subsequent results are compared with theoretical predictions. The confidence of bridge designers will be enhanced when bridge response predictions correlate with analytical models.

Experiment

Experimental studies, conducted in the laboratory and field, are essential to evaluate the dynamic characteristics and response of the complete bridge system as well as individual components. Laboratory studies that would provide information useful to bridge designers in regions of lower seismic activity include the evaluation of commonly used anchorage and splice requirements of reinforcing bars under cyclic load. Several earthquakes have demonstrated vividly that these details are inadequate during strong ground motion. The vulnerable connection details occur between columns and footings and columns and column caps. An experimental program could demonstrate that cost-effective details adequately can resist the rather infrequent seismic activity that occurs in the Eastern United States.

Bearings commonly used in Eastern bridges probably are the most vulnerable connection detail of all. They significantly affect the seismic behavior of the structures. Studies should be conducted to investigate dynamic force-deflection characteristics. In conjunction with bearing response, restrainer devices similar to those commonly used in California should be investigated. Restrainers used with bearings may provide the most cost-effective means of increasing the seismic resistance of structures subjected to infrequent seismic loading.

Most laboratory studies are conducted on scale models and details. Periodically, bridges are removed from service because of rerouting or reuse of the land where the structure is located. All too frequently the structure is dismantled without experimental investigation. It would be prudent to conduct large amplitude dynamic load testing on typical kinds of surplused bridges. A properly designed experiment conducted under controlled loading can provide more insight into the performance of the bridge than any other test.

It is essential to obtain strong motion records from structures subjected to earthquake excitation. Instrumentation, if placed only on a few bridges in the more vulnerable areas of the Eastern United States, can provide very useful data and further insight into the behavior of bridges during an earthquake. Actual measurement of bridge response, whether from controlled field tests or strong motion records, is the only reliable way to obtain data that can be correlated with theoretical analysis.

Retrofit

Retrofit details have been developed to reduce the likelihood of bridge collapse (*12, 13*) but there still remains much research that should be conducted to establish a basis for selecting cost-effective retrofit details as well as determining the strength level to which a bridge can be economically retrofitted. An appropriate retrofit design level must be established, above which replacement would become feasible.

User needs

Communication between researcher and designer is essential if research is to be implemented. Significant research results on the earthquake resistance of highway bridges must be synthesized and presented to the designer in a useful format. Special reports, booklets, and journal articles can be developed and meetings and short courses can be held to integrate research and design.

Conclusions

Currently, cost-effective guidelines exist that can reduce the potential for earthquake-induced damage of Eastern highway bridges. These newly developed design guidelines should be implemented in new construction. However, this will occur only if the seismic design guidelines are adopted by AASHTO.

As awareness grows of the potential hazards associated with Eastern earthquakes, initial steps will be taken to identify the more important structures that must remain operational and to apply retrofit techniques. Hazards and risks associated with infrequent Eastern events must be identified and information disseminated so that rational decisions can be made for design and retrofit. At the same time, many unknowns must be addressed through additional research. This long, tedious process is the only way to provide solutions on how best to protect the highway system from the rarely occurring, but potentially devastating, earthquake.

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

James D. Cooper serves as Federal project manager for the Federal Highway Administration's FCP Project 5A "Improved Protection Against Natural Hazards of Earthquake and Wind." He is responsible for the organization, planning, and management of this part of the national program. Mr. Cooper is also responsible for coordinating and integrating the results of research sponsored by others into a cohesive plan to attain the goals of the FCP Project 5A. Before this position, Mr. Cooper served a Presidential Internship in Engineering and Science with the Office of Research, Federal Highway Administration, and was an instructor at Syracuse University. Mr. Cooper has authored and coauthored several research papers.

