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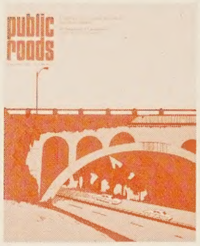

**COVER:**

Artist's concept of a bridge spanning a highway.

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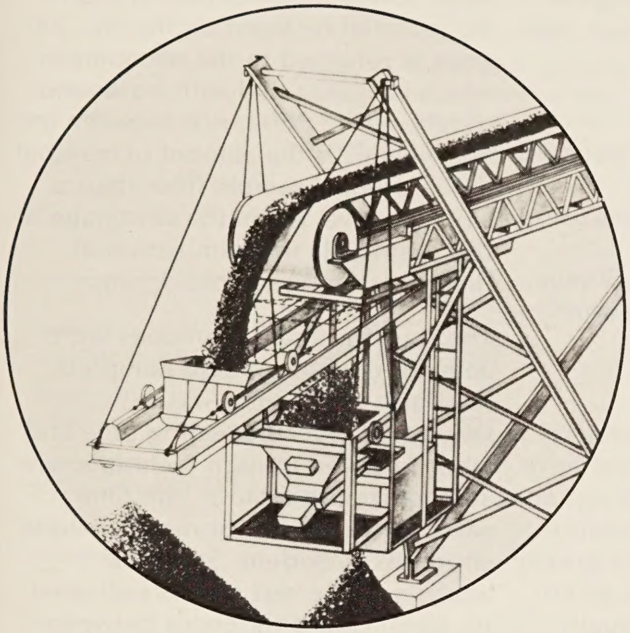

# Process Control for Aggregate Production and Use

by

**Stephen W. Forster**

## Introduction

Process control is the maintenance or control of a certain characteristic or set of characteristics for a product during its manufacture or processing. In aggregate production, one of the characteristics controlled is the gradation of the particles, that is, the percentage of the sample retained on or passing different-sized sieves. The percentage may vary within a limited range specified for each sieve. For example, 25 to 35 percent of the sample may pass the No. 4 sieve.[1]

Gradation control insures that products made from the aggregate are acceptable and uniform. Undue changes in aggregate gradation affect, for example, the optimum moisture content and maximum density of base courses, the cement and water requirements in portland cement concrete, and the amount of binder required in bituminous mixes.

[1] Standard sieve numbers referred to in this article are the No. 4 sieve (4.75 mm [0.19 in]), the No. 10 sieve (2 mm [0.08 in]), and the No. 200 sieve (0.075 mm [0.003 in]). Sieve numbers will be used instead of measurements.

State highway and transportation departments run more than 1 million gradation process control tests annually, at a minimum of $15 per test. Because these tests require significant labor time, speeding up or automating the testing process would be cost effective. A task was established under the Federally Coordinated Program (FCP) of Highway Research and Development to examine current test methods and equipment and develop faster, more economical methods for monitoring the gradation of aggregates during production or use.

The objective of the FCP Task 4F7, "Process Control for Aggregate Production and Use," is being pursued in three ways:

- Shortcuts and alternatives to standard sieve analysis procedures are being examined. Alternative tests currently in limited use are simpler and faster than standard methods, even though these tests still are based on the sieving technique.
- Automated gradation testing is also being studied. Techniques being evaluated are not limited to variations of mechanical screening but include any technology that can be adapted to sizing aggregate. In one approach the testing device is online with the aggregate production process; therefore, samples do not have to be taken to a laboratory for testing. In another approach a prototype of an automated instrument for laboratory testing is being built.
- Several State highway and transportation department testing programs are being studied to determine the costs and benefits of shifting responsibility for most of the gradation testing from the State to the aggregate producers.

## Current Practice

Under the FCP task, the Federal Highway Administration (FHWA) Region 10 Office of Federal Highway Projects (Vancouver, Wash.) surveyed the status of process control testing of aggregate gradation in the United States. Shortcuts or alternative methods that had the potential to reduce costs while maintaining the accuracy of test results were noted.

The state-of-the-art survey indicated that the frequency of aggregate sampling and gradation testing varies widely. For example, the frequency of base course aggregate sampling and testing varies from once per 225 Mg (250 tons) to once per 6400 Mg (7,000 tons). FHWA's Region 10 recommends one test per 910 Mg (1,000 tons) in most instances.

The survey also revealed numerous gradation specifications, even for a given maximum size aggregate. Probably most specifications were developed empirically, independent of what others were using. Although the differences among the specifications are minor, aggregate producers often need multiple stockpiles to meet several slightly different specifications. Regional (or national, where possible) specifications would simplify the jobs of both aggregate users and producers, particularly when the producers sell to more than one user.

Survey results showed that in almost every case, the State or other contracting agency is performing the aggregate control testing rather than the producer. The efficiency of this use of personnel is addressed later in this article.

After examining the alternative testing procedures reported, four techniques were chosen for comparison with standard sieving methods:

- FHWA's Region 10 field technique for speed drying.
- Gap sieving technique derived from FHWA Regions 3 (Baltimore, Md.), 6 (Fort Worth, Tex.), and 10 (Portland, Oreg.). *(1)*[2]
- Iowa's sieve analysis of coarse aggregate (quick method).
- Kentucky's method of testing for the percentage of material finer than a No. 200 sieve by using a pycnometer.

[2] Italic numbers in parentheses identify references on page 61.

In the Region 10 technique, standard procedures are modified by accelerating the drying process using an infrared heater to reach temperatures of 204° C (400° F) plus. This reduces the total testing time by approximately 5 hours.

The gap sieving technique determines the percent of material retained on or passing a critical sieve size. Because the test can be run on aggregate material in any moisture condition, drying time may be greatly reduced or eliminated. For a given aggregate source, there is usually one sieve size at which the material will drop out of the specification band, if it is going out of specification at all. Therefore, once this "critical" size is determined, it is necessary to monitor only that size to control overall gradation. The full range of sieve sizes can be tested periodically to check the gap sieving results.

In the Iowa quick method, only aggregate coarser than the No. 10 sieve is tested because all drying of the aggregate material is eliminated. The finer-sized material cannot be separated because of the moisture in the sample. In practice, the sample is washed over a No. 10 sieve, drained, and sieved in a wet condition. The advantage is that without drying the aggregate, accurate gradation results for the coarse material can be obtained quickly.

In Kentucky's method of determining material finer than a No. 200 sieve, the entire aggregate sample is put in a 3.8 l (1 gal) pycnometer jar that is then filled with water and weighed. Through a series of agitations and decantations, the fine material is suspended and poured from the jar onto a No. 10 sieve over a No. 200 sieve. Once the decantate is clear, the material retained on the No. 200 sieve is returned to the pycnometer, which is again filled with water and weighed. The difference between the two weights is the amount of material in the original sample finer than a No. 200 sieve. Again the advantage of this method is the elimination of time-consuming sample drying.

The four shortcut techniques were compared with standard complete wash test procedures in the laboratory. Specific testing time and labor involved in each technique are summarized in table 1. The time saved by using one or more of these shortcuts is evident. Statistical analysis of the test results indicated no significant differences between percentages of material passing a given sieve for the various test methods when compared with the standard. Field trials of these shortcuts are necessary to determine if they can adequately control aggregate gradation in practice. *(2)*

## Automatic Size Analysis

An estimate of the savings from an automatic technique that are necessary to justify the initial investment in equipment can be derived from labor and testing times for sieving procedures (table 1). Within these economic constraints, two studies were initiated to address the automation of aggregate gradation testing. One study investigated an automated, online sampling and testing system, and the other study is developing a prototype laboratory testing instrument.

### Online systems

The first study investigated available sizing techniques for their adaptability to online use at the aggregate plant. There were no constraints on the techniques studied; that is, techniques were not limited to sieving. Online systems

**Table 1.—Testing times for shortcut techniques using a 16 kg (35 lb) sample**

| Test technique | Initial weighing, drying, and cooling | Initial sieving, cleaning of sieves, and calculation | Batching or splitting | Washing for passing No. 200 sieve | Drying, cooling, and sieving wash sample. Cleaning sieves, and weighing wash sample | Washing material over No. 10 sieve | Placing sample in pycnometer for initial and final weighing | Draining sample | Final calculation | Total test | Actual labor |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | Times | | | | | | | | | | |
| | *Minutes* | *Minutes* | *Minutes* | *Minutes* | *Minutes* | *Minutes* | *Minutes* | *Minutes* | *Minutes* | *Hours* | *Hours* |
| Standard complete wash[1] [2] | 149 | | | 35 | 259 | | | | 5 | 7.47 | 2.05 |
| Region 10's speed drying[1] [2] | 27 | 27 | 9 | 6 | 88 | | | | 9 | 2.76 | 1.38 |
| Gap sieving (one sieve method)[1] [2] | 31 | | | | 9 | | | | 2 | 0.70 | 0.22 |
| Kentucky's pycnometer method | 2 | | 9 | 21 | | | 30 | | 4 | 1.10 | 1.10 |
| Iowa's quick method | 2 | | | | 25 | 9 | | 12 | 5 | 0.88 | 0.88 |

[1] Six percent moisture before initial drying.
[2] Drying temperatures: Standard complete wash=110° C; Region 10=156° C; Gap sieving=160° C.

$°C = \frac{°F - 32}{1.8}$

were investigated for two reasons. First, any automation of the sampling/testing procedure decreases the labor time involved for each gradation test. This either reduces the technical staff required or frees the current staff for other work. Second, in many aggregate plants, considerable time is spent obtaining and transporting the samples from the belt, stockpile, or other source to the laboratory for testing (fig. 1). If the material could be sampled and tested automatically during production (as it came off a belt, for instance), the time-consuming and sometimes hazardous task of sample gathering would be eliminated.

In the online systems study, researchers surveyed the literature, manufacturers, patent files, and professional societies for sizing techniques currently used within and outside the aggregate industry. A panel of experts, including aggregate producers and trade association representatives, State highway and transportation department representatives, and experts in general-sizing techniques, advised the researchers throughout the study. The panel aided in developing specifications for an online sizing system that would be technically and economically practical for use in plants producing over 91 000 Mg (100,000 tons) per year. The specification developed includes plant coverage, sampling, measurement/analysis, and annual cost.

*Plant coverage:* To meet the plant coverage requirements, the system should be economically applicable to plants producing 90 to 1800 Mg (100 to 2,000 tons) per hour. Although many plants produce less than 90 Mg (100 tons) per hour, their impact on total production is relatively small. For instance, plants producing 23 000 Mg (25,000 tons) or less per

*Figure 1.—Sampling aggregate for gradation testing.*

year of aggregate constitute 36 percent of the sand and gravel operations and 40 percent of the crushed stone operations in the United States. However, these same plants produce only 3.5 percent of the total sand and gravel and 2 percent of the total crushed stone in the United States.

*Sampling:* Sampling should be done where the material is flowing to minimize segregation of the aggregate. Random samples should be taken according to the sample weights listed in American Society for Testing and Materials test method C 136–76. *(3)*

*Measurement/analysis:* Samples must be measured over a range of the No. 200 sieve to 102 mm (4 in). This range, which includes particles more than 1,000 times as large as others, posed a problem for the researchers. Further, if the measurement technique is based on particle number (as opposed to mass), the distribution obtained must be converted to a mass distribution to be comparable with sieve analysis. This means that particles smaller than the No. 200 sieve must be counted (in large numbers) to include the majority of mass passing the No. 200 sieve. In the specification, allowable measurement error is equivalent to that found in present sieve analysis methods from variation in both the sieve openings and sieve fraction weights. The specification also states that a result should be available 20 minutes after the sample is obtained.

*Annual cost:* The specification sets the upper limit of the annual equipment cost at 10 percent of the plant's annual sales. This cost includes the installed purchase price of the equipment amortized over 15 years, as well as loan payments, and operation and maintenance costs. The purchase price is therefore approximately $110,000 for a plant producing 91 000 Mg (100,000 tons) of aggregate per year.

Forty-five potential techniques were identified during the initial survey. Of these, the following six warranted detailed study: Optical array image analysis, hydrocyclone separation, air table (specific gravity) separation, trommel (cylindrical screen) separation, vidicon (television image) counting and measuring, and optical shadowing. Detailed study revealed inherent limitations in four of the techniques. As a result, the final design work focused only on the vidicon system and the optical shadowing technique.

*Vidicon system*

Vidicon cameras previously have been used for counting and measuring material. The vidicon system stores optical images as latent electrostatic charges on the surface of a target layer. This target layer is scanned by a low-energy electron beam that produces a video signal. The video signal then can be used to reproduce the image or record it onto other media. One advantage of a vidicon system is that it is a noncontacting, nondestructive technique. Particle sizes between 0.01 mm and 102 mm (0.0004 in and 4 in) can be detected by choosing the appropriate optical systems.

A vidicon system that analyzes particle size requires several basic components (fig. 2). The controlled light source illuminates the particles for projection onto the vidicon scanner, which minimizes noise and optimizes resolution and speed. The hardware electronics convert the signal from the scanner so that the information can be analyzed by computer and translated to an image on the video monitor.

In an aggregate plant, one data analysis subsystem could be centrally located, with a light source and vidicon camera located on each product line to be monitored. Data from the vidicons would be processed at the same data analysis subsystem to reduce the cost for multiple setups. Also, the service life of the data analysis electronic equipment would be prolonged because the equipment could be housed away from the harsh environment surrounding the production line.

Several problems inherent in any attempt to automate gradation testing were identified during the process of adapting the vidicon system to aggregate gradation measurement.

*Measurement location:* To prevent erroneous measurements caused by coincidence, the sensing device must be located where aggregate particles can be adequately dispersed. The best way to adequately disperse the particles is to remove some material from the main product flow and then control the flow of this sample through the sensing device.

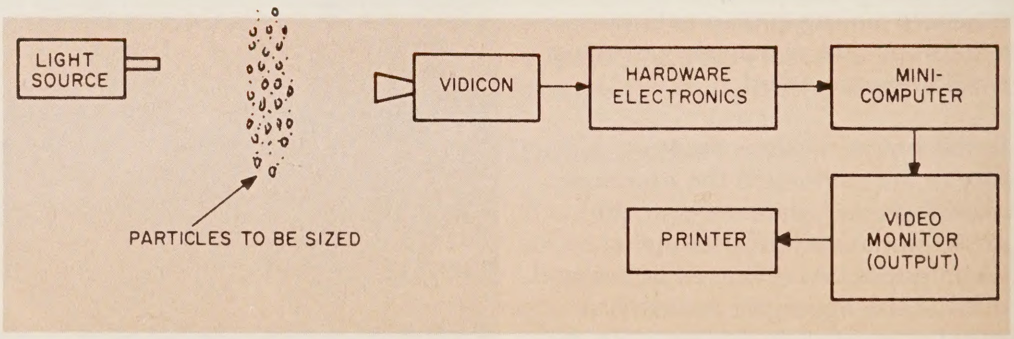


*Figure 2.—Typical vidicon system arrangement.*

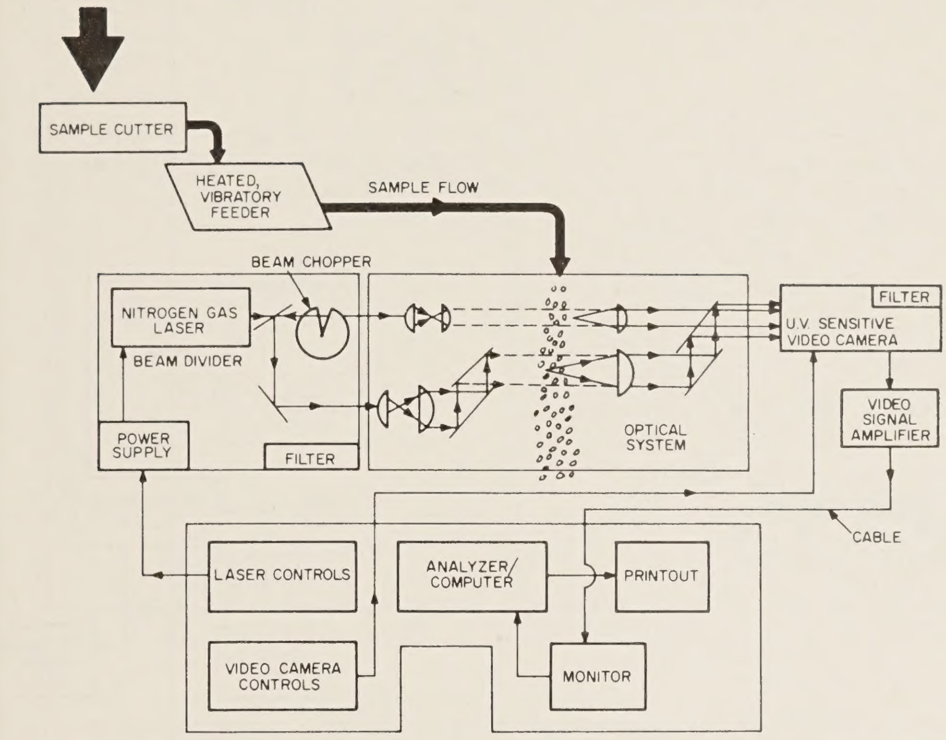


*Figure 3.—Vidicon system functional schematic.*

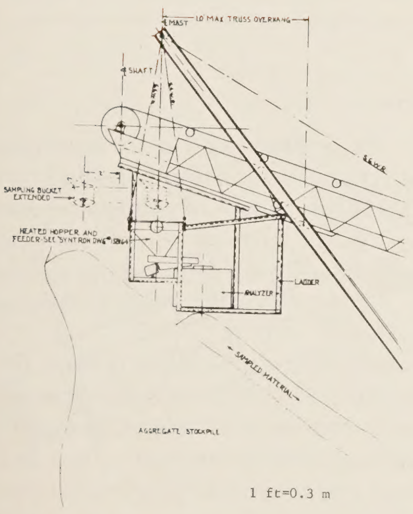


*Figure 4.—Typical arrangement of vidicon system mounting at head of product conveyor.*

*Range of particle sizes:* As previously noted, the range of particle sizes to be measured (No. 200 sieve to 102 mm [4 in]) is an obstacle with any automating technique. To successfully measure a sample with some particles more than 1,000 times as large as others, two lens systems that could time-share the vidicon camera would be needed. Each lens system would measure half of the particle-size range.

*Coincidence and agglomeration:* Particle measurement can be inaccurate if particles coincide or agglomerate. Coincidence occurs when small particles touch and are counted as one particle or when small particles are hidden by larger particles. Coincidence can be minimized by insuring that the aggregate particles to be measured are adequately dispersed.

Agglomeration, occurring when small particles cling to each other or to larger ones, can be minimized by drying the aggregate and breaking down the adhesion. A heated, vibrating feeder positioned in front of the vidicon camera can accomplish this.

*Installation requirements and operating environment:* Continued reliability of a vidicon system in an aggregate plant requires an environmental enclosure, shock mounts, and an optical cleaning device. The video monitor and image analyzer can be located away from the production line in a more favorable environment.

A schematic diagram of the complete vidicon system and support devices necessary for sampling and measuring is shown in figure 3. Figure 4 is a conceptual drawing of how the system might appear installed on a conveyor belt line.

The cost of a vidicon system to monitor a plant with five production lines was estimated. Based on quoted prices for the equipment already in existence and engineering and production estimates for the remaining components, such an installation was estimated at $212,000. Including operation and maintenance costs, the annual cost for operating the system is estimated at $46,000.

The estimated purchase price of the vidicon system described is approximately twice that allowed by the original specification, which probably excludes an additional number of plants from using this system. However, in applicable plants it would be a powerful process control tool, determining gradation on each of five production lines every 20 minutes.

*Optical shadowing*

The second technique investigated is based on optical shadowing. A particle-size distribution transmitter, a commercially available device designed for the mining and mineral industries, sizes particles on a conveyor belt by measuring the difference in reflectivity of the aggregate particles and the shadows among them (figs. 5 and 6). To maximize this difference, the material is illuminated at a low angle to insure shadowing among the particles. The sensor head is positioned above the belt and between two lamps that provide the low angle illumination. The sensor consists of a lens system, two photodetectors, and preamplifiers connected to the data processing unit. The photodetectors scan illuminated areas (particles) and shadow areas (between particles) along a narrow band in the center of the belt. The signal from these optical detectors is converted to an electric analog signal from which a threshold value is determined. Every feature with a value above the threshold is considered a particle and every feature with a value below is considered a space. A particle chord length is determined from the time it takes the particle to pass under the detector, the speed of the belt, and the magnification of the lens system.