Jerry Wachtel and Ross Netherton Receive Awards

Mr. Jerry A. Wachtel and Dr. Ross B. Netherton were the recipients of the 1981 award in the annual outstanding technical achievement competition held among the employees of the Federal Highway Administration (FHWA) Offices of Research and Development. The award covers the documentation of a unique technical accomplishment, which may be a technical paper, staff report, or implementation package; an innovative engineering concept; a patent; an instrumentation system; a test procedure or specification; a mathematical model; or a complex computer program. Candidates for the award are judged on excellence, creativity, and contribution to the highway community, general public, and FHWA.

Mr. Wachtel, a research psychologist in the Urban Environment Group, and Dr. Netherton, a social scientist analyst also in the Urban Environment Group, Environmental Division, Office of Research, received awards for their assessment of the potential influence of commercial electronic variable-message signs (CEVMS) on highway safety and environmental quality. Through critical analyses of state-of-the-art sign presentation technology, accident studies, and other literature, they concluded that indiscriminate use of this new technology can cause serious safety, environmental, and highway investment problems.

Their assessment is documented in the report "Safety and Environmental Design Considerations in the Use of Commercial Electronic Variable-Message Signage," Report No. FHWA/RD–80/051. A summary of this research effort, "Electronic Advertising Along Highways—Concern for Traffic Safety," by Jerry Wachtel, was presented on page 1 of the June 1981 issue of *Public Roads*.

The research report has been used in billboard litigation and recently was lodged with the U.S. Supreme Court by the Attorney General for use in deciding a commercial signing case involving the First Amendment. It also has led to additional research to define the conditions under which CEVMS may safely be used without creating environmental problems or economic burdens for the jurisdictions where the signs are used.

Recent Research Reports You Should Know About

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

Age Effects on Symbol Sign Recognition, Report No. FHWA/RD–80/126

by FHWA Traffic Systems Division

This report is concerned with the effects of age and training on drivers' learning and retention of symbol knowledge. The research was done using a driving simulator. Subjects were scored on how often they correctly recognized the signs and the distance from the signs at which recognition took place.

It was found that all age groups learned and retained approximately the same number of symbols, but the older age groups began with less symbol knowledge. Data interpretation also indicated that the older subjects required longer recognition and response times. Simple, bold, and unique symbols were found to be the most effective.

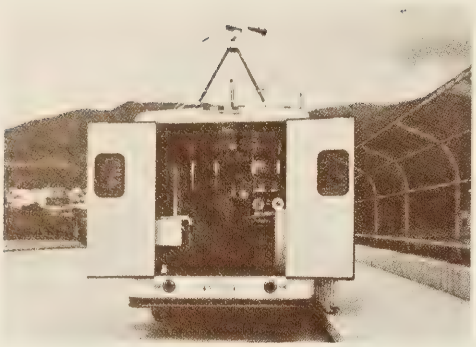
Limited copies of the report are available from the Traffic Systems Division, HRS–32, Federal Highway Administration, Washington, D.C. 20590.

Control Strategies in Response to Freeway Incidents: Executive Summary (Report No. FHWA/RD–80/004), Technical Report (Report No. FHWA/RD–80/005), Software Documentation (Report No. FHWA/RD–80/006), and Operational Guidelines (Report No. FHWA/RD–80/007)

by FHWA Traffic Systems Division

New control strategies for use with electronic surveillance and control systems have been developed for responding to freeway incidents. Two kinds of incident situations were considered—the control of onramps in the immediate vicinity of the incident site and the control of freeway segments and connectors located upstream of the freeway segment containing the incident. The strategies were tested extensively under a variety of simulated incident conditions. For example, control strategy performance was evaluated under three levels of service with incidents blocking either one or two lanes for 10 to 30 minutes. A new strategy based on an optimization of mainline service and travel time performed better under freeway incident conditions than control strategies previously used for recurrent congestion.