Certain characteristics of the device make it more suitable for controlling the aggregate process than for determining gradation. Measurements are made only along a narrow band down the center of the belt. Thus, if any material segregates, the sample will not be representative. Similarly, measurements made only at the upper surface of the material probably will not be representative. Also, the device measures chord length, which may have only an indirect relationship to particle

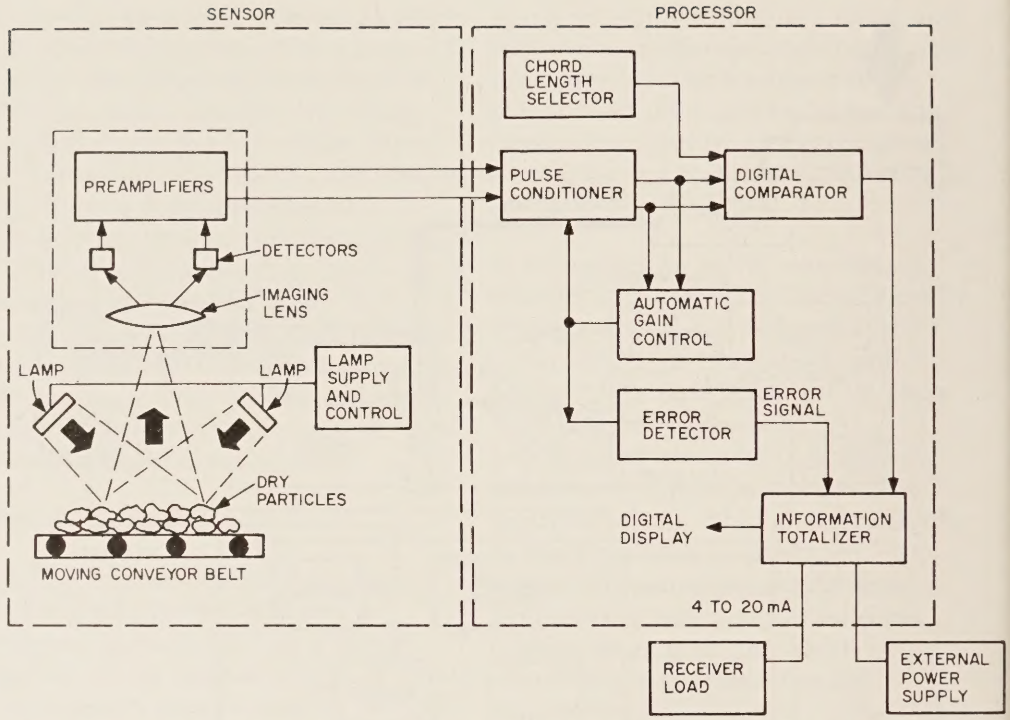


*Figure 5.—Simplified block diagram of particle-size distribution transmitter (optical shadow technique).*

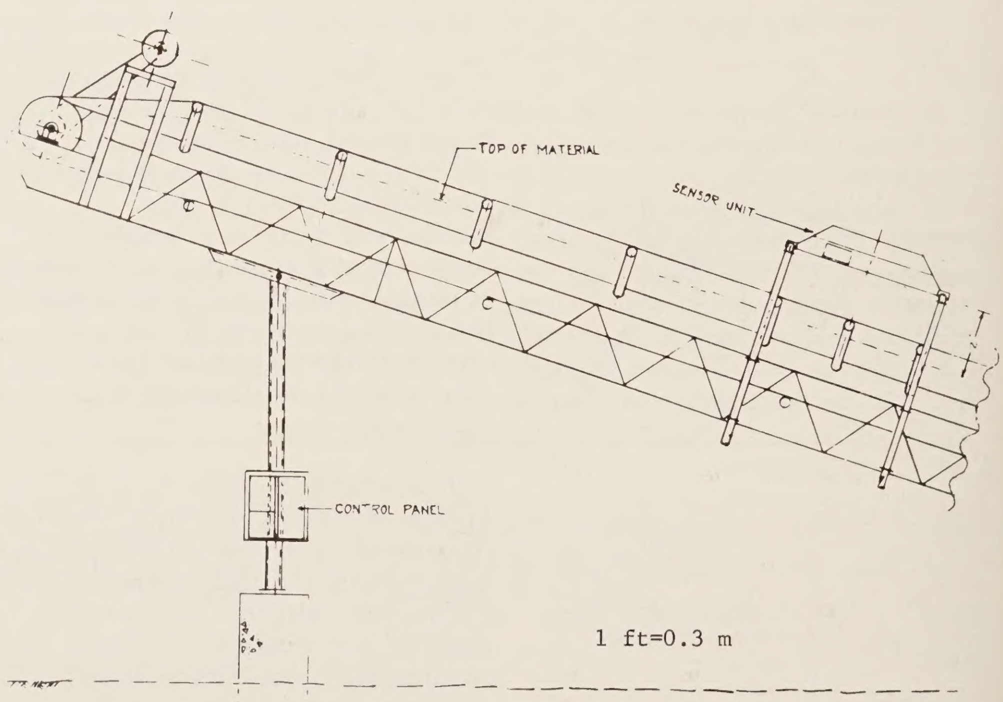


*Figure 6.—Typical arrangement of optical shadow mounting on product conveyor.*

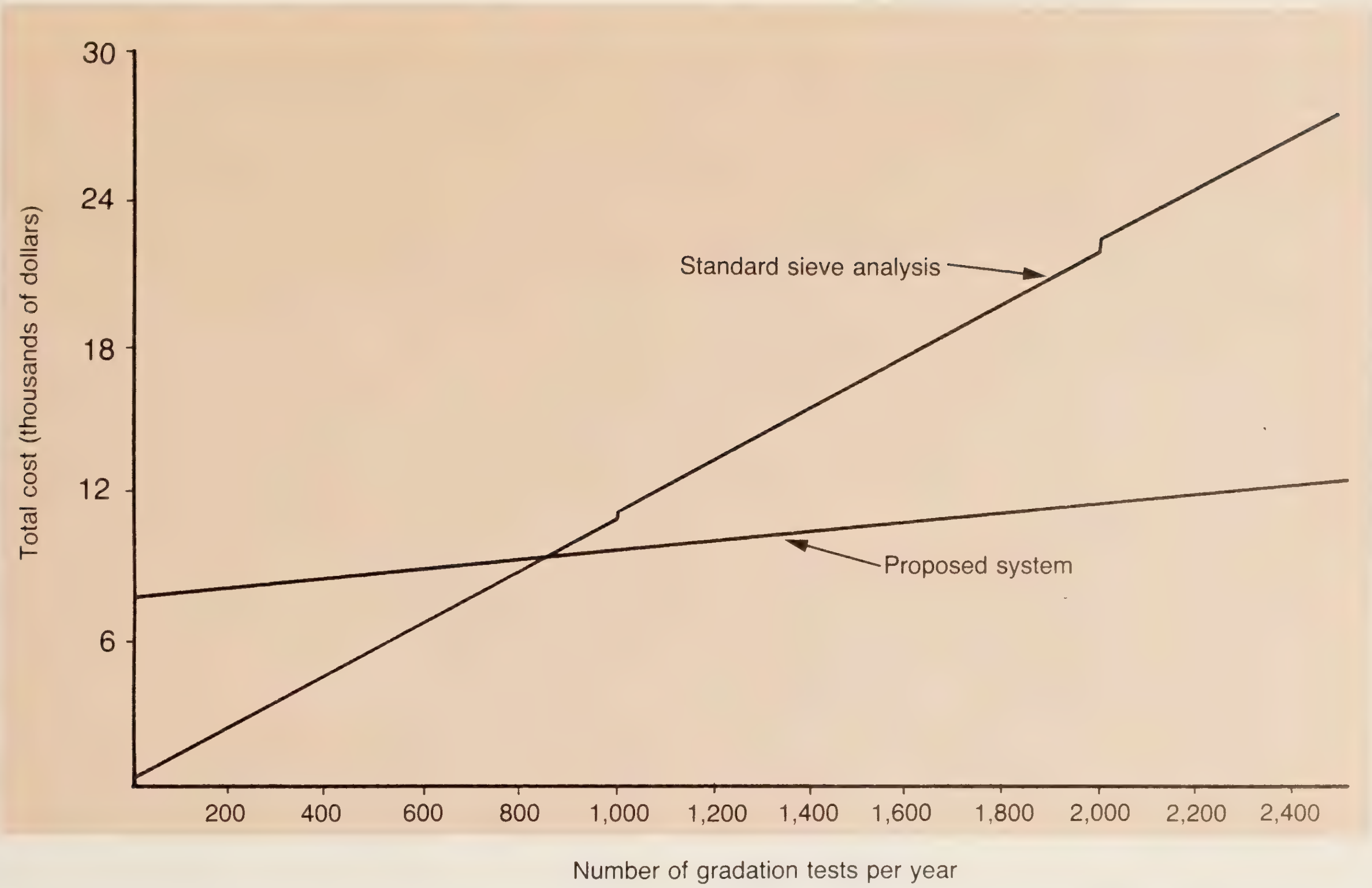


*Figure 7.—Gradation testing cost comparison.*

diameter or projected area for nonspherical particles. Finally, the device cannot measure particles smaller than 3 mm (0.1 in) in diameter because of poor shadowing with this size particle.

An optical shadowing unit costs approximately $12,000. A plant with five production lines would require five units. Including installation, operation, and maintenance costs, a five-unit system would cost approximately $12,500 annually. Gradation determinations for each of the five lines would be supplied every 20 minutes.

Reference 4 summarizes investigations of online systems and includes complete descriptions of the most promising techniques.

**Automated laboratory testing instrument**

A second approach to automatic size analysis for gradation control was to develop a portable laboratory instrument that would reduce both labor and testing times. Because the system would not be tied to the production line, it would be adaptable to various situations. For example, State highway and transportation departments could use it to test samples in their central or district laboratories if the quantity of aggregate samples was sufficient to justify using the instrument.

As in the study of online systems, the problem of accurately measuring a wide range (material finer than the No. 200 sieve to 102 mm [4 in]) of particle sizes was evident. Extensive study indicated that two subsystems were necessary—one for material larger than the No. 4 sieve and one for material smaller than the No. 4 sieve. Prototypes for each of the subsystems are being constructed for FHWA and are expected late in 1981. Both subsystems will pass the aggregate particles between a laser and photodetector arrays. The particles will block some of the light from the arrays and thereby can be measured and counted. Hardware for the two subsystems will differ significantly because of the different range of particle sizes to be measured.

Preliminary cost analyses have determined the number of tests that a laboratory would have to run per year to justify the cost of the proposed system. Figure 7 compares annual costs for the standard sieve analysis procedure with the proposed system as a function of the number of tests run per year. Costs for the standard procedure are based on $10.78 per test for labor, $388 for fixed yearly amortization and maintenance, and $300 per 1,000 tests for screen replacement. The proposed system is projected to cost $60,000 in production. Fixed maintenance plus amortization over 10 years is estimated at $7,800 annually; however, because each test requires only 7.5 minutes of labor, the labor cost is only $2 per test. Figure 7 shows that the proposed system is expected to show cost advantages for laboratories running more than 800 gradation tests per year.

## Economic Analysis of Gradation Testing Programs

FCP Task 4F7 to develop faster, more economical methods of aggregate gradation was not limited to investigations of techniques and equipment. Research also addressed the question of whose personnel—the State highway and transportation department's or the aggregate producer's—can perform process control sampling and testing the most efficiently and economically. State highway and transportation departments often assign an employee to an aggregate plant to test material used in State construction work. The efficiency of this procedure is questionable. An employee of the producer (who is already at the site) could test the material and thus free the State employee for other duties.

Georgia has instituted voluntary producer certification so that producers can maintain their own process control. *(5)* As part of the certification requirements, an aggregate producer must attain the following:

- A quality control program acceptable to the Georgia Department of Transportation (DOT).
- An approved laboratory with certified equipment and personnel.
- Sampling and testing frequencies acceptable to the Georgia DOT.
- Good agreement between the producer's test results and verification testing by the Georgia DOT to monitor these results.

The Georgia DOT estimated it was spending 4.1 cents per Mg (4.5 cents per ton) on control gradation testing before instituting the program. Since the program's introduction, this cost has decreased to about 1.3 cents per Mg (1.4 cents per ton). There are, of course, additional costs to the producer for performing more of the testing. One of the largest aggregate producers in Georgia has estimated this increase at 0.9 to 1.4 cents per Mg (1.0 to 1.5 cents per ton). *(5)* Overall, then, the program is saving 1.4 to 1.9 cents per Mg (1.6 to 2.1 cents per ton) of aggregate. Based on an annual aggregate production of approximately 36 million Mg (40 million tons), the total cost reduction is $400,000 per year.

There are other benefits from the program. Producers have greater flexibility in plant operations because material can be produced and shipped any time in response to consumer needs rather than only when State inspectors are available. Also, because the producer is doing the control testing, a testing schedule that gives the producer optimum control of the product without overtesting can be developed.

A study was undertaken to seek additional information on the most efficient process control program. Onsite visits were made to selected State highway and transportation departments and aggregate producers in those States to study current programs for process control testing. States selected for the study represented the range of programs currently in use—from programs in which a State employee does the majority of testing to programs in which a producer employee does the majority of testing.

Preliminary results from studies of three State programs revealed that a significant part of a State employee's time frequently was spent sampling and transporting samples to the testing laboratory. Also, even in those States with process control testing by the producer, verification testing by the State may still be too frequent to be most cost effective.

Comparisons of combined labor times (hours per 910 Mg [1,000 tons]) devoted to testing by both State personnel and producers reveal differences in efficiency between the three State programs. For example, both States A and B have transferred much of the process control testing to the producer. State A personnel spend 15.4 hours per 910 Mg (1,000 tons) for gradation testing. The aggregate producers in State A spend approximately 2.3 hours per 910 Mg (1,000 tons), bringing the total gradation testing effort to 17.7 hours per 910 Mg (1,000 tons).

State B personnel, on the other hand, devote 2.8 hours per 910 Mg (1,000 tons) to gradation testing; aggregate producers spend 0.8 hours. The total is thus only 3.6 hours spent on gradation testing for every 910 Mg (1,000 tons) of aggregate.

The difference in labor times between State A and State B appears to be in the amount of verification testing performed by the States. The amount and frequency of verification testing by the States should be examined in relation to program efficiency.

Unlike States A and B, State C personnel perform all gradation testing. The State devotes 140.8 hours to gradation testing per 910 Mg (1,000 tons), while producers average only 0.3 hours per 910 Mg (1,000 tons). The combined total is 141.1 hours per 910 Mg (1,000 tons) for gradation testing. A significant portion of the total State time is spent in travel—collecting samples and taking them to the laboratory. However, travel is an integral part of this kind of program and therefore must be included in any comparisons. The excessive time devoted to gradation testing in this traditional program is evident.

The final report on State process control programs is expected in the fall. The report will discuss the data collected and will include a recommended program for the States based on economic analyses of the various programs investigated.

## Summary

Previous research indicates that gradation control during aggregate production is necessary for satisfactory products. The research described in this article was performed to develop less expensive and faster methods of aggregate gradation control testing.

In a survey of current State practices, alternatives to time-consuming standard sieve analysis procedures were evaluated in the laboratory. The alternative procedures included methods to either reduce or eliminate drying times or reduce the number of sieves required. Although the procedures appear to control gradation, field evaluation is necessary. If the procedures are adequate, they could be implemented easily because specialized equipment is not required.

To reduce the high labor costs of gradation testing, efforts are being made to automate the procedure. Attempts have been made to automate the entire sampling and testing process using a system online with the aggregate production. In addition, a laboratory instrument is being designed and built for automating just the gradation test. Further development of the online equipment by FHWA will depend on developmental results of the prototype laboratory instrument.

Various State programs for process control also are being analyzed. The question being addressed is whether it is more economical for State employees or producer employees to do the majority of the process control testing. Preliminary results indicate that testing can be done more efficiently and economically by the producer (with limited verification testing by the State) without losing product quality.

Field trials will be performed for the four shortcut testing techniques presented and the prototype laboratory instrument. These field trials will evaluate how well these approaches work in daily operations.

A final report of the FCP Task 4F7 research will discuss the approaches examined during the task, the conclusions reached, and the benefits from the various approaches. The report will include results of the field trials as well as a discussion of the implications of the economic analysis. States will be encouraged to apply the various approaches through the Highway Planning and Research Program.

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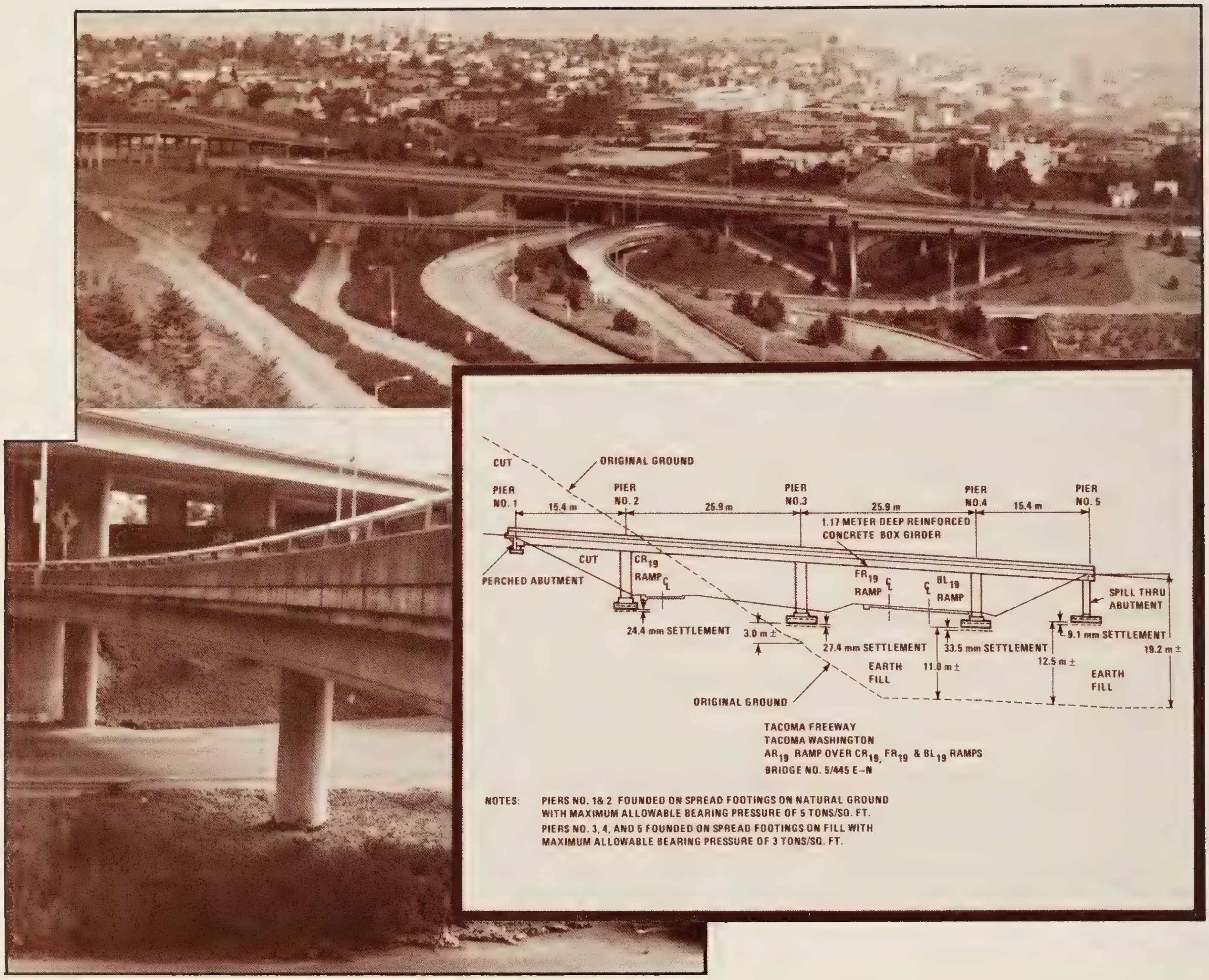
*(5)* Reid H. Brown, "Controlling the Quality of Construction Aggregates Through Process Control," Transportation Research Record No. 691, *Transportation Research Board,* Washington, D.C., January 1978, pp. 1-4.


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


**Stephen W. Forster** is a geologist in the Materials Division, Office of Research, Federal Highway Administration. He is task manager for Task 4F7, "Process Control for Aggregate Production and Use," in the Federally Coordinated Program of Highway Research and Development. Since joining FHWA in 1975, Dr. Forster has worked in the areas of rapid testing of aggregate gradation, skid resistant aggregates, and more durable aggregates for pavements.