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



Sensor for Control of Arterials and Networks (SCAN) Breadboard Hardware, Report No. FHWA/RD–80/024

by FHWA Traffic Systems Division

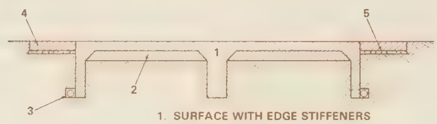
This report discusses the breadboard design of SCAN, a new traffic detection and tracking sensor. The sensor provides both television images of traffic and numerical values of traffic parameters. The traffic flow parameters of volume, speed, and density are determined by two existing traffic monitoring systems—the remote television camera system, which provides only pictures, and the computerized point-sensor-based system, which provides only limited traffic flow measurements.

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 81 202202).

New Structural Systems for Zero-Maintenance Pavements. Analytical and Experimental Studies of an Anchored Pavement: A Candidate Zero-Maintenance Pavement, Report No. FHWA/RD–80/026

by FHWA Structures and Applied Mechanics Division

Anchored Pavement



This report compares the behavior of a new anchored pavement concept with that of a conventional rigid pavement system in both continuous and jointed configurations. In cross section, the anchored pavement resembles a concrete double T-beam with the flange acting as the pavement surface. The 1.2 m (4 ft) deep by 0.3 m (1 ft) wide concrete webs are embedded in the subgrade soil to provide anchorage. The anchored pavement is deemed a promising zero-maintenance concept because it poses few construction difficulties, requires little subgrade preparation, and uses quantities of structural materials comparable to those in conventional rigid pavement systems.

The study methodology included the following: Finite element analyses of environmental stresses and of vehicle wheel loading stresses that were subsequently superimposed; load tests on aluminum 1:20 scale models of an anchored pavement and of a conventional rigid pavement slab of equal length each supported on an identical subgrade of known properties; and comparisons of analytical and experimental results with field data collected from various road tests and reported in the literature. A two-dimensional finite element mesh was selected for the environmental stress analysis and a three-dimensional finite element mesh for the wheel load stress analysis. Both were used in conjunction with the computer program Engineering Analysis System (ANSYS).

The anchored pavement concept offers two advantages over a conventional rigid pavement system. First, deflections of the anchored pavement are generally lower by as much as one order of magnitude and are more uniformly distributed; curling deflections are nearly eliminated by the stiffened edges of the anchored pavement. Second, loads applied to the anchored pavement are distributed more uniformly and at greater depth to the subgrade soil, making subgrade failure less likely to occur.

Stresses induced in the anchored pavement from the anchorage reaction restraints may require reinforcing steel. Also, the anchored pavement concept poses possible problems in joint design and construction. Theoretically, separation of the flange and webs may be necessary at joints to permit differential movement. A proposed joint design is presented for an anchored pavement with a sleeper slab beneath the joint that may overcome the problem.

In the first of two supplemental reports, **Analysis of Anchored Pavements Using ANSYS** (Report No. FHWA/RD–80/027), a program manual is provided for evaluating the response of an anchored pavement to vehicle static loads, moisture variation in the subgrade, and temperature variation through the pavement.

In the second supplemental report, **Anchored Pavement Design for Edens Expressway** (Report No. FHWA/RD–80/028), a design and cost estimate is provided for the hypothetical case of construction of an anchored pavement on a heavily traveled highway in Chicago, Ill.

The reports are available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock Nos. PB 81 157554, PB 81 157562, and PB 81 157570).

Evaluation of Test Methods and Use Criteria for Geotechnical Fabrics in Highway Applications, Report No. FHWA/RD–80/021

by FHWA Structures and Applied Mechanics Division

In recent years, geotechnical fabrics have been used widely in a variety of highway construction applications. Four categories of highway construction use of geotechnical fabrics are drainage, soil layer separation, soil reinforcement, and erosion control. This report will assist the engineer in choosing the most suitable and cost-effective fabric for a particular application. The more common polymers used in the manufacture of fabric filaments and their general characteristics are described. Also covered are the general relationships of the properties of a fabric to the fabric construction method, the nature of the filament, and the weight of the material per unit area. In most instances fabric construction is more significant than polymer type.