# Bridge Movements and Their Effects

by
Lyle K. Moulton and Joseph R. Kula




**This is the first in a series of three articles on tolerable bridge movements. The series was introduced in the last issue of *Public Roads,* vol. 44, No. 1, June 1980. This article discusses the collection and analysis of field data on bridge movements and the ability of bridges to tolerate these movements. It was concluded that many bridges, depending upon kind of span, length of span, and kind of construction material, can tolerate significant levels of differential settlement without sustaining intolerable structural damage. However, the horizontal movement of substructure elements was found to be more critical than vertical movement and must be controlled closely to avoid structural damage.**

## Introduction

A structure type that considers functional, architectural, engineering, and economic aspects usually is selected before buildings, bridges, and other structures are designed. A preliminary design is prepared for the geotechnical engineer with a request for recommendations for founding the structure. Subsurface explorations, sampling, and geotechnical analyses are conducted, in addition to bearing capacity evaluations and estimates of immediate and long term total and differential movements. The estimates are used to decide how the structure should be founded to be safe and economical. Often, one of the major considerations is whether or not the structure can tolerate the estimated total and differential movements.

Much data have been collected and used to establish criteria for tolerable movements of buildings and some industrial structures. *(1–5)*[1] Unfortunately, adequate criteria are not presently available for highway bridges, so foundation selection for bridges is often based on conservative rules of thumb or textbook guidelines, building codes, and specifications. Current American Association of State Highway and Transportation Officials (AASHTO) specifications state: "In general, piling shall be used when footings cannot, at a reasonable expense, be founded on rock or other solid material." *(6)* However, using arbitrary or restrictive guidelines for the tolerable movements of bridges often has led to the unwarranted and uneconomical use of piling or other deep foundations. Under certain circumstances, bridges founded on piles or other deep foundations can move—sometimes substantially. *(7–13)*

To satisfy the need for more rational criteria for the tolerable movements of highway bridges, the Federal Highway Administration (FHWA) set out to develop bridge design criteria that recognize the interaction between the foundation and the bridge structure and establish tolerable bridge movement limits for a range of bridge types. Although this research was divided into several tasks and subtasks, the following three general study categories evolved:

- A state-of-the-art assessment of tolerable bridge movements based on a literature review, an appraisal of existing design specifications, and a compilation and analysis of movement, damage, and tolerance data from bridges in the United States and Canada.
- A series of analytical studies to determine the limits of tolerable foundation movements, with respect to structural integrity and serviceability, for various concrete and steel bridge types.
- A study to develop methodology for optimizing the design of superstructure and substructure systems as an integrated unit, and recommendations for modifying current design specifications.

This article summarizes the first of these studies. The remaining two studies will be discussed in subsequent articles.

[1]Italic numbers in parentheses identify references on page 75.

## Collection and Analysis of Field Data

### Sources of data

In 1967 and 1975, Transportation Research Board (TRB) committees[2] sent questionnaires to highway agencies throughout the United States and Canada to survey bridge movements. *(14–17)* In addition to identification information, the questionnaires requested information on the year the bridge was completed, the kind and number of spans, the kind of abutment, soil and foundation conditions, and estimated or observed movements and their effects on the structure. Some of the highway agencies supplied soils reports and design drawings as well. Overall, information was supplied by 34 States, the District of Columbia, and 4 Canadian provinces.

As a first step in the current investigation, various highway agencies were asked to supply additional information, including boring logs, settlement data, as-built plans, and tolerance ratings, for those bridges that had been included in the 1967 and 1975 surveys. Information was also requested on any bridges not included in the 1967 and 1975 survey responses that had experienced movement. Supplementary data were supplied by 15 States and from an FHWA study on 28 bridges in the State of Washington.[3]

Data are now available on 200 bridges that have experienced some movement, and as-built plans have been obtained for 94 of these bridges. At present, the data from 180 bridges have been analyzed.

### Assumptions and definitions

The data have certain limitations. Because a substantial amount of the data was obtained by questionnaires, the quality of the data depends on the information requested in the questionnaire and the completeness and accuracy of the responses. In some instances the responses were incomplete or unclear, and there was a general lack of

[2]The 1967 survey was conducted by TRB Committee SGF-B3 and the 1975 survey was conducted by TRB Committee A2K03.

[3]"An Evaluation of the Performance and Cost Effectiveness of the Use of Spread Footings in Fill Embankments for the Support of Highway Bridge Abutments in the State of Washington," thesis by S. J. Seguirant for Master of Science in Civil Engineering, University of Washington, Seattle, Wash., 1979. "A Field Evaluation of Highway Bridge Abutments Supported by Spread Footings on Compacted Fill," Report No. FHWA/RD-80/044, *Federal Highway Administration*, Washington, D.C. Not yet published.

common terminology. Consequently, definitions and simplifying assumptions were adopted to generalize the data for classification and analysis:[4]

*Vertical displacement*—When applied to structural damage, this term includes the raising or lowering of the superstructure above or below planned grade or a sag or heave in the deck. Structures requiring shimming or jacking as well as truss structures with increased camber are also included.

*Horizontal displacement*—When applied to structural damage, this term includes the misalinement of bearings and the superstructure or beams jammed against the abutments. Also included in this category are bridges whose superstructure extended beyond the abutment or whose beams required cutting, bridges having an open space between the abutment or pier seats and the beams, and bridges experiencing horizontal movement of the floor system.

*Tolerable*—The subjectivity of this term is one reason for the lack of generally accepted tolerable movement criteria. The TRB committee defined intolerable movement as follows: "Movement is *not* tolerable if damage requires costly maintenance and/or repairs *and* a more expensive construction to avoid this would have been preferable." *(16)* For the sake of consistency, this definition was adopted for the FHWA study.

**Table 1.—Substructure data**

| General soil conditions | Number of bridges |
|---|---|
| Fine grained soils | 62 |
| Granular soils | 17 |
| Fine grained soils over granular soils | 16 |
| Granular soils over fine grained soils | 25 |
| Interlayered/intermixed soils | 43 |
| Bedrock | 7 |
| Permafrost soils | 2 |
| Soil conditions not given | 8 |
| Foundation type | |
| Spread footings | 48 |
| Piles | 80 |
| Abutments on spread footings/piers on piles | 14 |
| Abutments on piles/piers on spread footings | 19 |
| Abutments and piers on both spread footings and piles | 10 |
| Abutments on caissons/piers on spread footings | 2 |
| Foundation type not given | 5 |
| Abutment type | |
| Full height | 36 |
| Perched | 79 |
| Spill-through | 35 |
| Perched, spill-through | 3 |
| Abutment type not given or unknown | 27 |
| Height of approach embankments | |
| *Metres* | |
| Cut | 2 |
| 0 to 3 | 11 |
| 3 to 6.1 | 35 |
| 6.1 to 9.1 | 61 |
| 9.1 to 12.2 | 21 |
| 12.2 to 15.2 | 10 |
| 15.2 to over 30.5 | 10 |
| Approach heights vary | 8 |
| Approach height not given | 22 |

1 m=3.3 ft

## Data analysis methods

The field data were analyzed to delineate trends of bridge foundation movements, their effects, and the ability of the bridge to tolerate these movements. Three separate analyses were conducted, each with a different methodology.

The first analysis involved investigating the influence of substructure variables—general soil condition, foundation type (spread footings or piles), abutment type (full height, perched, or spill-through), and height of approach embankment—on bridge abutments. Table 1 summarizes the substructure data that were incorporated into this analysis. Combinations of these variables were considered to determine if they result in foundation movement.

Additional variables—span type (simply supported or continuous), and the abutment-embankment-pier geometry—were investigated to determine their influence on pier movements. Table 2 summarizes the superstructure data that were incorporated into this analysis. Again, the influence of each of the selected variables was considered separately and in selected combinations.

The second analysis involved investigating the influence of bridge foundation movements on the bridge structure to determine what kinds and magnitudes of movements most frequently cause structural damage. Kind of movement (vertical only, horizontal only, or vertical and horizontal), magnitude of movement (maximum differential vertical movement between two successive abutments or piers and maximum horizontal movement), span type, kind of structural material (steel or concrete), number of spans, and abutment type were considered. Table 3 summarizes the kinds of structural damage and the bridges that experienced them. Many of these bridges experienced multiple damaging effects.

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

**Table 2.—Superstructure data**

| Span type | Number of bridges |
|---|---|
| Simple | 71 |
| Continuous | 45 |
| Rigid frame | 8 |
| Cantilever | 4 |
| Miscellaneous or not given | 52 |
| **Structural material** | |
| Steel | 73 |
| Concrete | 55 |
| Steel and concrete | 6 |
| Material type not given | 46 |
| **Number of spans** | |
| One | 20 |
| Two | 16 |
| Three | 61 |
| Four | 30 |
| Five | 12 |
| More than five | 35 |
| Number of spans not given | 6 |

**Table 3.—Structural damage**

| Structural damage | Number of bridges |
|---|---|
| Abutment damage | 47 |
| Pier damage | 13 |
| Vertical displacement | 24 |
| Horizontal displacement | 41 |
| Superstructure distress | 68 |
| Damage to rails, curbs, sidewalks, parapets | 18 |
| Bearing damage | 38 |
| Poor riding quality | 11 |
| Not given or corrected during construction | 8 |
| None | 26 |

The third analysis involved investigating the various bridge structures' tolerance to movements. Kind of structural damage, kind of movement, magnitude of movement (maximum differential vertical movement between successive units of the substructure, maximum angular distortion, and maximum horizontal movement), span type, kind of structural material, number of spans, and abutment type were considered.

## Influence of Substructure Variables on Foundation Movements

### Abutment movements

Three hundred thirty-nine abutments were analyzed. Table 4 summarizes data for the 248 abutments that experienced movement. These data show that most of the abutments moved vertically, one-half of them moved horizontally, and almost one-third of them simultaneously moved vertically and horizontally. The magnitudes of the vertical movements were greater than the horizontal movements. In many instances the abutments moved inward until they jammed against the beams or girders, which acted as struts and prevented further horizontal movement. Table 4 also shows that abutment movements tended to be larger and more variable for those abutments that moved both vertically and horizontally.

Table 5 summarizes abutment movements by abutment type. Because perched and spill-through abutments had vertical and horizontal movements over a wider range than did the full height abutments, foundation system analysis and design for perched and spill-through abutments needs further study. The conventional design advantages of using perched or spill-through abutments rather than full height abutments may be considerably offset by the larger probability of intolerable movements occurring.

Abutments founded on spread footings moved more than abutments founded on piles. Table 6 shows that abutments on spread footings experienced slightly larger vertical movements but smaller horizontal movements than did those founded on piles. These same trends were

**Table 4.—Abutment movements**

| Movement | Number of abutments | Percent moved | Range | Average | Standard deviation |
|---|---|---|---|---|---|
| | | | *Millimetres* | *Millimetres* | *Millimetres* |
| All types | 248 | 100 | | | |
| Vertical | 197[1] | 79.4 | 2.5–1280.1 | 154.9 | 193 |
| Horizontal | 125 | 50.4 | 7.6– 457.2 | 83.8 | 81.3 |
| Vertical and | 76 | 30.6 | 10.2–1280.1 | 210.8 | 254 |
| horizontal | | | 7.6– 457.2 | 88.9 | 94 |

[1] Two abutments that raised vertically are not included.

1 mm=3.9 × $10^{-2}$ in

Table 5.—Movements by abutment type

| Abutment | Movement | Frequency Number of abutments | Percent moved | Range | Average | Standard deviation |
|---|---|---|---|---|---|---|
| | | | | *Millimetres* | *Millimetres* | *Millimetres* |
| Full height | All types | 62 | 100 | | | |
| | Vertical | 55[1] | 88.7 | 7.6–518.2 | 91.4 | 94 |
| | Horizontal | 30 | 48.4 | 7.6–203.2 | 61 | 55.9 |
| | Vertical and | 25 | 40.3 | 12.7–518.2 | 109.2 | 119.4 |
| | horizontal | | | 7.6–203.2 | 66 | 61 |
| Perched | All types | 118 | 100 | | | |
| | Vertical | 94 | 79.7 | 2.5–965.2 | 170.2 | 208.3 |
| | Horizontal | 58 | 49.2 | 7.6–457.2 | 88.9 | 86.4 |
| | Vertical and | 34 | 28.8 | 20.3–965.2 | 274.3 | 279.4 |
| | horizontal | | | | 99.1 | 99.1 |
| Spill-through | All types | 37 | 100 | | | |
| | Vertical | 26 | 70.3 | 12.7–609.6 | 200.7 | 193 |
| | Horizontal | 19 | 51.4 | 25.4–228.6 | 83.8 | 66 |
| | Vertical and | 8 | 21.6 | 12.7–396.2 | 144.8 | 157.5 |
| | horizontal | | | 25.4–139.7 | 48.3 | 38.1 |

[1] Includes two full height abutments that raised 76.2 mm vertically.

1 mm=$3.9 \times 10^{-2}$ in

observed when the data were analyzed by abutment type; however, the differences in average movements between spread footing and pile foundations were not large. This finding, coupled with the relatively large number of pile foundations that did move, tends to refute the generally accepted notion of the superiority of pile foundations in limiting abutment movements, particularly for perched abutments in fills. These findings also contradict those reported for Ohio bridges. *(14)*

Vertical movement was frequent for abutments founded on spread footings on predominantly fine grained soils. Horizontal movement occurred most often for pile foundations in fine grained soils overlying granular soils. The largest vertical movements occurred for abutments on spread footings in fine grained soils and on pile foundations in granular soils overlying fine grained soils. The largest horizontal movements occurred for pile foundations and spread footings in fine grained soils.

Approach embankment heights did not correlate significantly with the frequency and magnitude of abutment movements, which agrees with the findings for Ohio bridges. *(14)* As might be expected, frequency and magnitude of vertical movements increased with an increase in height of approach embankments. However, additional analyses of embankment height, considering abutment type, foundation type, and soil conditions, did not show meaningful trends.

**Pier movements**

The pier movements analysis showed that piers move less often than abutments. Only 31 percent of the 636 piers analyzed moved, compared with 73 percent of abutments. Table 7 shows that vertical movements for piers were considerably less than for abutments. Unlike the abutment movements, horizontal pier movements were larger than the vertical movements. Also, unlike abutment movements, the magnitudes of movements for piers with both vertical and horizontal movements were smaller than for piers with movement in only one direction.

Although there were more piers founded on piles than on spread footings, about half of those that moved were founded on spread footings. This suggests that founding piers on piles is more successful than founding abutments on piles, particularly for perched and spill-through abutments. Table 8 shows that the average magnitudes of movements were similar for both spread footing and pile foundations. However, the range of vertical movements was wider for the piers on spread footings than for piers on piles.

Few trends were evident on the relationship of pier movements with soils and foundation types. Movements were most frequent for spread footings on fine grained soils. A surprising finding was that over 33 percent of 95 piers on piles in granular soils moved, compared with just 17.5 percent of 206 piers on piles in fine grained soils.

Piers in or near the toe of approach embankments moved almost twice as often as piers away from the embankment. The data showed that, contrary to what might be expected, vertical movements were slightly larger for piers away from the embankment, with an average movement of 79 mm (3.1 in), compared with 66 mm (2.6 in) for piers in or near the embankment. Horizontal movements, however, were significantly larger for piers in or near the embankment, with an average movement of 127 mm (5 in), compared with only 28 mm (1.1 in) for piers away from the embankment. These data indicate that when designing bridge piers in or near the toe of embankments, more consideration needs to be given to the increased horizontal stresses in these areas.

## Influence of Foundation Movements on Bridge Structures

As shown in table 3, the most frequently occurring kinds of structural damage were superstructure distress, abutment damage, horizontal displacement, and bearing damage. Table 9 shows that bridges having only abutment movements had a high frequency of abutment damage and superstructure distress. Superstructure distress also occurred frequently for bridges with pier movement only. Table 10 shows that more kinds of structural damage occur for those bridges with simultaneous vertical and horizontal movements. Horizontal displacement, abutment damage, and superstructure distress all occurred frequently for bridges with both vertical and horizontal movements. In contrast, structures experiencing vertical movement only had the lowest frequency of structural damage, with 20 structures having no damage at all.

**Table 6.—Abutment movements by foundation type**

| Foundation | Movement | Frequency: Number of abutments | Frequency: Percent moved | Range | Average | Standard deviation |
|---|---|---|---|---|---|---|
| | | | | *Millimetres* | *Millimetres* | *Millimetres* |
| Spread | All types | 120 | 100 | | | |
| footings | Vertical | 115 | 95.8 | 2.5– 965.2 | 162.6 | 188 |
| | Horizontal | 44 | 36.7 | 7.6– 355.6 | 76.2 | 68.6 |
| | Vertical and | 39 | 32.5 | 12.7– 965.2 | 254 | 264.2 |
| | horizontal | | | 7.6– 355.6 | 76.2 | 71.1 |
| Piles | All types | 123 | 100 | | | |
| | Vertical | 77 | 62.6 | 7.6–1280.1 | 149.9 | 205.7 |
| | Horizontal | 79 | 64.2 | 12.7– 457.2 | 88.9 | 88.9 |
| | Vertical and | 35 | 28.5 | 12.7–1280.1 | 165.1 | 246.4 |
| | horizontal | | | 10.2– 457.2 | 104.1 | 114.3 |

1 mm=3.9 × $10^{-2}$ in

**Table 7.—Pier movements**

| Movement | Frequency: Number of piers | Frequency: Percent moved | Range | Average | Standard deviation |
|---|---|---|---|---|---|
| | | | *Millimetres* | *Millimetres* | *Millimetres* |
| All types | 196[1] | 100 | | | |
| Vertical | 137 | 73.5 | 2.5–1066.8 | 63.5 | 106.7 |
| Horizontal | 73 | 37.2 | 2.5– 508 | 101.6 | 134.6 |
| Vertical and | 19 | 9.7 | 12.7– 348 | 61 | 86.4 |
| horizontal | | | 15.2– 508 | 94 | 147.3 |

[1] Includes seven piers that raised vertically. These piers are not included in the total with vertical movement.

1 mm=3.9 × $10^{-2}$ in

This same trend was evident with magnitudes of movements—even moderate differential vertical movements produced a low incidence of structural damage. Of the 41 bridges with maximum differential vertical settlements of less than 102 mm (4 in), 20 experienced no damage. Most of the remaining 21 bridges experienced primarily abutment damage—minor cracking and opening or closing of construction joints—and minor superstructure distress. However, some bridges with larger differential vertical movements surprisingly did not experience abutment damage. For differential vertical movements larger than 102 mm (4 in), superstructure distress was predominant. Vertical displacement and poor riding quality were common for differential vertical movements of 203 mm (8 in) and greater. However, only 11 of the 180 bridges considered had poor riding quality. This matter will be discussed later in this article.

It was more difficult to correlate structural damage with magnitudes of substructure movements for those bridges where vertical and horizontal movements occurred simultaneously because of the possible interaction of the two movements. However, some observations can be made. Abutment damage was more frequent for bridges with both vertical and horizontal movements less than 102 mm (4 in) than for bridges with movement in only one direction. More than one-half of the 29 bridges experiencing movement in both directions had abutment damage when the magnitude of the vertical component was less than 102 mm (4 in). Similarly, more than one-half of the 39 bridges had abutment damage when the magnitude of the horizontal component of movement was less than 102 mm (4 in). In addition, horizontal displacement and bearing damage were more frequent for bridges having both vertical and horizontal movements (fig. 1). A detailed review of the actual causes of the kinds of distress in the bridges revealed that the horizontal component of the movement was usually responsible for the damage. Thus, as suggested earlier, horizontal movements appear to be much more critical than differential vertical settlement in causing most kinds of structural distress. This agrees with findings in references 16 and 17.

**Table 8.—Pier movements by foundation type**

| Foundation | Movement | Number of piers | Percent moved | Range | Average | Standard deviation |
|---|---|---|---|---|---|---|
| | | | | *Millimetres* | *Millimetres* | *Millimetres* |
| Spread | All types | 92 | | | | |
| footings | Vertical | 67 | 72.8 | 5.1–1066.8 | 68.6 | 137.2 |
| | Horizontal | 36 | 39.1 | 7.6– 508 | 104.1 | 142.2 |
| | Vertical and | 11 | 12 | 7.6– 614.7 | 170.2 | 182.9 |
| | horizontal | | | 5.1– 228.6 | 109.2 | 154.9 |
| Piles | All types | 93[1] | | | | |
| | Vertical | 56 | 60.2 | 2.5– 355.6 | 63.5 | 73.7 |
| | Horizontal | 38 | 40.9 | 2.5– 457.2 | 101.6 | 127 |
| | Vertical and | 6 | 6.8 | 12.7– 348 | 111.8 | 129.5 |
| | horizontal | | | 25.4– 86.4 | 58.4 | 27.9 |

[1] Includes seven piers that raised vertically. These are not included for vertical movement.