Many kinds of fabric are already on the market and new kinds are being introduced. Currently, the two polymers used in the manufacture of most geotechnical fabrics in the United States are polypropylene and polyester. Various polymers are significantly different, and the fibers produced by different processes from the same polymer may also differ. The more important polymer characteristics influencing the properties of the fabric are specific gravity, strength, strain at failure, modulus of elasticity, creep, reaction to cyclic loading, and the fabric stability or durability relative to temperature, water immersion, ultraviolet light, chemical activity, and biological attack.

The report presents test criteria for highway applications of fabrics and evaluates existing test methods for determining fabric properties. Findings in the report were derived from references on previous research and from field visits to selected State highway and transportation agencies to collect information on fabric usage. Limited analytical studies were conducted to facilitate the understanding of soil-fabric interaction and the development of design criteria.

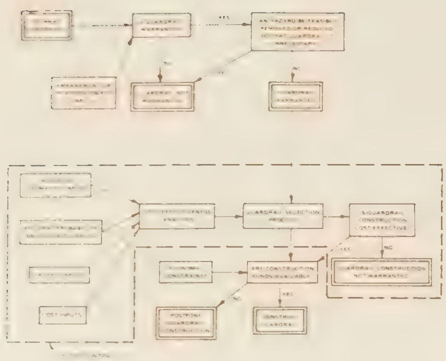
The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 81 156150).

Improving the Residential Street Environment, Report Nos. FHWA/ RD–80/092, FHWA/ RD–81/030, and FHWA/ RD–81/031

by FHWA Environmental Division

Three reports have been prepared on techniques for managing traffic on residential streets. The first, State of the Art Report: Residential Traffic Management, discusses current engineering practices and assesses the performance of various control devices affecting traffic on existing residential streets. The second two reports, an executive summary and a final technical report, discuss research on the effectiveness of "road humps" on street control on residential streets, resident preferences on traffic speed and volume control techniques, and legal considerations in neighborhood traffic management.

Limited copies of the reports are available from the Environmental Division, HRS–41, Federal Highway Administration, Washington, D.C. 20590.



Development of a Cost–Effectiveness Model for Guardrail Selection, Executive Summary (Report No. FHWA–RD–78–73), Vol. I, Technical Documentation (Report No. FHWA–RD–78–74), and Vol. II, Users Manual (Report No. FHWA–RD–78–75)

by FHWA Environmental Division

These reports document the development of a cost-effectiveness model for guardrail selection and include cost parameters for various guardrail configurations as well as criteria for analysis of system effectiveness under various dynamic conditions.

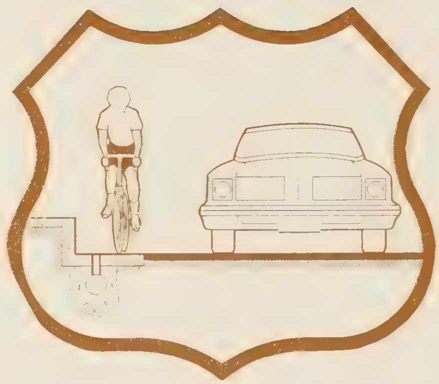
Initially, two computer programs were developed to determine the cost effectiveness of a specific guardrail type and comparative cost effectiveness rankings of 11 guardrail types. It is important to note that in the development and application of this model the guardrail was assumed to be warranted.

Site selection tables also were developed from the models to approximate how the various guardrails would rank. Only the direct costs of the accidents were used to avoid the controversy surrounding societal costs of accidents. This result has little effect on the relative rankings but does affect the site selection tables. The cost variable is also an input variable when using the models to determine appropriate values for the costs of accidents.

Limited copies of each volume of the report are available from the Environmental Division, HRS–43, Federal Highway Administration, Washington, D.C. 20590.

Bicycle-Safe Grate Inlets Study: Vol. 4, Report No. FHWA–RD–79–106, and Vol. 5, Report No. FHWA/RD–80/081

by FHWA Environmental Division



The first three volumes of this study deal with results of tests on grate inlets. They were reported in *Public Roads,* vol. 41, No. 4 (March 1978) and vol. 42, No. 4 (March 1979).

These two reports present results of tests conducted on slotted drain inlets. This kind of inlet is an excellent alternative for the commonly used grate inlet. The reports cover slot length needed to capture the entire gutter flow, slot width needed to capture the entire road surface flow under very heavy rainfall along both edges of a roadway, partial flow interception by the slot, and the relationship between the depth of ponded water, and the flow interception of a slot at the low point of a sag vertical curve (the sump condition). Tests were conducted with a range of longitudinal and cross slopes of roadway. Full-scale tests were conducted for most of these tests; a scale model was used in the sump condition. Debris handling capability was evaluated on a relative basis. Preliminary design equations were derived from the test data.

Volume 4, **Hydraulic Characteristics of Slotted Drain Inlets,** deals with the basic design of a 44.5 mm (1.75 in) wide slot inlet in the curb and gutter road section. Volume 5, **Hydraulic Design of General Slotted Drain Inlets,** deals with more general slotted drain inlet applications. It covers a range of slot widths and curb to slot distances so the results can be used not only in the curb and gutter section of the road but also in the drainage channel.

The reports are available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock Nos. PB 80 174253 and PB 81 210742).

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

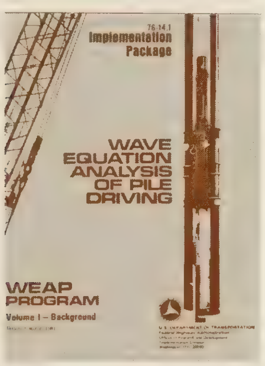
The following are brief descriptions of selected items that have been recently completed by State and Federal highway units in cooperation with the Implementation Division, Office of Development, Federal Highway Administration (FHWA). Some items by others are included when they have a special interest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

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

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

Wave Equation Analysis of Pile Driving, Implementation Package 76–14 (Updated March 1981)

by FHWA Implementation Division



In 1976 the Implementation Division developed the computer program "Wave Equation Analysis of Pile Driving" (WEAP). An implementation package consisting of four volumes was developed to explain the theory and application of the wave equation to analyze pile driving problems. Subsequently, in 1977–1979 more than 10 seminars were held around the country to train highway engineers in the use of the wave equation. Since then, additional data and knowledge on the efficiency of pile driving hammers have been obtained, and suggestions for improvement of the computer program have been made by the users. As a result, the computer program has been updated and three of the four volumes have been revised.

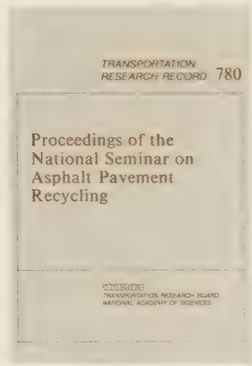
The implementation package, including Volume I, **Background,** Volume II, **Users Manual,** and Volume III, **Program Documentation,** will be of interest to geotechnical, bridge, and construction engineers involved in the design and construction of pile foundations.

Limited copies of the implementation package are available from the Implementation Division. A copy of the computer program is available to State highway agencies interested in implementing the updated wave equation.

Proceedings of the National Seminar on Asphalt Pavement Recycling, Transportation Research Record No. 780

by Transportation Research Board

This seminar conducted by the Transportation Research Board under the sponsorship of FHWA's Implementation Division was held in Dallas/Fort Worth, Tex., on October 14–16, 1980. The seminar covered all aspects of asphalt pavement recycling, including surface, hot, and cold recycling in both rural and



urban situations. Representatives from State, local, and Federal governments, industries, and universities attended the seminar. This report of the proceedings covers the advantages and disadvantages of various kinds of recycling, cost and energy considerations, specifications, quality control, and environmental considerations. The report should be of interest to highway agencies and the paving industry.

Limited copies of the report are available from the Implementation Division. Copies are also available from the Transportation Research Board.