1 mm=3.9 × $10^{-2}$ in

Bridges that experienced horizontal movement only or simultaneous horizontal and differential vertical movement had a high frequency of structural damage, even for small horizontal movements. This suggests that horizontal movements are more critical than vertical movements in causing structural damage. For those structures with horizontal movements only, small movements (less than 51 mm [2 in]) caused superstructure distress in more than two-thirds of the bridges. The bearings were also damaged in more than one-third of these bridges. Abutment damage and horizontal displacement occurred more frequently for horizontal movements in excess of 102 mm (4 in).

Table 11 shows that abutment damage occurred most frequently in simple span bridges but rarely in continuous bridges. Superstructure distress was the most frequent structural damage for continuous bridges and the second most frequent for simple span bridges.

**Table 9.—Structural damage from moving substructure elements**

| Structural damage | Moving element: Abutment only, Number of bridges | Abutment only, Percent of bridges | Pier only, Number of bridges | Pier only, Percent of bridges | Abutment and pier, Number of bridges | Abutment and pier, Percent of bridges |
|---|---|---|---|---|---|---|
| Abutment damage | 33 | 36.3 | 2 | 7.1 | 11 | 22 |
| Pier damage | 1 | 1.1 | 5 | 17.9 | 6 | 12 |
| Vertical displacement | 12 | 13.2 | 5 | 17.9 | 6 | 12 |
| Horizontal displacement | 21 | 23.1 | 6 | 21.4 | 13 | 26 |
| Superstructure distress | 37 | 40.7 | 9 | 32.1 | 19 | 38 |
| Damage to rails, curbs, sidewalks, parapets | 10 | 11 | 3 | 10.7 | 3 | 6 |
| Bearing damage | 22 | 24.2 | 4 | 14.3 | 9 | 18 |
| Poor riding quality | 8 | 8.8 | 2 | 7.1 | 1 | 2 |
| Not given or corrected during construction | 4 | 4.4 | 2 | 7.1 | 1 | 2 |
| None | 6 | 6.6 | 0 | 0 | 16 | 32 |
| Total number of bridges | 91 | | 28 | | 50 | |

**Table 10.—Structural damage from movements**

| Structural damage | Movement: Vertical, Number of bridges | Vertical, Percent of bridges | Horizontal, Number of bridges | Horizontal, Percent of bridges | Vertical and horizontal, Number of bridges | Vertical and horizontal, Percent of bridges |
|---|---|---|---|---|---|---|
| Abutment damage | 13 | 18.8 | 11 | 26.8 | 22 | 37.3 |
| Pier damage | 2 | 2.9 | 4 | 9.8 | 6 | 10.2 |
| Vertical displacement | 12 | 17.4 | 2 | 4.9 | 9 | 15.3 |
| Horizontal displacement | 3 | 4.3 | 12 | 29.3 | 25 | 42.4 |
| Superstructure distress | 21 | 30.4 | 22 | 53.7 | 22 | 37.3 |
| Damage to rails, curbs, sidewalks, parapets | 4 | 5.8 | 2 | 4.9 | 10 | 16.9 |
| Bearing damage | 1 | 1.4 | 16 | 39 | 18 | 30.5 |
| Poor riding quality | 7 | 10.1 | 0 | 0 | 4 | 6.8 |
| Not given or corrected during construction | 2 | 2.9 | 4 | 9.8 | 1 | 1.7 |
| None | 20 | 29 | 0 | 0 | 2 | 3.4 |
| Total number of bridges | 69 | | 41 | | 59 | |

Bearing damage was also frequent for continuous bridges but occurred in few of the simply supported bridges.

The analysis of structural distress, in terms of moving foundation elements, indicated differences between simple and continuous bridges. For the 33 simply supported bridges with abutment movement only, abutment damage and superstructure distress were the most common kinds of structural damage. However, for the 23 continuous bridges with abutment movement only, bearing damage was most frequent. Horizontal displacement and superstructure distress each occurred in over 30 percent of these bridges. However, for bridges with both abutment and pier movement, those 26 bridges with simple spans had a high occurrence of horizontal displacement and superstructure distress. For the 12 continuous bridges having both abutment and pier movement, the most prominent structural damage was superstructure distress.

Sample groups for structures with pier movement only were too small to validly compare. For some unknown reason, bridges with no structural damage were more frequent where both abutments and piers moved, regardless of span type. For both kinds of span, the most frequent and serious types of structural damage were related to horizontal movements.

Table 12 summarizes the frequency of bridge damage in terms of structural material. These data show that

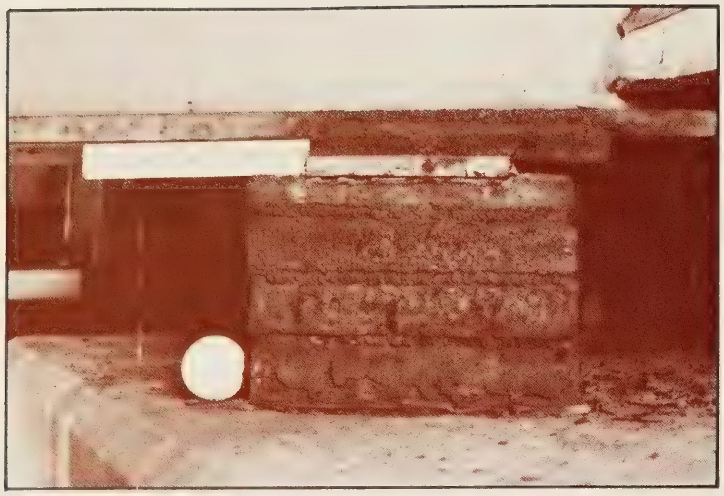


*Figure 1.—Horizontal and vertical movements.*

superstructure distress was reported much more often for concrete bridges than for steel bridges. Slightly more than one-half of the 55 concrete bridges experienced superstructure distress compared with less than one-quarter of the 73 steel bridges. However, the steel bridges had more abutment damage, horizontal displacement, and bearing damage. Overall, steel bridges, particularly those with differential vertical movement only (fig. 2), had less frequent and severe structural damage than did the concrete bridges. Of the 29 steel bridges that experienced only vertical movements, 41.4 percent had no noticeable damage, while more than one-half of the 23 concrete bridges with the same type of movement experienced superstructure distress. Over two-thirds of the steel bridges that had vertical movements of both abutments and piers experienced no structural damage. Again, it was found that even small horizontal movements—51 mm (2 in) or less—caused more frequent and severe structural damage than did much larger differential vertical movements, regardless of the structural material.

Table 13 shows the frequency of structural damage in terms of abutment type. These data show that bridges on full height abutments had the most abutment damage, but the least superstructure distress, bearing damage, and vertical and horizontal displacement. Although bridges on perched abutments had more serious structural damage, they experienced less structural damage in general. This is somewhat of a paradox; as reported earlier, perched abutments experienced a larger and wider range of movements than did full height abutments. However, a detailed examination of the data revealed that primarily differential vertical abutment movements of less than 102 mm (4 in) caused no damage to these bridges with perched abutments. The most damage was caused primarily by horizontal movements between 25 mm (1 in) and 102 mm (4 in), especially when these horizontal movements were accompanied by differential vertical movements greater than 102 mm (4 in). The high vertical movements experienced by the spill-through abutments (table 5) were largely responsible for the high incidence of superstructure distress.

## Tolerance of Bridges to Foundation Movements

Movements were considered tolerable for slightly over one-half of the 154 bridges studied. Table 14 shows that of all the structural damage associated with tolerable foundation movements, abutment damage and superstructure distress occurred most frequently. In most instances, the reported damage involved minor cracking, the opening or closing of construction joints in the abutments, and cracking and spalling of concrete decks. The foundation movements of the 28 bridges that experienced no structural damage obviously were considered tolerable, accounting for 35 percent of the total structures with tolerable movements.

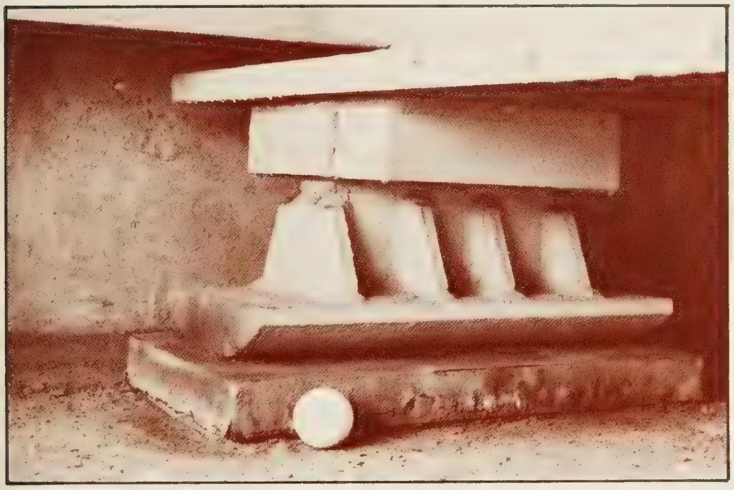
Almost one-half of the 74 bridges with intolerable movements had superstructure distress. Horizontal displacement and bearing damage also occurred frequently. In addition, over 25 percent of those bridges with intolerable movements had abutment damage. As expected, a larger number of bridges having intolerable movements had multiple structural damage than did the bridges having tolerable movements. The most frequent combinations of intolerable structural damage were superstructure distress, horizontal displacement, and bearing damage. The kinds of structural damage occurring most frequently were related to horizontal

**Table 11.—Relationship of structural damage to span type**

| Structural damage | Span type: Simple — Number of bridges | Simple — Percent of bridges | Continuous — Number of bridges | Continuous — Percent of bridges |
|---|---|---|---|---|
| Abutment damage | 23 | 32.4 | 5 | 11.1 |
| Pier damage | 5 | 7 | 6 | 13.3 |
| Vertical displacement | 9 | 12.7 | 7 | 15.6 |
| Horizontal displacement | 15 | 21.1 | 9 | 20 |
| Superstructure distress | 20 | 28.2 | 17 | 37.8 |
| Damage to rails, curbs, sidewalks, parapets | 5 | 7 | 9 | 20 |
| Bearing damage | 10 | 14.1 | 13 | 28.9 |
| Poor riding quality | 2 | 2.8 | 6 | 13.3 |
| Not given or corrected during construction | 2 | 2.8 | 4 | 8.9 |
| None | 18 | 25.4 | 7 | 15.6 |
| Total number of bridges | 71 | | 45 | |

**Table 12.—Frequency of bridge damage by material type**

| Structural damage | Steel: Number of bridges | Steel: Percent of bridges | Concrete: Number of bridges | Concrete: Percent of bridges |
|---|---|---|---|---|
| Abutment damage | 24 | 32.9 | 16 | 29.1 |
| Pier damage | 4 | 5.5 | 5 | 9.1 |
| Vertical displacement | 12 | 16.4 | 9 | 16.4 |
| Horizontal displacement | 20 | 27.4 | 12 | 21.8 |
| Superstructure distress | 17 | 23.3 | 28 | 50.9 |
| Damage to rails, curbs, sidewalks, parapets | 5 | 6.8 | 9 | 16.4 |
| Bearing damage | 19 | 26 | 10 | 18.2 |
| Poor riding quality | 4 | 5.5 | 3 | 5.5 |
| Not given or corrected during construction | 3 | 4.1 | 2 | 3.6 |
| None | 15 | 20.5 | 7 | 12.7 |
| Total number of bridges | 73 | | 55 | |



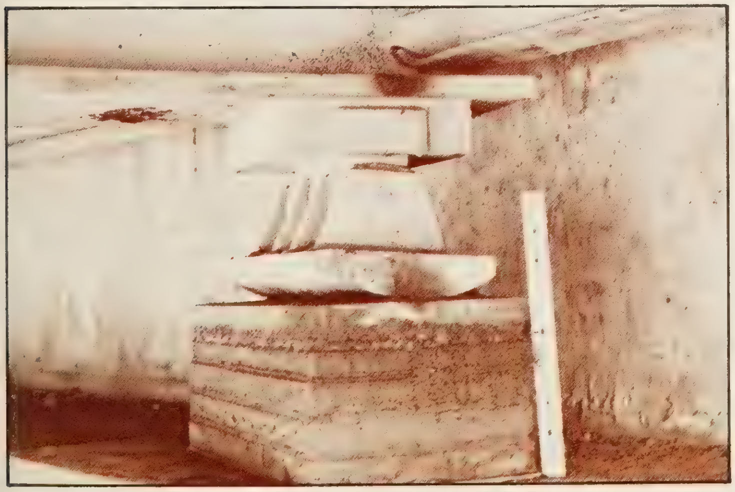


*Figure 2.—Normal bearing device (above), and a steel girder bridge supported by a fabricated bearing rocker that has been shimmed 152 mm (6 in) (below).*

movements or horizontal movements combined with vertical movements. The most frequently occurring sequence of events was the inward horizontal movement of abutments, which jammed the beams or girders against the back wall of the abutments, closed the expansion joints in the deck, and seriously damaged the bearings.

Because of poor riding quality on approaches to many bridges, riding quality was initially identified as one of the major areas of emphasis in the evaluation of tolerable bridge movements. However, poor riding quality was reported for only 11 actual bridge structures, and was intolerable for 10 of these. Of these 10 bridges, maximum differential vertical settlement ranged from 61 mm to 889 mm (2.4 in to 35 in), with an average of 353 mm (13.9 in). The maximum longitudinal angular distortion (maximum differential vertical settlement divided by the span length) ranged from 0.0077 to 0.063, with an average of 0.0203. Even the smallest of these values is larger than what reasonably might be expected to be tolerable from either a stress or serviceability standpoint. In other words, data indicate that the foundation movements would become intolerable for some other reason before creating intolerable rider discomfort. Consequently, riding quality probably will not be seriously considered when establishing tolerable movement criteria for highway bridges. This finding was based on static analysis and can change if dynamic aspects are found to govern this situation.

Intolerable movements generally were substantially larger than tolerable movements (table 15). Table 16 shows that moderate magnitudes of differential vertical movements were usually tolerable, while simultaneous vertical and horizontal movements were usually intolerable. All 28 of the differential vertical movements less than 51 mm (2 in) and 95 percent of those less than 102 mm (4 in) were tolerable. Although some larger differential vertical movements were tolerable, generally the tolerance to differential vertical movements decreased significantly for values over 102 mm (4 in). Only 50 percent of the differential vertical movements between 102 mm (4 in) and 203 mm (8 in) and 18 percent of those over 203 mm (8 in) were tolerable. Generally, maximum horizontal movements less than 51 mm (2 in) were tolerable and maximum horizontal movements of 51 mm (2 in) and greater were intolerable. Furthermore, table 16 shows that horizontal movements less than 51 mm (2 in) were tolerable in only 55 percent of the bridges when accompanied by differential vertical movements. Detailed data analysis showed that for simultaneous horizontal and vertical movements, horizontal movements were tolerable only when their magnitudes were 25 mm (1 in) and less.

Table 13.—Frequency of bridge damage by abutment type

| Structural damage | Abutment | | | | | |
|---|---|---|---|---|---|---|
| | Full height | | Perched | | Spill-through | |
| | Number of bridges | Percent of bridges | Number of bridges | Percent of bridges | Number of bridges | Percent of bridges |
| Abutment damage | 26 | 63.4 | 11 | 12.8 | 6 | 19.4 |
| Pier damage | 5 | 12.2 | 6 | 7 | 3 | 9.7 |
| Vertical displacement | 2 | 4.9 | 14 | 16.3 | 4 | 12.9 |
| Horizontal displacement | 4 | 9.8 | 21 | 24.4 | 10 | 32.3 |
| Superstructure distress | 12 | 29.3 | 30 | 34.9 | 17 | 54.8 |
| Damage to rails, curbs, sidewalks, parapets | 4 | 9.8 | 8 | 9.3 | 5 | 16.1 |
| Bearing damage | 5 | 12.2 | 22 | 25.6 | 5 | 16.1 |
| Poor riding quality | 1 | 2.4 | 7 | 8.1 | 1 | 3.2 |
| Not given or corrected during construction | 1 | 2.4 | 2 | 2.3 | 3 | 9.7 |
| None | 1 | 2.4 | 24 | 27.9 | 1 | 3.2 |
| Total number of bridges | 41 | | 86 | | 31 | |

Table 14.—Tolerance of bridges to structural damage

| Structural damage | Movement | | | | | |
|---|---|---|---|---|---|---|
| | Tolerable | | | Intolerable | | |
| | Number of bridges | Percent of bridges[1] | Multiple damages[2] | Number of bridges | Percent of bridges | Multiple damages |
| Abutment damage | 22 | 27.5 | 9 | 19 | 25.7 | 16 |
| Pier damage | 3 | 3.8 | 3 | 8 | 10.8 | 6 |
| Vertical displacement | 7 | 8.8 | 5 | 15 | 20.3 | 13 |
| Horizontal displacement | 7 | 8.8 | 4 | 29 | 39.2 | 24 |
| Superstructure distress | 17 | 21.3 | 6 | 36 | 48.6 | 29 |
| Damage to rails, curbs, sidewalks, parapets | 5 | 6.3 | 4 | 9 | 12.2 | 7 |
| Bearing damage | 5 | 6.3 | 2 | 24 | 32.4 | 21 |
| Poor riding quality | 1 | 1.3 | 1 | 10 | 13.5 | 6 |
| Not given or corrected during construction | 3 | 3.8 | 1 | 4 | 5.4 | 0 |
| None | 28 | 35 | 0 | 0 | 0 | 0 |
| Total number of bridges | 80 | | | 74 | | |

[1] Percent of bridges with indicated structural damage.

[2] Multiple damages refers to the number of bridges that had structural damage in addition to the indicated damage.

Although the sample sizes were smaller, the same trends shown in table 16 and described above held, regardless of span type (simply supported or continuous) and structural materials (steel or concrete). However, the lack of tolerance to horizontal movements was slightly more evident for all continuous structures and for concrete bridges. When the tolerance data were analyzed by number of spans the same trends held, but the sample sizes were often too small to be statistically reliable.

When the data shown in table 16 were analyzed by abutment type, some differences in tolerance to foundation movements became evident. The data clearly showed that bridges with full height abutments were more tolerant to both differential vertical and horizontal movements than bridges with either perched or spill-through abutments. For example, for full height abutments, 94.1 percent of the differential vertical movements less than 152 mm (6 in) were reported as tolerable. Although all of the differential vertical movements less than 102 mm (4 in) were reported as tolerable for bridges with perched abutments, only 18.2 percent of those greater than 102 mm (4 in) were considered tolerable. Similar trends were observed with horizontal movements. An unsuccessful attempt was made to explain the significant findings among abutment types by correlating tolerance to movement with the various design and construction parameters used to select the most appropriate abutment for a particular bridge.

The influence of span length on the tolerance of bridges to foundation movements was studied in terms of maximum longitudinal angular distortion. Table 17 summarizes magnitudes of angular distortion considered tolerable and intolerable for all kinds of bridges included in this portion of the study and for subdivisions by span type and material type. When all of the bridges in the analysis are considered, 100 percent of the angular distortions less than 0.001 and 96.6 percent of the 58 angular distortions less than 0.005 are tolerable. However, only 36.4 percent of the values of angular distortion between 0.005 and 0.01 and 25 percent of those over 0.01 are tolerable. Therefore, an upper limit on angular distortion of 0.005 would be reasonable. For example, differential settlements of 76 mm (3 in) and 152 mm (6 in) would be tolerable most probably for spans of 15.2 m (50 ft) and 30.5 m (100 ft), respectively.