Culvert Design System, Report No. FHWA–TS–80–245

by FHWA Implementation Division

Highway engineers traditionally have designed culverts by analyzing data on the design flood, the hydraulics of culverts, and the storage effect of temporary ponding. Until recently this was sufficient to satisfy highway drainage design requirements. However, a need for more exact drainage design and documentation has been realized because of the escalating cost of highway drainage construction, increased environmental concern, and recent Federal flood hazard regulations.

This report documents a Wyoming computer system (program), which can, through hydraulic analysis, design a culvert or review an existing or proposed culvert size. The system provides certain environmental and flood data in addition to the culvert hydraulics. The system can be used in any geographical region provided discharges, hydrographs, and flood volumes can be identified.

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

High Pressure Salt Brine Deicing Conference, Report No. FHWA–TS–81–202

by FHWA Implementation Division

The prime objective during ice and snow control operations is to maintain or obtain an acceptable level of highway service by destroying an existing snow pack or ice glaze. A system has been developed to directly apply salt brine which significantly reduces the salt needed for snow and ice control. Six units have been built and are being field tested to determine the effectiveness and costs of using salt brine for deicing.

This report covers the conference held in Connecticut on March 11–12, 1980, to acquaint maintenance and research personnel from States, counties, and cities with the salt brine system. The conference provided the States of California, Minnesota, Utah, and Washington the opportunity to discuss their field work.

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

Safety Rest Area—Planning, Location, Design, Report No. FHWA–IP–81–1

by FHWA Implementation Division

Of the approximately 2,000 safety rest areas planned for the Interstate System, about 1,400 have been constructed. Of these, more than 700 must be enlarged or modified to meet the needs of today's traveling public. This manual provides information needed by State highway engineers to complete safety rest area programs and improve existing facilities. The material presented covers the essential principles in planning, locating, designing, and upgrading safety rest areas.

Copies of the manual may be purchased for $5 from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Stock No. 050–001–00108–6).

New Research in Progress

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

FCP Category 1—Improved Highway Design and Operation for Safety

FCP Project 1P: Visual Guidance for Night Driving

Title: Quantifying Visual Complexity of the Night Driving Scene. (FCP No. 31P1012)
Objective: Develop and field validate a technique for quantifying the visual complexity of various highway situations that will aid in predicting the driver's need for visual guidance information.
Performing Organization: Comis Corporation, Wheaton, Md. 20902
Expected Completion Date: November 1983
Estimated Cost: $340,000 (FHWA Administrative Contract)

FCP Project 1V: Roadside Safety Hardware for Nonfreeway Facilities

Title: Guardrail and Median Barrier Terminals for Mini-Sized Cars. (FCP No. 31V3262)
Objective: Develop and test one or more end terminals for straight sections of guardrails and median barriers that can safely handle mini-sized cars as well as full-sized cars in impacts. Adapt and test flared breakaway cable terminals using mini-sized cars.
Performing Organization: Southwest Research Institute, San Antonio, Tex. 78284
Expected Completion Date: June 1983
Estimated Cost: $263,000 (FHWA Administrative Contract)

FCP Category 2—Reduce Congestion and Improve Energy Efficiency

FCP Project 2J: Automated Highway Systems

Title: Studies in Passive Guideway Automatic Vehicle Control. (FCP No. 32J1082)
Objective: Develop, test, and evaluate technologies required for automatic control of vehicles using passive guideways. Focus on the use of radar for lateral control.
Performing Organization: Ohio State University, Columbus, Ohio 43210
Expected Completion Date: May 1983
Estimated Cost: $542,000 (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 Highway Systems

Title: Effects of Highway Runoff on Receiving Waters. (FCP No. 33E3232)
Objective: Design and install water quality sampling and flow measurement station, collect samples, and service and maintain monitoring stations.
Performing Organization: U.S. Geological Survey, Raleigh, N.C. 27602
Expected Completion Date: July 1982
Estimated Cost: $93,000 (FHWA Administrative Contract)