However, when the data are subdivided by span type, table 17 shows that the continuous bridges are more sensitive to angular distortion than are the simply supported bridges. Although this was expected, the difference was not as great as anticipated. For the continuous bridges 96.3 percent of the angular distortions less than 0.004 were tolerable, while only 28.6 percent of those over 0.004 were tolerable. In contrast, for the simply supported bridges, 100 percent of the angular distortions less than 0.005 were tolerable and no

**Table 15.—Tolerance to movements**

| Tolerance to movements | Movement | Frequency: Number of bridges | Frequency: Percent moved | Range | Average | Standard deviation |
|---|---|---|---|---|---|---|
| | | | | *Millimetres* | *Millimetres* | *Millimetres* |
| Tolerable | All types | 72 | 100 | | | |
| | Vertical | 44 | 61.1 | 0–457.2 | 61 | 88.9 |
| | Horizontal | 12 | 16.7 | 7.6–228.6 | 66 | 73.7 |
| | Vertical and | 16 | 22.2 | 0–304.8 | 61 | 86.4 |
| | horizontal | | | 7.6–152.4 | 43.2 | 43.2 |
| Intolerable | All types | 71 | 100 | | | |
| | Vertical | 16 | 23.9 | 58.4–1066.8 | 320 | 312.4 |
| | Horizontal | 20 | 26.8 | 25.4–457.2 | 182.9 | 129.5 |
| | Vertical and | 35 | 49.3 | 0–1280.2 | 276.9 | 335.3 |
| | horizontal | | | 10.2–508 | 127 | 119.4 |

1 mm = $3.9 \times 10^{-2}$ in

**Table 16.—Range of movement magnitudes considered tolerable or intolerable**

| Movement interval[1] | Vertical only: Tolerable | Vertical only: Intolerable | Horizontal only: Tolerable | Horizontal only: Intolerable | Vertical and horizontal, Vertical component: Tolerable | Vertical and horizontal, Vertical component: Intolerable | Vertical and horizontal, Horizontal component: Tolerable | Vertical and horizontal, Horizontal component: Intolerable |
|---|---|---|---|---|---|---|---|---|
| *Millimetres* | | | | | | | | |
| 0–50.8 | 28 | 0 | 8 | 1 | 10 | 5 | 11 | 9 |
| 50.8–101.6 | 9 | 2 | 1 | 7 | 4 | 9 | 3 | 10 |
| 101.6–152.4 | 2 | 3 | 1 | 0 | 0 | 4 | 1 | 6 |
| 152.4–203.2 | 3 | 2 | 0 | 3 | 0 | 3 | 1 | 3 |
| 203.2–254 | 0 | 3 | 2 | 4 | 1 | 1 | 0 | 3 |
| 254–381 | 1 | 2 | 0 | 3 | 1 | 6 | 0 | 2 |
| 381–508 | 1 | 2 | 0 | 2 | 0 | 2 | 0 | 1 |
| 508–1524 | 0 | 2 | 0 | 0 | 0 | 5 | 0 | 1 |
| Total | 44 | 16 | 12 | 20 | 16 | 35 | 16 | 35 |

[1] For vertical movements, magnitudes refer to maximum differential vertical movement. For horizontal movements, magnitudes refer to maximum horizontal movement of a single foundation element.

1 mm = $3.9 \times 10^{-2}$ in

**Table 17.—Tolerable or intolerable magnitudes of longitudinal angular distortion**

| Angular distortion interval ($\times 10^{-3}$) | Number of bridges of given type and tolerance | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | Span | | | | Material | | | |
| | All bridges | | Simple | | Continuous | | Concrete | | Steel | |
| | Tolerable | Intolerable | Tolerable | Intolerable | Tolerable | Intolerable | Tolerable | Intolerable | Tolerable | Intolerable |
| 0–0.99 | 31 | 0 | 13 | 0 | 18 | 0 | 20 | 0 | 11 | 0 |
| 1–1.99 | 8 | 1 | 5 | 0 | 3 | 1 | 5 | 1 | 3 | 0 |
| 2–2.99 | 6 | 0 | 3 | 0 | 3 | 0 | 3 | 0 | 3 | 0 |
| 3–3.99 | 7 | 0 | 5 | 0 | 2 | 0 | 2 | 0 | 5 | 0 |
| 4–4.99 | 4 | 1 | 3 | 0 | 1 | 1 | 3 | 1 | 1 | 0 |
| 5–5.99 | 0 | 1 | 0 | 0 | 0 | 1 | 0 | 0 | 0 | 1 |
| 6–6.99 | 1 | 2 | 0 | 1 | 1 | 1 | 1 | 0 | 0 | 2 |
| 7–7.99 | 1 | 2 | 1 | 1 | 0 | 1 | 1 | 0 | 0 | 0 |
| 8–8.99 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| 9–9.99 | 2 | 2 | 1 | 1 | 1 | 1 | 1 | 2 | 1 | 0 |
| 10–19.9 | 3 | 6 | 2 | 3 | 1 | 3 | 2 | 2 | 1 | 4 |
| 20–39.9 | 1 | 5 | 1 | 4 | 0 | 1 | 1 | 2 | 0 | 3 |
| 40–59.9 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| 60–79.9 | 0 | 1 | 0 | 0 | 0 | 1 | 0 | 0 | 0 | 0 |
| Total | 64 | 21 | 34 | 10 | 30 | 11 | 39 | 8 | 25 | 10 |

angular distortion was intolerable until the value was greater than 0.006. For the above example, differential settlements of 61 mm (2.4 in) and 122 mm (4.8 in) would be more reasonable tolerable limits for continuous spans of 15.2 m (50 ft) and 30.5 m (100 ft), respectively.

Table 17 suggests that a maximum angular distortion of 0.005 would be a reasonable limit for both concrete and steel bridges. The reported trend (table 12) for the concrete bridges to experience more frequent and severe superstructure damage than the steel bridges as a result of foundation movements was not supported by the tolerance data in table 17. This implies that the frequently reported superstructure distress of concrete bridges was quite often determined to be tolerable. A detailed analysis of the data in table 14 by material type verified this.

## Summary

The data presented above show that a range of both vertical and horizontal movements of substructure elements has been experienced by a substantial number of U.S. and Canadian highway bridges. Generally, abutment movements occurred more frequently than pier movements. Although both the frequency and magnitude of vertical movements were substantially greater than horizontal movements for both abutments and piers, the horizontal movements were more damaging to bridge superstructures.

There were more pile foundations than spread footing foundations for both abutments and piers. Although spread footing foundations moved more frequently and with greater magnitudes than pile foundations, the differences were not large. It can be concluded that pile foundations do not always eliminate foundation movement and therefore are not necessarily superior to spread footings.

Depending upon the span type, span length, and construction material, the field data show that many highway bridges can tolerate significant magnitudes of total and differential vertical settlement without sustaining serious structural damage. Horizontal movement of substructure elements, particularly abutments, was found to be much more damaging than vertical movement and must be closely controlled to avoid serious structural damage. These results of the field data analysis will be combined with the results of the analytical studies of the behavior of bridges subjected to movements and used to develop a methodology for designing bridges to provide a rational set of criteria for tolerable bridge movements.

## Acknowledgments

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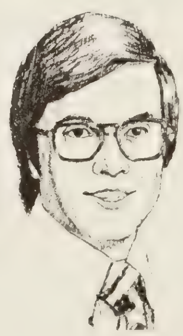
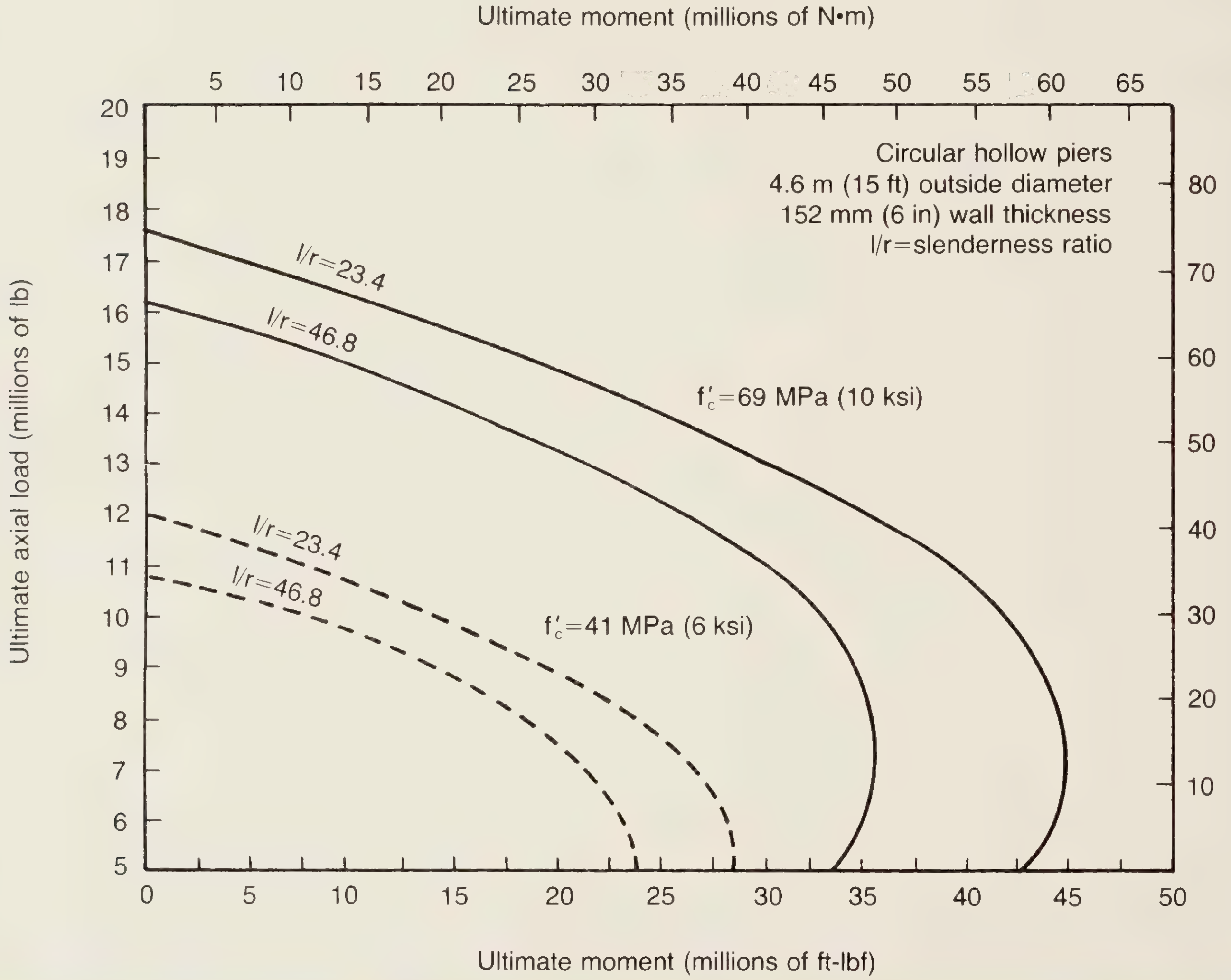


# Applications of High Strength Concrete for Highway Bridges

by
J. E. Carpenter




**Over the years the practical design compressive strength of portland cement concrete has steadily increased from 21 MPa (3 ksi) to 41 MPa (6 ksi). Now it is feasible to produce and design for 69 MPa (10 ksi) concrete. Concrete of this strength requires careful selection of high quality mixing materials and more rigid quality control. High strength concrete has been used successfully in the columns of a 76-story office building and in a 140 m (461 ft) liquid petroleum gas storage vessel.**

**High strength concrete has been made and used by prestressed concrete producers for years. However, the producers use the high *early* strength of the concrete so that the formwork may be removed, the concrete members prestressed, and the forms reused every day—without taking advantage of the eventual 55 to 69 MPa (8 to 10 ksi) compressive strength in the structural design.**

**This article describes the results of the first task of a three-task Federal Highway Administration (FHWA) research project to identify optimum structural applications of high strength concrete (approximately 69 MPa [10 ksi] compressive strength) in highway bridges. Task A,**

**Analytical Design Study, has been completed. Mixing specifications, physical properties, and advantages and disadvantages of high strength concrete were studied. Based on this information and analytical design studies, applications were developed for using high strength concrete in various kinds of bridges. Task A recommended a test program that will be performed and interpreted in Task B, Structural Laboratory Testing. Task C, Revisions to Design Criteria, will revise current bridge design criteria as necessary to implement the use of high strength concrete.**

## Mixing and Placing High Strength Concrete

Specifications for mixing and placing high strength concrete are based on the following: *(1–3)* [1]

*Constituents:* Cement, aggregates, and admixtures that will produce concrete of the required strength and insure consistency of properties over a period of time must be selected. The maximum size of coarse aggregate must be small, usually not greater than 13 mm (0.5 in). Superplasticizers may be used.

*Water/cement ratio:* The water/cement ratio must be low, usually between 0.30 and 0.35, by weight.

*Cement content:* The cement content must be high, usually between 475 and 565 kg/m$^3$ (800 and 950 lb/yd$^3$).

*Compaction:* Good compaction, combined with sturdy forms, is necessary to produce adequately dense concrete.

*Quality control:* A well-managed quality control program, including planning and cooperation of all parties involved, is essential to success.

[1] Italic numbers in parentheses identify references on page 83.

## Physical Properties of High Strength Concrete

The properties of high strength concrete, similar to the properties of normal strength concrete, can be predicted by anyone familiar with concrete engineering. Ratios of tensile strength to square root compressive strength for high strength concrete are similar to those obtained for normal strength concrete. *(4)* Stress-strain relationships for high strength concrete are shown in figure 1. *(4–6)* The initial slope of the curves tends to increase with increasing compressive strength (modulus of elasticity increases), and the strain at maximum stress tends to increase slightly. Also, the slope of the descending portion of the curves is steeper for the high strength concretes than for normal strength concretes, so the high strength concrete in effect is more brittle.

Data indicate that modulus of elasticity for 69 MPa (10 ksi) concrete is represented better by the formula $E_c = 26w^{1.5}\sqrt{f'_c}(0.0337w^{1.5}\sqrt{f'_c})$ *(4, 6, 7)* than by the formula $E_c = 33w^{1.5}\sqrt{f'_c}\ (0.0428w^{1.5}\sqrt{f'_c})$ presently suggested by the American Association of State Highway and Transportation Officials (AASHTO) code and the American Concrete Institute (ACI) code and used for normal strength concrete. *(8, 9)* High strength concretes creep less than normal strength concretes when loaded to a given percentage of compressive strength. *(4)* Shrinkage of high strength concrete is about the same as that of normal strength concrete. *(4, 7)*

## Advantages and Disadvantages of High Strength Concrete

One advantage of high strength concrete is its greater compressive strength, which can be evaluated in relation to unit cost, unit weight, and unit volume. High strength concrete, with its greater compressive strength per unit cost, is the least expensive means of carrying compressive force. In addition, its greater compressive strength per unit weight and unit volume allows lighter, more slender bridge piers, providing improved horizontal clearances.

Other advantages of high strength concrete include increased modulus of elasticity and increased tensile strength. Increased stiffness is advantageous when deflections or stability govern the bridge design, and increased tensile strength is advantageous in service load design in prestressed concrete.

A disadvantage of high strength concrete is that the mix has much less water than normal strength concrete. This results in mixes that have reduced workability and handling time, making them more difficult to place and properly compact. In addition, obtaining high quality aggregates in necessary sizes and obtaining cement that will consistently produce concrete of the required strength may be difficult.

Structural design considerations may preclude effective use of the increased concrete strength. Cross section dimensions often are governed by factors other than stress, such as minimum cover, so that the full strength capability of the concrete is not used. Further, the total prestress force that can be generated may not be sufficient to take advantage of the high strength concrete.

Other disadvantages of high strength concrete are its additional cost and the additional expenses of increased quality control. Finally, AASHTO specifications tend to discourage the use of high strength concrete because the specifications are based on the properties of normal strength concrete. *(8)*

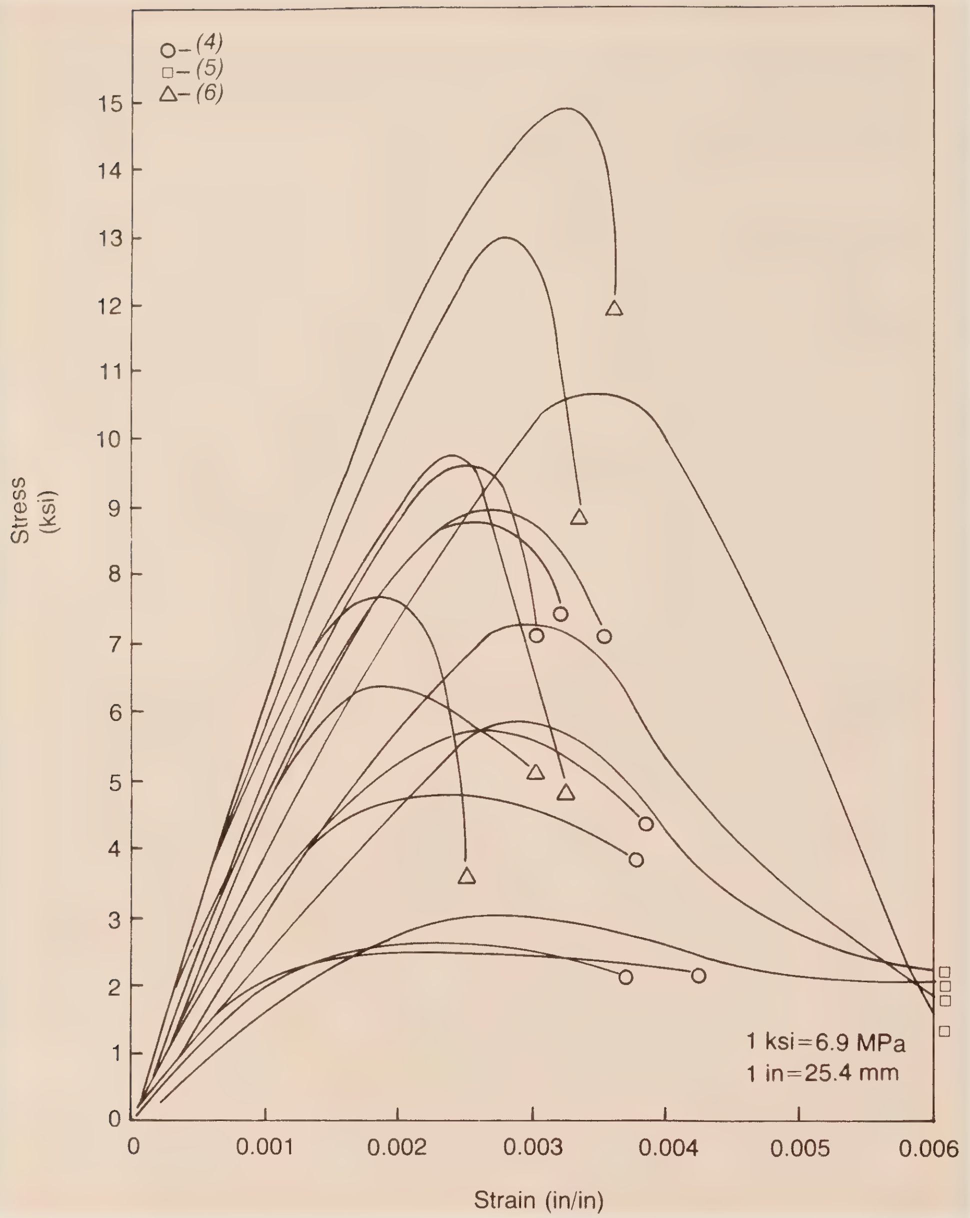



*Figure 1.—Stress-strain curves for high strength concrete obtained in three different studies.*

## Applications of High Strength Concrete

### Solid section girders

The effect of using high strength concrete in four different solid section girders was investigated. Cross section dimensions of the four girders—the bulb tee, the Washington State Department of Transportation (WSDOT) Standard, the AASHTO-Prestressed Concrete Institute (PCI) Standard, and the Colorado Standard—are shown in figure 2. Two methods of attaching the decks were considered: integrally cast and cast in place. Decks were cast in place on the completed girder without shoring, so that the entire dead load of both girder and deck was carried by the girder section.

To determine span capabilities of the girders, the AASHTO HS20-44 loading was used, with a lateral distribution factor S/5.5. Allowable stresses conformed to the AASHTO code, with allowable tension in the precompressed tension zone equal to $3\sqrt{f'_c}$ $(0.249\sqrt{f'_c})$ and allowable compression equal to 0.4 $f'_c$.

The span capabilities for a 1829 mm (72 in) integral deck bulb tee is shown in figure 3. Span capabilities at various girder spacings are given for concrete strengths of 41 MPa (6 ksi), 55 MPa (8 ksi), and 69 MPa (10 ksi). As shown, span capability for the closer spaced girders increases with each increase in concrete strength. For the wider spaced girders, capability increases when concrete strength is increased from 41 MPa (6 ksi) to 55 MPa (8 ksi). However, for concrete strengths above 55 MPa (8 ksi), span capability does not increase because insufficient prestress force inhibits the capabilities of the stronger concrete. Similar patterns were found for the other cross sections investigated.

Figure 3 also shows span capabilities based on a live load deflection criterion of l/800 (section 1.7.6 of AASHTO code for steel structures). *(8)* If applicable, this criterion governs most beam spacings.

Figure 4 shows span capabilities for the same 1829 mm (72 in) section with a cast-in-place deck. Although span capabilities increase with each increase in concrete strength, span capabilities are constantly lower for the cast-in-place deck than for the integral deck.

Making maximum use of the greater load carrying capabilities of high strength concrete girders requires different designs for bridges. Two designs of a 46 m (150 ft) simple span bridge are shown in figure 5. In the nine-girder design, 41 MPa (6 ksi) concrete girders were used. In the four-girder design, 69 MPa (10 ksi) concrete girders were used. The advantages of the high strength concrete are evident: Only four girders are needed for the high strength concrete design, while nine girders are needed for the normal strength concrete design. In spite of the thicker cast-in-place deck needed for the greater transverse span between the four girders, the overall dead load is reduced, and therefore total prestressing requirements are reduced.

## Posttensioned box girders

Multiple span, cast-in-place, continuous posttensioned box girder bridges of constant depth were represented by the two-span, continuous structure shown in figure 6. Concrete strengths were 41 MPa (6 ksi), 55 MPa (8 ksi), and 69 MPa (10 ksi). Overall beam depths were 1.4 m (4.5 ft), 1.7 m (5.5 ft), and 2 m (6.5 ft). Allowable stresses were the same as those used in the solid section girder analysis, except that allowable tension in the precompressed tension zone was assumed to be $6\sqrt{f'_c}$ $(0.498\sqrt{f'_c})$. Loading was three lanes of AASHTO HS20-44, without lane reduction.

Span capabilities for the different beam depths are shown in figure 7. High strength concrete for continuous box girders of 46 m to 76 m (150 ft to 250 ft) spans increased span capabilities. As with the integral deck solid section girders, the maximum available prestress force limited capabilities of the high strength concrete.

## Segmentally posttensioned box girders

Segmentally posttensioned box girder bridges of medium to long span were represented by the recently completed free cantilever Shubenacadie Bridge (South Mainland, Nova Scotia) with a 213 m (700 ft) main span and 113 m (372 ft) side spans. *(10)* Overall dimensions

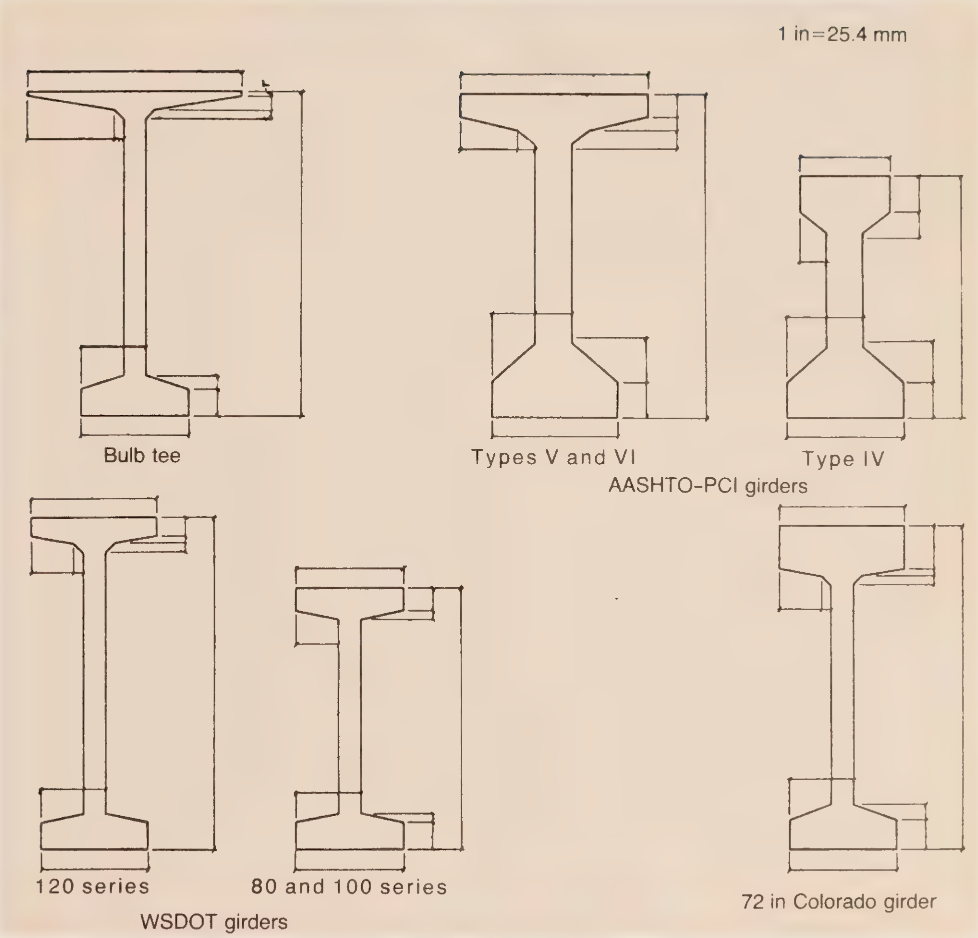
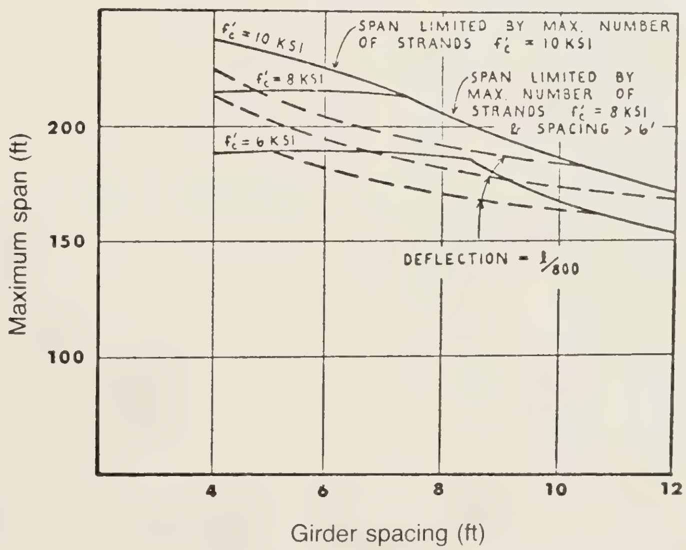



*Figure 2.—Cross sections of four kinds of solid section girders.*




1 ksi=6.9 MPa
1 ft=0.305 m

*Figure 3.—Span capabilities for a 1829 mm (72 in) integral deck bulb tee.*

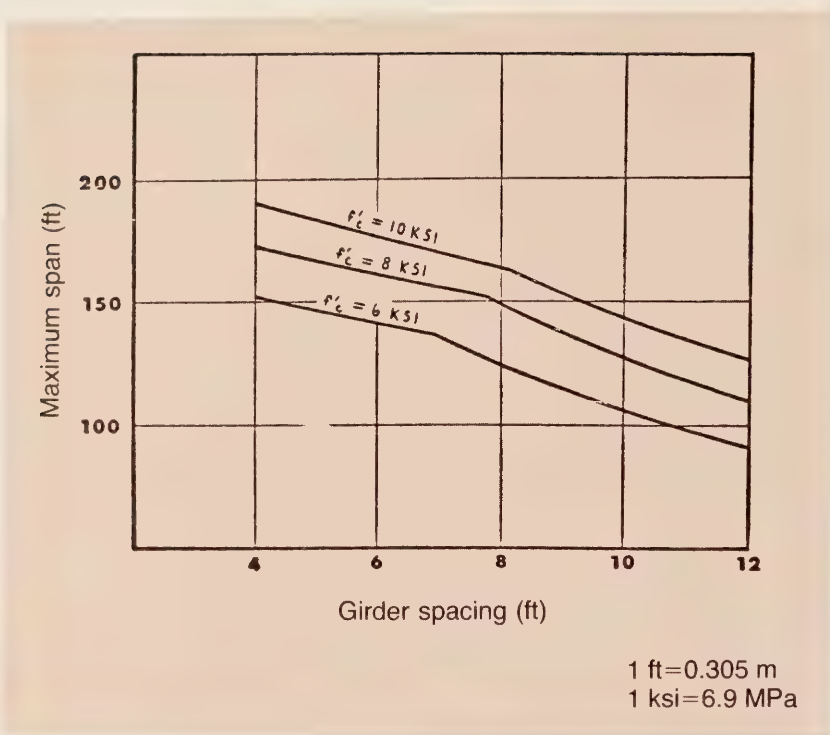



*Figure 4.—Span capabilities for a 1829 mm (72 in) cast-in-place deck bulb tee.*

of the bridge are shown in figure 8. The bridge was constructed with 34 MPa (5 ksi) concrete and used 31.75 mm (1.25 in) diameter thread bars for posttensioning. No tension was allowed in the precompressed tension zone. The AASHTO HS20-44 loading was used.

The bridge was redesigned with high strength concrete to determine how much the thickness of the lower flange could be reduced and what effect this reduction would have on the overall moments. As shown in figure 9, using high strength concrete reduced the total flexural prestress force by more than 10 percent as a result of the reduced dead load. The optimum lower flange thickness is about 0.5 m (1.6 ft), obtained at 55 MPa (8 ksi) strength.

Despite the large size of the Shubenacadie Bridge, geometric requirements for cover over reinforcement and prestressing still

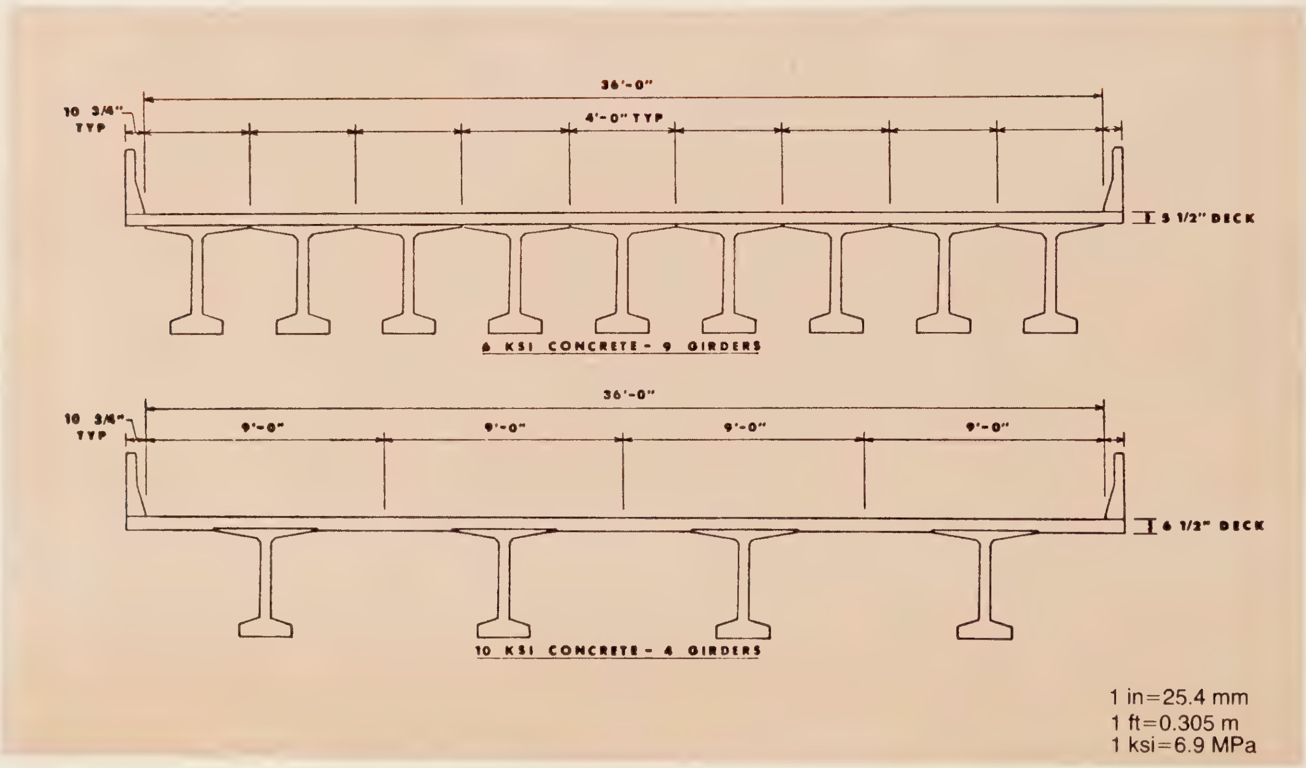



*Figure 5.—Two 46 m (150 ft) simple span bridges with different concrete strengths.*

govern thicknesses of deck and webs. By using high strength concrete in wider bridges with three or more webs, the number of webs can be decreased and the transverse span lengths of the deck increased, similar to the wider spacings used for the solid section girders made of high strength concrete. Transverse posttensioning is included in the design. The reduced weight and loads possible with high strength concrete further reduce concrete area and prestress force.

For segmentally posttensioned box girder bridges, high strength concrete is feasible in regions, such as the lower flange, where the design is controlled by stress. In regions, such as in the deck, where the design is controlled by other factors, normal strength concrete can be used. The webs may be constructed of either high strength concrete or normal strength concrete, depending on minimum thickness requirements, minimum shear reinforcement requirements, and the contribution of concrete in the webs to the shear carrying capacity.

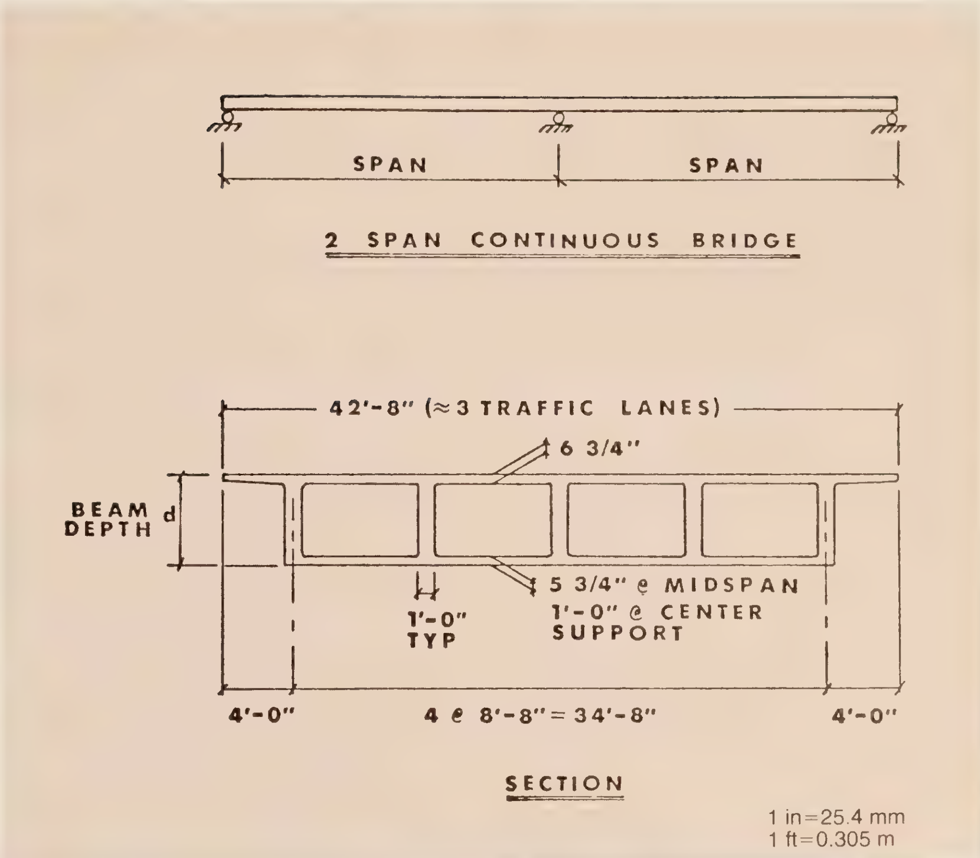



*Figure 6.—Two-span continuous posttensioned box girder bridge.*

## Compression members

Pier shafts and elements of Y-piers (fig. 10) are examples of compression members in bridges. Compression members were expected to benefit greatly from high strength concrete.

To study the effect of increasing the compressive strength of concrete from 41 MPa (6 ksi) to 69 MPa (10 ksi), interaction diagrams were developed for the compression member shown in figure 11. The diagrams were developed for the pin end condition, assuming that the cross section possessed the bilinear moment-curvature relationship defined by strain compatibility and the elastic properties of the concrete and prestressing strand. The deflected shapes were determined by integrating the curvatures along the

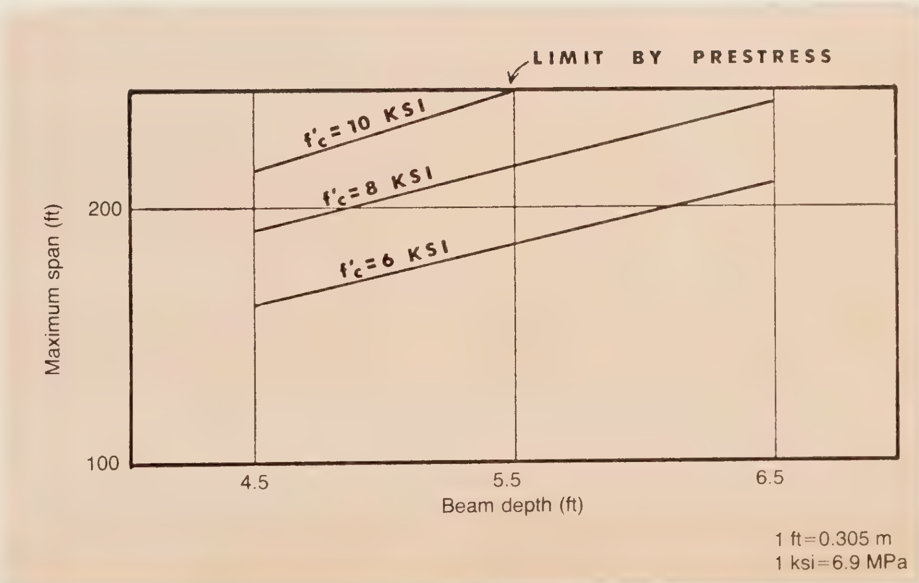



*Figure 7.—Span capabilities for a two-span continuous box girder bridge with different beam depths.*

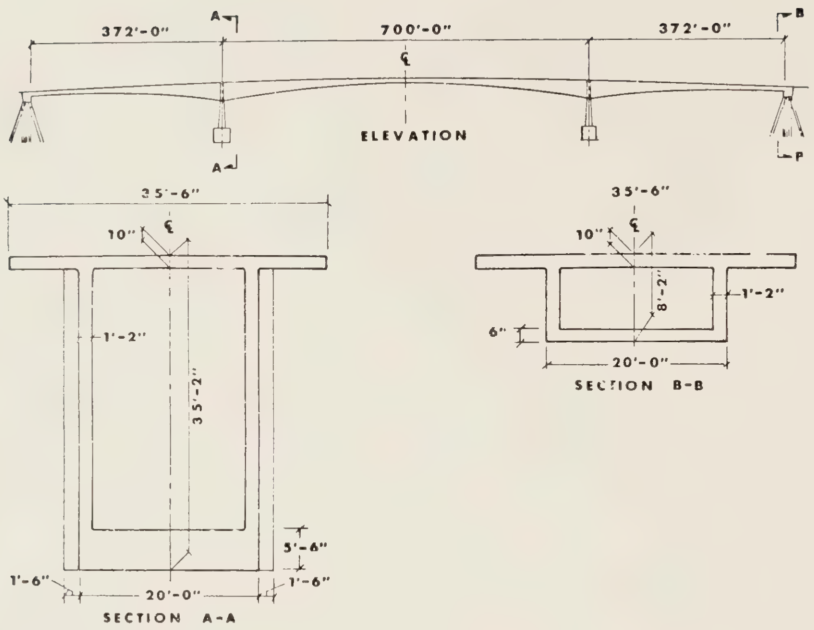



*Figure 8.—Overall dimensions of the free cantilever Shubenacadie Bridge.*

length of the member. The ultimate strength design method was used with factored loads.

Figures 12 and 13 show that capacities of the compression members increase with the increase in concrete strength.

Results of the compression member study are shown in table 1. Two slenderness ratios, three concrete strengths, and three end eccentricities are shown. For short, concentrically loaded struts, capacity is determined by multiplying the cylinder strength by the cross sectional area and then subtracting the prestress force. Because the prestress force is constant, the benefits of increasing concrete strength are more than the strength increase itself. This is illustrated by the strength ratios in the table that exceed 10/6=1.67 for those struts.

For combined axial load and bending, using high strength concrete, even for relatively slender columns, is beneficial. It can be concluded that compression members are an excellent application for high strength concrete. Smaller sections can be used for a given number of members, or fewer members can be used in a given location. In either case, weight as well as material and construction cost are reduced.

## Summary

The analytical design studies discussed above demonstrate the benefits of using high strength concrete in solid section girders, posttensioned box girders, segmentally posttensioned box girders, and compression members. These benefits include increased span lengths, reduced dead loads, and greater load capacities.