FCP Project 3F: Pollution Reduction and Environmental Enhancement

Title: Traffic Detector Handbook. (FCP No. 33F2000)
Objective: Develop a practical handbook on the selection, design, installation, and testing of traffic detectors.
Performing Organization: Diaz, Seckinger & Associates, Tampa, Fla. 33607
Expected Completion Date: September 1983
Estimated Cost: $151,000 (FHWA Administrative Contract)

Title: NEMA Training. (FCP No. 33F6201)
Objective: Develop training on NEMA local intersection traffic signal controllers. Combine with training on California/New York Type 170 nonproprietary general purpose traffic signal controller system to produce a microprocessor based traffic controller course.
Performing Organization: Texas State Department of Highways and Public Transportation, Austin, Tex. 78787
Expected Completion Date: June 1983
Estimated Cost: $135,000 (FHWA Administrative Contract)

FCP Category 4—Improved Materials Utilization and Durability

FCP Project 4H: Improved Foundations for Highway Structures

Title: Lateral Resistance of Cast-In-Drilled-Hole Piles in Sloping Ground. (FCP No. 44H4042)
Objective: Determine if the horizontal resistance of cast-in-drilled-hole piling is greater than predicted in sloping ground, and develop design formulas for lateral capacity.
Performing Organization: California Department of Transportation, Sacramento, Calif. 95807
Expected Completion Date: June 1983
Estimated Cost: $249,000 (HP&R)

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

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

Title: Earthquake Response and Design of Long-Span Suspension Bridges. (FCP No. 35A1032)
Objective: Investigate the dynamic response of long-span suspension bridges to large magnitude earthquakes through analytical and experimental research. Prepare seismic design guidelines based on parametric investigations.
Performing Organization: National Science Foundation, Washington, D.C. 20550
Expected Completion Date: December 1984
Estimated Cost: $100,000 (FHWA Administrative Contract)

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

Title: Predicting Scour at Bridges: Questions Not Fully Answered. (FCP No. 45H1362)
Objective: Conduct a laboratory study of local scour below sills used to safeguard existing bridges from stream degradation.
Performing Organization: University of Arizona, Tucson, Ariz. 85721
Funding Agency: Arizona Department of Transportation
Expected Completion Date: June 1983
Estimated Cost: $130,000 (HP&R)

FCP Project 5L: Safe Life Design for Bridges

Title: Risk Analysis of Extending Bridge Service Life. (FCP No. 45L3162)
Objective: Improve the economy of bridge replacement by determining the conditions under which the safe fatigue life of a steel bridge can be extended without exceeding a specified level of risk.
Performing Organization: University of Maryland, College Park, Md. 20742
Funding Agency: Maryland State Highway Administration
Expected Completion Date: December 1982
Estimated Cost: $125,000 (HP&R)

FCP Category 6—Improved Technology for Highway Construction

FCP Project 6C: Use of Waste as Material for Highways

Title: Early Age Durability of Fly Ash Concrete. (FCP No. 46C2282)
Objective: Determine the early durability of fly ash concrete mixtures and compare them with conventional concrete mix. Substitute fly ash for cement on equal volume and greater volume basis. Establish relationship between durability and early age.
Performing Organization: West Virginia University, Morgantown, W. Va. 26506
Funding Agency: West Virginia Department of Highways
Expected Completion Date: May 1983
Estimated Cost: $65,000 (HP&R)

FCP Project 6D: Pavement Rehabilitation

Title: Correlating Dynamic Deflections With Pavement Performance. (FCP No. 46D1412)
Objective: Select a deflection testing device for State pavement inventories. Demonstrate the level of correlation between pavement performance and deflection measurements made with Benkelman Beam, Road Rater, and falling weight deflectometer. Compare elastic layer properties derived from the measurements.
Performing Organization: Alaska Department of Transportation and Public Facilities, Juneau, Alaska 99811
Expected Completion Date: March 1983
Estimated Cost: $86,000 (HP&R)