Continuing research under this project will provide additional design information on potential structural applications for high strength concrete as well as evaluation of data obtained from laboratory tests of structural specimens.

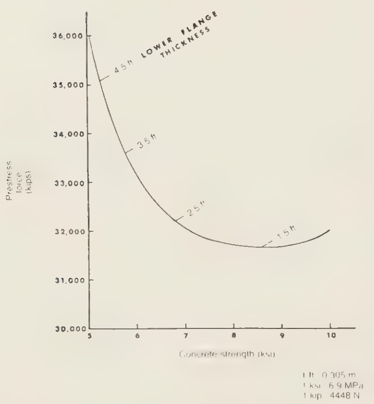



*Figure 9.—Variation of prestress force with concrete strength (Shubenacadie Bridge).*

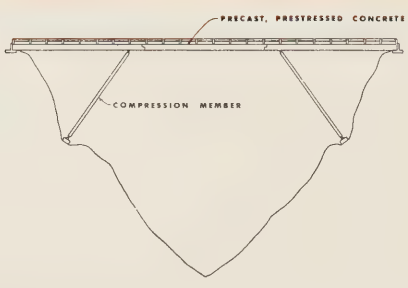



*Figure 10.—Application of Y-piers in bridges.*

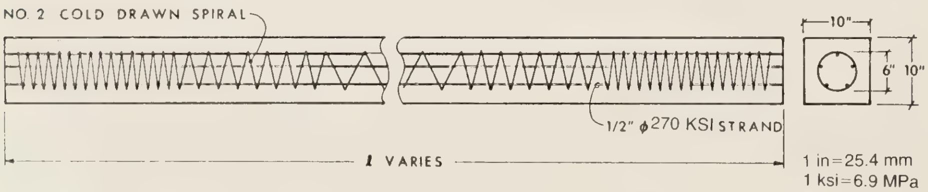



*Figure 11.—Compression member details.*

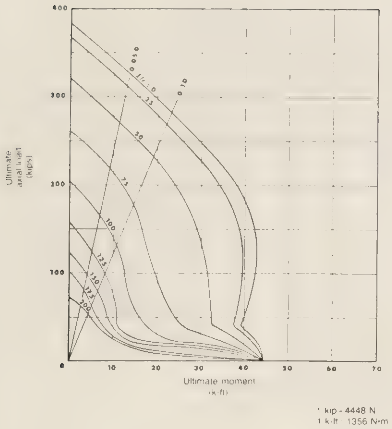



*Figure 12.—Interaction diagram for compression member of 41 MPa (6 ksi) concrete.*

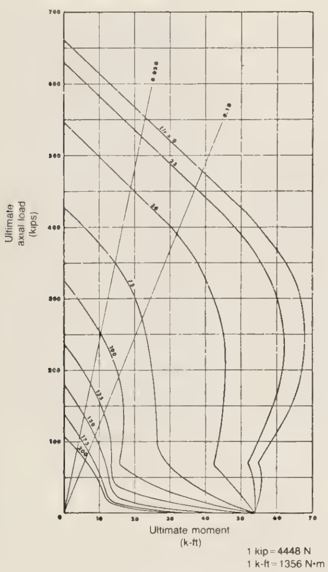



*Figure 13.—Interaction diagram for compression member of 69 MPa (10 ksi) concrete.*

**Table 1.—Axial load capacities for a 254 mm (10 in) square strut**

| | | Eccentricity | | | | | |
|---|---|---|---|---|---|---|---|
| Slenderness ratio l/r | Concrete strength $f'_c$ | e=0 P | e=0 $P/P_6$ | e=0.1D P | e=0.1D $P/P_6$ | e=0.2D P | e=0.2D $P/P_6$ |
| | *ksi* | *kips* | | *kips* | | *kips* | |
| | 6 | 375 | 1.00 | 284 | 1.00 | 217 | 1.00 |
| 0 | 8 | 518 | 1.38 | 389 | 1.37 | 294 | 1.35 |
| | 10 | 662 | 1.77 | 478 | 1.69 | 362 | 1.67 |
| | 6 | 320 | 1.00 | 233 | 1.00 | 168 | 1.00 |
| 50 | 8 | 437 | 1.37 | 317 | 1.36 | 220 | 1.31 |
| | 10 | 545 | 1.70 | 390 | 1.67 | 267 | 1.59 |
| | 6 | 205 | 1.00 | 133 | 1.00 | 80 | 1.00 |
| 100 | 8 | 270 | 1.32 | 165 | 1.24 | 90 | 1.13 |
| | 10 | 325 | 1.59 | 190 | 1.43 | 100 | 1.25 |

1 ksi = 6.9 MPa
1 kip = 4448 N

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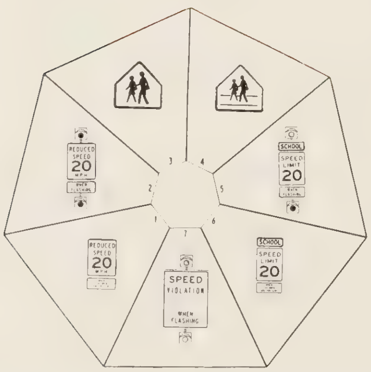
**James E. Carpenter** recently joined Andersen–Bjornstad–Kane–Jacobs, Inc., where he is project leader in charge of substructure design for the West Seattle Bridge in Washington State. In his 25 years experience in bridge design and construction, Dr. Carpenter has specialized in reinforced and prestressed concrete structures.

Dr. Carpenter wrote this article while with the Concrete Technology Corporation where he supervised an investigation of the benefits of very high strength concrete in highway bridges and the development of a biaxially prestressed, precast deck system. Dr. Carpenter also supervised the development of design procedures for integral bent caps in reinforced concrete box girder bridges and participated in construction and loading of the Three Sisters Bridge model—a research project that aided the acceptance of prestressed concrete free cantilever bridge construction in the United States.

# 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.**




## Effectiveness of Speed Control Signs in Rural School Zones and Small Communities, Report No. FHWA-RD-79-20

**by FHWA Traffic Systems Division**

This report assesses the effectiveness of various speed control signs located on high-speed, two-lane highways in rural school zones and small communities. Vehicle speed data were collected in two small communities in Mississippi and at six school zones in California, Oregon, and Mississippi. Existing signs in the test areas were evaluated in relation to the addition of a reduced speed ahead sign, beacon flashers coupled with the speed limit sign and reduced speed ahead sign, and a traffic-actuated speed violation sign. Roadside interviews were conducted to study driver understanding of and attitude toward the speed control devices. Vehicle speed data and interview data determined if each sign tested increased safety and driver awareness of potential hazards.

Results indicated there was a 3.22 km/h to 16.10 km/h (2 mph to 10 mph) reduction in average operating speed using beacon flasher signs and speed violation signs.

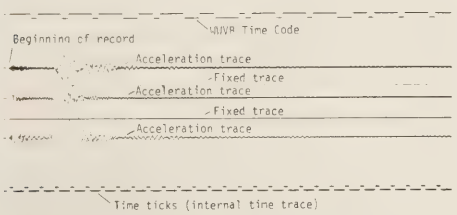
A limited number of copies of the report are available from the Traffic Systems Division, HRS-33, Federal Highway Administration, Washington, D.C. 20590.


## Identification of Traffic Management Problems in Work Zones, Report No. FHWA-RD-79-4

**by FHWA Traffic Systems Division**

This report presents a prioritized listing of 20 problem statements on traffic safety in construction, maintenance, and utility work zones. These problem statements were developed from data collected from more than 100 site visits. All aspects of work zone traffic control were examined including planning, design, installation, operation, maintenance, and removal. Detailed analyses were made of accidents in 30 construction zones to determine why accidents happen in work zones. From the site observations, accident analyses, and a literature review, the principal areas of work zone traffic control needing research were defined. Results indicate that approximately two-thirds of the work zone safety problems would be less severe if current standards and knowledge were properly applied.

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 80 139520).




## Use and Interpretation of Strong-Motion Records From Highway Bridges, Report No. FHWA-RD-78-158

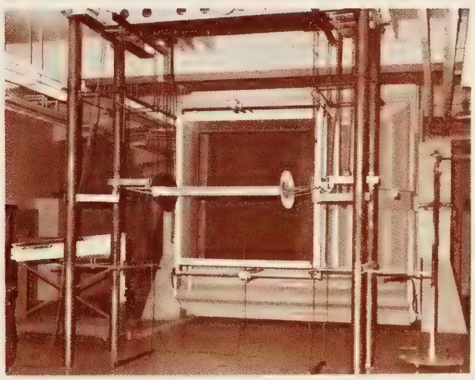
**by FHWA Structures and Applied Mechanics Division**

In recent years several State highway bridges in high seismic probability regions of the United States have been permanently instrumented with strong motion accelerometers and recorders to obtain a time-based record of any bridge dynamic structural motions caused by a local earthquake.

This report presents background information on strong motion instrumentation and strong motion records and describes analytical methods proposed for interpreting the complex data contained in records of the dynamic responses of moderate length bridges to strong ground shaking. A linear mathematical model is formulated for calculating the bridge responses. Techniques are outlined for reducing both analog and digital records of strong motion responses. The objective of the analysis of either type of record is to determine force levels throughout a given bridge for a particular earthquake input and to identify the mathematical model parameters that best describe the measured responses. A computerized linear least squares fitting procedure in the time domain is proposed to identify mathematical model parameters from recorded strong motions. In two example problems, force levels in a typical bridge are calculated for hypothetical strong motion inputs.

The appendixes to the report provide an extensive discussion of the dynamics of structures and a listing for the computer program used.

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 292254).



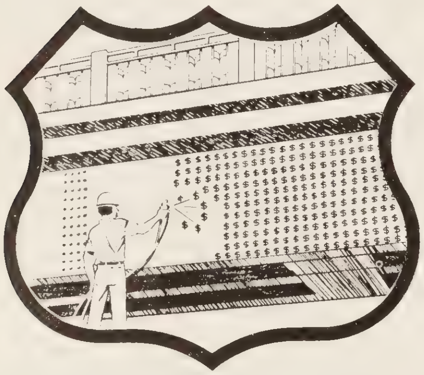
## Aerodynamic Investigations of the Luling, Louisiana Cable-Stayed Bridge, Report No. FHWA-RD-77-161

**by FHWA Structures and Applied Mechanics Division**

The selection of the final design for a proposed cable-stayed girder Interstate highway bridge over the Mississippi River at Luling, La., was preceded by a comprehensive wind tunnel section model study of the aerodynamic stability of seven alternate suspended superstructure configurations. This experimental study was conducted in the George S. Vincent Memorial Wind Tunnel at the FHWA Fairbank Highway Research Station in McLean, Va. The motion responses of the section models were evaluated over a range of wind velocities and aspect ratios to identify possible susceptibility of the prototype suspended span designs to excessive vertical or torsional vibrations from wind. After the relative stability of each of the seven configurations had been established, additional wind tunnel studies were conducted on models of three partially completed stages of construction of the preferred design and of a cable support tower to determine whether aerodynamic instability might occur at intermediate stages of construction. Wind flow patterns around the preferred suspended span design model were investigated to determine optimum locations for wind-monitoring sensors on the prototype structure.

The results of the study indicated that critical aerodynamic flutter should not occur with the preferred configuration of the suspended span at wind velocities up to 240 km/h (150 mph) and that significant vibrations from wind vortex shedding action should not occur if the damping ratio in the prototype is at least 1 percent of critical. None of the sequential construction stages of the prototype bridge suspended span or tower should show any aerodynamic instability in the same wind velocity range. Drag forces measured on the models correlated with the American Association of State Highway and Transportation Officials design values. The results of the wind flow pattern study showed that if the wind sensors are mounted on top of 10.7 m (35 ft) light poles in the bridge roadway median, the measured ambient wind velocities and directions will be free of distortions caused by the boundary layer effect.

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 292859).



## Coating and Corrosion Costs of Highway Structural Steel, Report No. FHWA-RD-79-121

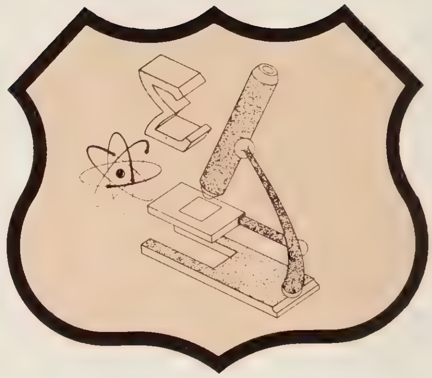
**by FHWA Materials Division**

A simulation model has been developed to aid in selecting cost-effective maintenance policies for highway bridge painting. In addition to analyzing costs for a specific maintenance measure selected, the model can determine the optimal protection method and painting schedule for a specific bridge.

This model shows that the most commonly used protection method—a 0.13 mm (5 mils) oleoresinous system—is not optimal. In a marine environment optimal protection is offered by a 0.18 mm (7 mils) zinc-rich system. In an industrial environment optimal results are obtained by either a 0.18 mm (7 mils) zinc-rich system or a 0.15 mm (6 mils) synthetic polymer system. The latter system is also optimal in a rural environment. According to data gathered in 1979, the minimum cost of adequately painting highway bridges under State jurisdiction is about $160 million. The actual amount spent is only about $80 million. The result is annual rehabilitation costs of $300 to $400 million.

The cost to the taxpayer for corrosion of the Nation's bridges has been estimated at about $120 to $150 million for 1979. This cost includes painting, inspection, contract preparation, and overhead but does not include rehabilitation costs.

The report is available from the Materials Division, HRS-23, Federal Highway Administration, Washington, D.C. 20590.



## Alternate Highway Deicing Chemicals, Executive Summary, Report No. FHWA-RD-79-109

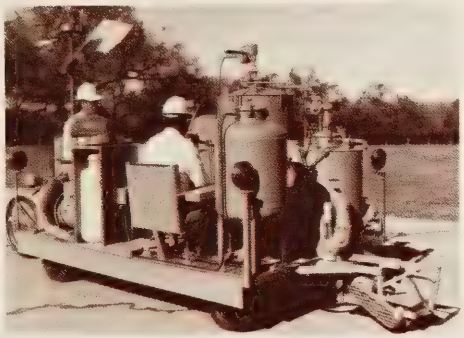
**by FHWA Materials Division**

Numerous chemical compounds with the potential to replace sodium chloride (NaCl) as a roadway deicer were studied because of drawbacks associated with using NaCl. Potential replacements were selected after considering their water solubility and freezing point lowering, corrosion, toxicity, relative cost or cost potential, effects on soils, plants, and water supplies, flammability, concrete compatibility, traction, friction, and highway performance. Information found in literature was supplemented or verified in the laboratory as needed.

Two candidate deicers—methanol and calcium magnesium acetate (CMA)—show promise as being as effective as NaCl. Methanol reacts almost immediately upon contact with snow and ice but is not as long lasting as NaCl and has greater utility as a fuel. CMA acts at about the same rate within the same temperature ranges and for approximately the same length of time as NaCl. CMA and NaCl produce similar changes in braking traction and skidding friction.

Unlike NaCl, CMA is not commercially available and a process for its manufacture would have to be developed. Unpurified CMA can be derived from solid wastes, primarily cellulose, which would aid traction and reduce production costs. In addition, limited laboratory tests indicate that CMA may be beneficial to most soils, unharmful to drinking supplies, and significantly less corrosive than NaCl.

The report is available from the Materials Division, HRS-23, Federal Highway Administration, Washington, D.C. 20590.

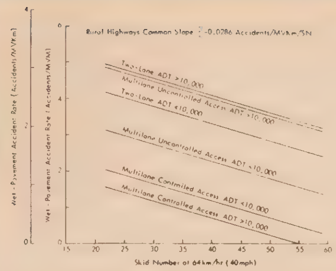


### Equipment for Applying Epoxy Thermoplastic Paving Marking Material, Report No. FHWA-RD-79-130

**by FHWA Materials Division**

This report provides general specifications for an airless, low-pressure spray application system for epoxy thermoplastic pavement marking material. The main design and operation features of the equipment are documented. The specifications presented in this report are not intended to be rigid, but rather are guides for those seeking a system that is both simple and effective to use.

The report is available from the Materials Division, HRS-23, Federal Highway Administration, Washington, D.C. 20590.




Relationship Between Wet-Pavement Accident Rate and Skid Number at 64 km/h (40 mph) for Rural Highways

### Effectiveness of Alternative Skid Reduction Measures, Executive Summary and Volumes I-IV, Report Nos. FHWA-RD-79-21/25

**by FHWA Environmental Division**

These reports describe a study that examined the relationship between wet-pavement accidents and tire-pavement skid resistance, developed a computerized benefit-cost model for evaluating accident countermeasures, and produced a guide for improving pavement macrotexture.

Accident rate, skid number, and related data were collected for 2 years on 428 highway sections in 16 States. Extensive statistical analyses indicated a linear relationship between wet-pavement accident rate and skid number. The slope of the relationship (decrease in accident rate with increase in skid number) was independent of highway and area type and average daily traffic; however, the magnitude of the accident rate depended on such factors.

The executive summary presents the relationships between wet-pavement accident rates and skid numbers and includes a brief description of the study. Volume I describes the evaluation of accident rate-skid number relationships, and Volume II describes the development of a computerized benefit-cost model for wet-pavement accident countermeasures involving surface treatments as well as geometric, traffic control, and other remedial measures. Volume III contains instructions for using the computerized model, and Volume IV summarizes methods for measuring and achieving macrotexture.

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

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

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

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

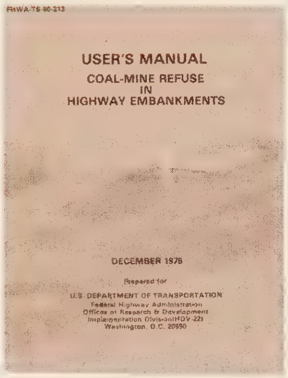
**Items available from the Implementation Division can be obtained by including a self-addressed mailing label with the request.**

## User's Manual, Coal-Mine Refuse in Highway Embankments, Report No. FHWA-TS-80-213

**by FHWA Implementation Division**

Coal-mine refuse (CMR), the waste product of mining and processing coal for market, has commonly been disposed of in large refuse banks. These banks are not only unsightly but also pollute surrounding water and air.

This manual provides guidelines for using CMR in the design and construction of highway embankments. A separate section of




the manual presents data that are needed for a complete understanding of CMR properties, its origins, and regulations governing its disposal. Case histories of highway embankment construction with CMR are also included. The manual is intended for highway and environmental planners and those concerned with highway design, materials testing, and construction.

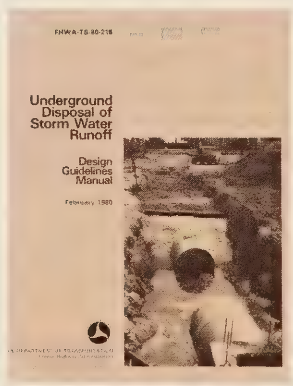
The manual may be purchased for $6 from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Stock No. 050-001-00169-1).

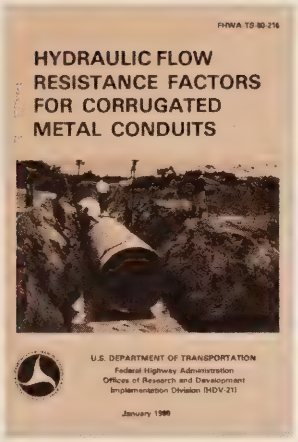
## Underground Disposal of Storm Water Runoff, Design Guidelines Manual, Report No. FHWA-TS-80-218

**by FHWA Implementation Division**

Disposal of storm water by infiltration into soil can provide a practical and attractive alternative to the more conventional and often more expensive storm water conveyance systems. This manual provides design guidelines for subsurface disposal of storm water by infiltration. The state of the art for design, maintenance, and construction of various infiltration drainage systems is presented, and infiltration basins, trenches, and wells are discussed in detail. Environmental and legal considerations relative to these systems are also presented.

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






## Hydraulic Flow Resistance Factors for Corrugated Metal Conduits, Report No. FHWA-TS-80-216

**by FHWA Implementation Division**

The resistance of the various corrugation shapes used in making corrugated metal conduits for highway drainage must be determined for reliable hydraulic design. The resistance varies widely for each of the corrugation shapes now available.

This report presents methods for estimating the resistance of all available corrugation shapes. Variables considered include conduit size and shape, corrugation shape, flow rate, flow depth, and manufacturing method. Resistance is presented in terms of the Darcy f or the Manning n, and design charts, geometric tables, and the International System of Units (SI) conversion factors are included. A method of estimating resistance for untested corrugations, along with design examples, also is included.