Title: Analysis of Low Modulus Interlayers in Existing Pavements. (FCP No. 46D2784)
Objective: Investigate the mechanistic behavior of stress absorbing concrete in the overall

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pavement overlay system to optimize its use in different applications. Establish criteria for overlay design.
Performing Organization: Arizona Department of Transportation, Phoenix, Ariz. 85007
Expected Completion Date: June 1983
Estimated Cost: $57,000 (HP&R)

Title: Alternate Resurfacing Strategies. (FCP No. 46D2804)
Objective: Develop alternative resurfacing schemes that are more cost effective than the current standard procedures. Conduct laboratory tests of unconventional materials, and produce a field evaluation plan for the promising strategies.
Performing Organization: Austin Research Engineers, Austin, Tex. 78746
Funding Agency: Texas State Department of Highways and Public Transportation
Expected Completion Date: September 1983
Estimated Cost: $175,000 (HP&R)

Title: Forecasting Effects of Frost Action. (FCP No. 46D3412)
Objective: Relate deflection to subgrade moisture, frost, and pavement temperatures. Develop procedures to permit timely application of load restrictions.
Performing Organization: University of Washington, Seattle, Wash. 98105
Funding Agency: Washington State Department of Transportation
Expected Completion Date: June 1983
Estimated Cost: $81,000 (HP&R)

FCP Project 6G: Quality of Construction

Title: State of the Art in Flexible Pavement Specifications. (FCP No. 36G1013)
Objective: Assess the current flexible pavement specifications used by the States. Consider full-depth flexible pavements and one-course overlays and construction and materials requirements. Develop the general framework for performance-related flexible pavement specifications.
Performing Organization: Sheladia Associates, Inc., Riverdale, Md. 20840
Expected Completion Date: May 1983
Estimated Cost: $181,000 (FHWA Administrative Contract)

Title: Performance-Related Specifications for Hydraulic Cement Concrete Used in Construction and Rehabilitation of Ground Transportation Facilities. (FCP No. 46G2012)
Objective: Establish quality assurance procedures for concrete that not only indicate compliance or noncompliance with requirements but also indicate the quality level and variability of the concrete inplace.
Performing Organization: Virginia Highway Research Council, Charlottesville, Va. 22903
Funding Agency: Virginia Department of Highways and Transportation
Expected Completion Date: December 1983
Estimated Cost: $100,000 (HP&R)

FCP Category 0—Other New Studies

Title: Improvements in Highway Materials and Pavement Systems. (FCP No. 40F2974)
Objective: Develop an improved seal coat design method based on field experience.
Performing Organization: Texas Transportation Institute, Austin, Tex. 78763
Funding Agency: Texas State Department of Highways and Public Transportation
Expected Completion Date: May 1984
Estimated Cost: $175,000 (HP&R)

Title: Principles of Construction of Quality Hot-Mix Asphalt Pavements. (FCP No. 30F6190)
Objective: Develop a manual, training course, and related instructor and student materials on the principles of construction quality hot-mix asphalt pavements.
Performing Organization: The Asphalt Institute, College Park, Md. 20740
Expected Completion Date: October 1982
Estimated Cost: $100,000 (FHWA Administrative Contract)

Title: Evaluation of Experimental Portland Cement Concrete (PCC) Construction. (FCP No. 40S4664)
Objective: Study texturing patterns and joint faulting in PCC pavements.
Performing Organization: California Department of Transportation, Sacramento, Calif. 95814
Expected Completion Date: June 1983
Estimated Cost: $80,000 (HP&R)

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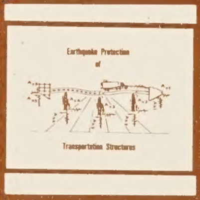
in this issue

Dallas Freeway Corridor Study

Proposed Plans for Measurement of the Aerodynamic Behavior of the Luling, Louisiana Cable-Stayed Bridge

Rapid Measurement of the Chloride Permeability of Concrete



Mitigation of Earthquake Damage on Eastern Highway Systems

public roads

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