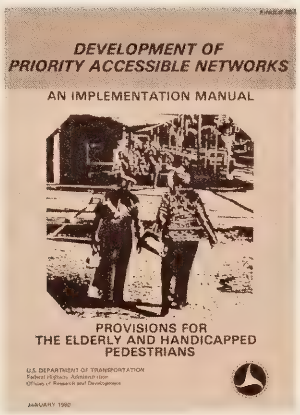
Limited copies of the report are available from the Implementation Division.

## Development of Priority Accessible Networks—Provisions for the Elderly and Handicapped Pedestrians, Implementation Package 80-8

**by FHWA Implementation Division**

This manual describes a procedure for assessing the needs of elderly and handicapped pedestrians in urban areas. The procedure aids metropolitan planning organizations and public works agencies in identifying routes or corridors heavily traveled by the elderly and handicapped. These routes or corridors then can be surveyed to determine features that impede movement and need modification.

The manual is divided into two parts. Part one describes the planning procedure for developing an accessible network and part two describes specific problems experienced by elderly and handicapped pedestrians and recommends solutions.



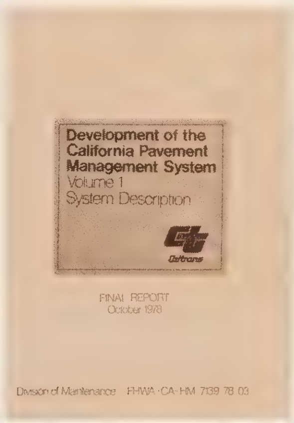

The manual will be useful to persons in urban communities involved with pedestrian programs. The features shown in the manual can improve the safety and convenience of facilities for all pedestrians. Planners, designers, and safety program personnel in State and local agencies and citizen groups also may find this information useful.

The manual may be purchased for $6 from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Stock No. 050-001-00163-2).

## Development of the California Pavement Management System, Volume I, System Description, Report No. FHWA-CA-HM-7139-78-03

**by California Department of Transportation**

A pavement management system (PMS) has been developed to monitor deteriorating pavement, to disseminate repair strategy information, to substantiate cost-effective rehabilitation strategies, and to manage existing pavements in the State highway system. PMS begins with a Statewide



pavement condition field survey taken every 2 years. This survey identifies the location, nature, severity, and extent of pavement problems so data from the survey together with costs, rehabilitation strategies, traffic data, highway classification data, and other information can be computerized. The computer follows an engineering logic process to identify an appropriate level of repair for pavement. Because of the characteristic differences in flexible and rigid pavement problems and remedies, each pavement type is treated separately in the PMS. The end product is a series of reports describing pavement condition, rehabilitation strategy, and cost; candidate projects for various highway programs; and alternative repair strategies. This information will be used to establish the magnitude and character of the various highway programs, the priority of repair work, and the funding to provide a designated level of service for California.

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

### Optimizing Maintenance Activities, Ninth Report, Rest Area Maintenance, Report No. FHWA-TS-80-210

**by FHWA Implementation Division**

This report summarizes the results of a value engineering study of rest area maintenance conducted by the States of Alabama, Illinois, Michigan, Virginia, and Wisconsin. It was estimated that more than $3 million annually could be saved among the five States with implementation of the recommendations of this study.

Because it was determined that 60 to 80 percent of rest area maintenance costs are for custodial labor,

Report No. FHWA-TS-80-210 FCP Category 7

Optimizing Maintenance Activities
Ninth Report

REST AREA MAINTENANCE

Combined State Studies
of Selected Maintenance Activities
A Cooperative Analysis by Teams from
Alabama, Illinois, Michigan, Virginia, Wisconsin

January 1980

U.S. DEPARTMENT OF TRANSPORTATION
Federal Highway Administration
Offices of Research and Development
Implementation Division
Washington, D.C. 20590

reducing the number of labor hours will reduce the overall maintenance cost. Other recommendations concern sewage systems, trash and litter pickup, and contract maintenance.

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

### Optimizing Maintenance Activities, Tenth Report, Repair of Concrete Pavement Joints, Report No. FHWA-TS-80-215

**by FHWA Implementation Division**

FCP Category 7 Report No. FHWA-TS-80-215

Optimizing Maintenance Activities
Tenth Report

REPAIR OF CONCRETE PAVEMENT JOINTS

Combined State Studies
Of Selected Maintenance Activities
A Cooperative Analysis By Teams From
Iowa, Kansas, Nebraska, South Dakota

February 1980

U.S. DEPARTMENT OF TRANSPORTATION
Federal Highway Administration
Office of Development
Implementation Division
Washington, D.C. 20590

This report summarizes the results of a value engineering study conducted by the States of Iowa, Kansas, Nebraska, and South Dakota. The study covers repair of noncontinuously reinforced concrete pavements only. Deteriorated or defective joints in pavement cause unsatisfactory riding quality and performance of rigid pavements. When deterioration is so extensive that temporary surface patching no longer provides a safe, smooth riding surface, removal and replacement of a portion of the pavement are necessary.

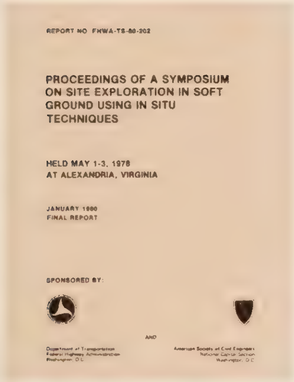
The joint repair problems of each State are individual and complex, requiring increased production in both contract and maintenance repair operations. Maximizing the use of machinery to accomplish the work instead of labor-oriented procedures is emphasized.

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

### Proceedings of a Symposium on Site Exploration in Soft Ground Using In Situ Techniques, Report No. FHWA-TS-80-202

**by FHWA Implementation Division**

This report contains the proceedings of a 3-day symposium on site exploration in soft ground using in situ techniques held in May 1978. This symposium was the result of a research and development program to assimilate current information on the use and effectiveness of several new soft ground site exploration devices, including the pressuremeter (dilatometer), the vane shear, the borehole shear, and the static cone penetrometer.




The objective of the symposium was to review, summarize, and document any research conducted on these exploration devices and to present case histories in which each device was used to determine engineering soil properties and obtain design criteria. This research and development generated interest in the developmental stages and the potential uses of the many new site exploration techniques. The need for a summary of the state of the art of in situ site exploration testing of soils was identified.

This report includes information on site exploration needs and compares exploration techniques as well as technical capabilities and uses of several site exploration devices. Classical pressuremeters are compared to the recently developed PAF (French) and Cambridge (English) self-boring pressuremeters. The report also includes a paper on the statics and dynamics of the standard penetration test (SPT).

The report may be purchased for $6.50 from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Stock No. 050-001-00170-5).

## Proposed Design Specifications for Steel Box Girder Bridges, Report No. FHWA-TS-80-205

**by FHWA Implementation Division**

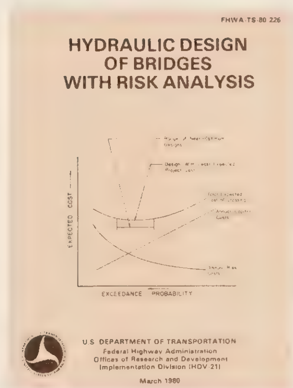
In 1975, the Federal Highway Administration (FHWA), the American Society of Civil Engineers (ASCE), and the American Association of State Highway and Transportation Officials (AASHTO) recognized the need for a set of design specifications for long-span steel box girder bridges. Design specifications then in use were complex and unworkable. Therefore, FHWA sponsored the development of a set of practical design rules that would reflect the state of the art and that could be applied to steel box girders for long-span bridges. This report contains the first set of comprehensive but practical design rules for long-span steel box girder highway bridges. The rules also provide a potential basis for a uniform set of specifications for steel box girders of all span lengths. The report includes background information, the specifications, commentary on the specifications, and an annotated bibliography.


The proposed design specifications use the load factor design method and are presented in the standard AASHTO format. The specifications are now under review by the AASHTO Subcommittee on Bridges and Structures for possible inclusion in the 1980 AASHTO Interim Specifications.

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 80 162324).

## Hydraulic Design of Bridges With Risk Analysis, Report No. FHWA-TS-80-226

**by FHWA Implementation Division**



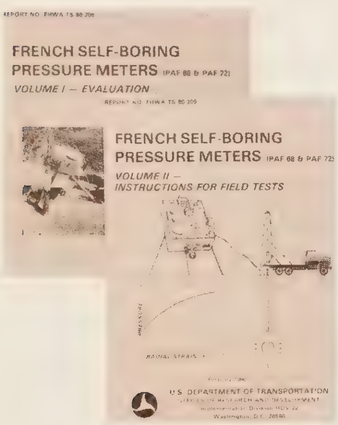

The hydraulic design of bridges involves an evaluation of the flood hazard to the highway and the effect of the proposed highway on lives, property, and stream stability. This manual presents example studies and reports that indicate that the total stream crossing, including the approach fill in the flood plains and all necessary waterway openings, should be designed and constructed for the least total expected cost to the public. The total expected cost to the public during the service life of the highway includes the capital investment in the highway, expected replacement and repair costs as a result of flood damages, expected user costs from traffic interruptions and detours, and expected backwater damages. Techniques for making engineering and economic studies for the least cost designs are presented with suggestions for managing the time and work required for these studies. A unique design flood is defined for each bridge as the flood whose upstream stage is equal to the lowest elevation of the approach fill or bridge deck.

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

### French Self-Boring Pressure Meters (PAF 68 and PAF 72), Volumes I and II, Report No. FHWA-TS-80-209

**by FHWA Implementation Division**

The pressure meter is a soil exploration tool that determines the strength of soft soils inplace without undisturbed sampling and laboratory tests. The self-boring concept advanced the practice of pressure meter testing by eliminating the need for a predrilled borehole; the self-boring pressure meter probe drills its own precisely sized borehole with minimal disturbance to the surrounding soil.



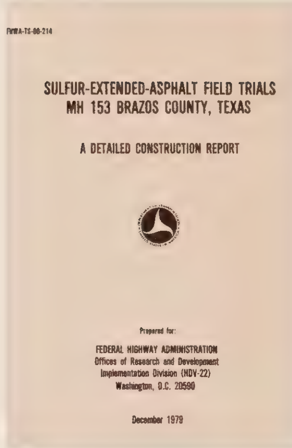

These reports include an evaluation of the PAF 68 and PAF 72 self-boring pressure meters and comprehensive instructions for running the pressure meter test. Although the self-boring technique shows potential for application in geotechnical engineering, it is considered a specialty technique and is still being evaluated.

Volume I, **Evaluation,** discusses the concept and practice of self-boring pressure meter tests. The evaluation project is presented in detail and the potential of the self-boring pressure meter as a soil exploration tool is reviewed. Applications of the self-boring pressure meter test to various geotechnical problems are discussed and recommended.

Volume II, **Instructions for Field Tests,** reviews preparations and procedures necessary to run a pressure meter test and to translate the resulting data into reliable soil parameters. The adaption of a probe-console system to different drill rigs is detailed and the field test procedures are discussed.

Of the three methods of test data reduction and analysis presented, a graphical method is proposed. Computer programs for data reduction are also presented. This volume includes a comparison of test results from several different probes and a discussion of soil disturbances during the self-boring pressure meter test.

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




### Sulphur-Extended-Asphalt Field Trials MH 153 Brazos County, Texas, Report No. FHWA-TS-80-214

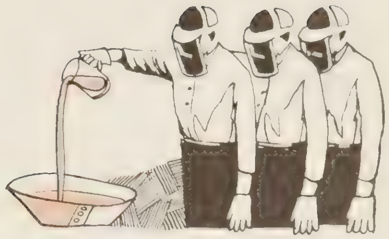
**by FHWA Implementation Division**

The sulphur in sulphur-extended-asphalt (SEA) binders replaces a part of the asphalt used in conventional asphalt paving and seems to enhance the strength and fatigue characteristics of the asphalt pavement mix. The increasing cost and potential shortage of asphalt have accelerated use of these binders.

This report contains the most comprehensive published description of equipment, materials, quality control procedures, and construction procedures used during the construction of pavement sections with SEA binders. The report also highlights design procedures and emissions control monitoring used before and during construction.

The report may be purchased for $3.75 from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Stock No. 050-001-00165-9).

# New Research in Progress



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

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

### FCP Project 2P: Improved Utilization of Available Freeway Lanes

**Title: Objectives and Parameters Associated With High Occupancy Vehicle Treatments. (FCP No. 42P3063)**
**Objective:** Identify the objectives associated with each high occupancy vehicle treatment and determine how effectively the objectives will be met. Develop a manual to be used to analyze data to make recommendations for high occupancy vehicle applications.
**Performing Organization:** New Jersey Department of Transportation, Trenton, N.J. 08625
**Expected Completion Date:** December 1983
**Estimated Cost:** $98,000 (HP&R)

## FCP Category 4—Improved Materials Utilization and Durability

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

**Title: Materials Characterization of Recycled Bituminous Paving Mixtures. (FCP No. 34C4063)**
**Objective:** Develop optimum mixture designs and evaluate their structural properties. Determine the effects of the environment, aging, and traffic. Evaluate road performance from field samples. Design alternative structural sections and predict performance using VESYS IIM. Develop interim guidelines for mixture and evaluate structural engineering properties.
**Performing Organization:** Resource International, Worthington, Ohio 43085
**Expected Completion Date:** January 1983
**Estimated Cost:** $134,000 (FHWA Administrative Contract)

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

**Title: Correlation of Quality Control Criteria and Performance of Portland Cement Concrete Pavements. (FCP No. 34F3352)**
**Objective:** Establish whether and how particular quality control tests, measurements, and observations relate to the future performance of portland cement concrete pavements and how much deficiencies indicated by particular test results will affect pavement life. Examine strength, density, air content, water/cement ratio, gradation, temperatures, steel placement, and joint construction.
**Performing Organization:** Resource International, Worthington, Ohio 43085
**Expected Completion Date:** January 1982
**Estimated Cost:** $152,000 (FHWA Administrative Contract)

### FCP Project 4K: Cost Effective Rigid Concrete Construction and Rehabilitation in Adverse Environments

**Title: Long Term Rehabilitation of Salt-Contaminated Bridge Decks. (FCP No. 54K2092)**
**Objective:** Develop and evaluate materials and procedures in conjunction with low-permeability overlays to prevent continued corrosion of reinforced steel surrounded by chloride-contaminated concrete bridge decks.
**Performing Organization:** Lehigh University, Bethlehem, Pa. 18015
**Expected Completion Date:** September 1982
**Estimated Cost:** $200,000 (NCHRP)

**Title: Slump Loss and Retempering of Superplasticized (Portland Cement) Concrete. (FCP No. 44K2304)**
**Objective:** Evaluate by a three-phase laboratory study slump loss and retempering, cement admixtures interactions, and freeze-thaw durability. Use different cements and admixtures. Try fly ash and use mini-slump tests, rheological tests, kinetics of reactions, adsorption, and microscopic tests.
**Performing Organization:** University of Illinois, Urbana, Ill. 61801
**Funding Agency:** Illinois Department of Transportation
**Expected Completion Date:** October 1982
**Estimated Cost:** $80,000 (HP&R)

**Title: Evaluation of the Construction and Initial Performance of Seven Polymer Concrete Overlays. (FCP No. 44K2324)**
**Objective:** Construct under contract seven polymer concrete overlays (BNL method) on I-64 near Williamsburg, Va. The Virginia Research Council will do the testing and evaluation. Technical assistance will be given by the Implementation Division (HDV-22) and Brookhaven National Laboratory.
**Performing Organization:** Virginia Department of Highways and Transportation, Richmond, Va. 23219
**Expected Completion Date:** December 1985
**Estimated Cost:** $84,000 (HP&R)

**Title: Corrosion Prevention of Reinforcing Steel in Concrete Exposed to Marine Environments. (FCP No. 44K3152)**
**Objective:** Examine and correlate by a field and laboratory study the performance of systems containing different concrete mixtures, reinforcement, claddings, and coatings. Study voltage potentials in conjunction with cathodic protection, anodes, and conductive concretes. Measure chloride contents.
**Performing Organization:** Florida Department of Transportation, Tallahassee, Fla. 32304
**Expected Completion Date:** September 1983
**Estimated Cost:** $240,000 (HP&R)

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

### FCP Project 5D: Structural Rehabilitation of Pavement Systems

**Title: Stress-Relieving Interlayers for Bituminous Pavements. (FCP No. 45D2664)**
**Objective:** Install and evaluate the effectiveness of seven different reflective cracking SRI measures in six test sections. Examine two different crack types, crack densities, and overlay thicknesses.
**Performing Organization:** New York State Department of Transportation, Albany, N.Y. 12226
**Expected Completion Date:** March 1985
**Estimated Cost:** $112,000 (HP&R)

**Title: Development of Design Procedures for ACHM Overlays on PCC Pavements. (FCP No. 45D2674)**
**Objective:** Collect field data and materials testing data, analyze the data with mechanistic analysis, develop a procedure for designing against reflection cracking, and verify this with additional data.
**Performing Organization:** University of Arkansas, Fayetteville, Ark. 72701
**Funding Agency:** Arkansas State Highway and Transportation Department
**Expected Completion Date:** January 1982
**Estimated Cost:** $156,000 (HP&R)

### FCP Project 5K: New Bridge Design Concepts

**Title: Connections for Modular Precast Concrete Bridge Decks. (FCP No. 35K3022)**
**Objective:** Develop new and improved connections for modular precast concrete bridge decks. Prepare recommendations for using these connections in the design and construction of highway bridges.
**Performing Organization:** Consulting Engineer Group, Glenview, Ill. 60025
**Expected Completion Date:** January 1982
**Estimated Cost:** $183,000 (FHWA Administrative Contract)

## FCP Category 0—Other New Studies

**Title: Develop a Maintenance Management System for Programing the Replacement of Reflective Pavement Markers. (FCP No. 40F0054)**
**Objective:** Determine if the reflectivity of reflective pavement markers can be measured in the field on a production basis, if it is possible to establish a lower limit of reflectivity, and if it is possible, determine the lower limit. Design a program for replacing reflective pavement markers.
**Performing Organization:** California Department of Transportation, Sacramento, Calif. 95814
**Expected Completion Date:** June 1982
**Estimated Cost:** $190,000 (HP&R)

# Instructions to Authors

**All articles proposed for publication in *Public Roads* magazine are reviewed for suitability by the technical editors. Authors will be notified of acceptance or rejection as soon as possible.**

**Recent issues of the magazine should be reviewed for kind of articles, style, illustrations, tables, references, and footnotes. *Public Roads* follows the U.S. Government Printing Office Style Manual.**

## Submission of Manuscripts

Authors in the Washington, D.C., area should submit two copies of the manuscript to the managing editor.

Managing Editor (HDV–10)
*Public Roads* Magazine
U.S. Department of Transportation
Federal Highway Administration
Washington, D.C. 20590

Authors outside the Federal Government, or in State, city, or local government agencies, should submit two copies of the manuscript through appropriate Federal Highway Administration regional offices (see page 96).

## Manuscript Treatment

Manuscripts should be typewritten, double spaced, with at least 1-inch margins on 8½- by 11-inch paper. Excluding art, 1 magazine page requires about 3 pages of manuscript. End each page with a completed paragraph. Type main headings flush left in initial caps. Subheadings should be flush left, and the first letter only capitalized. The article title and the name of each author should be typed on a separate page. If the article has been presented at a meeting, that should be indicated in a footnote at the bottom of the title page. Each page of the text should be numbered in the upper right margin. Because the Federal Government does not endorse products of manufacturers, avoid trademarks and brand names in articles unless their use is directly required by the objectives of the article.

## Biography

A brief biography should be supplied. This should include the author's present position and responsibilities and previous positions relevant to the subject matter of the article. Biographies are limited to approximately 100 words. Biographies are accompanied by sketches of the authors drawn from photographs. Send a photograph of each author to the editor when the author is notified that the article has been accepted for publication.

## Abstract

An abstract should be supplied with technical articles. For a *development paper,* the abstract should tell: (1) What has been accomplished, (2) its outstanding features, and (3) its applications, if known. The abstract of a *research paper* should focus on: (1) What has been accomplished, (2) its most important facts and implications, and (3) logical steps open to study.

## Tables

Nonessential technical tables should not be included in the article. Each table should be typed on a separate page. It should be identified by an Arabic number and a caption. Note the location of the table in the text in the margin, but avoid putting it on the first page. Details of data already presented in tables or charts should not be repeated in the text.

## Illustrations

Illustrations *referenced in the text* are called figures, and numbers and captions should be assigned to each. Organize the text so that illustrations can be scattered throughout the article. Avoid referencing several illustrations in one page to prevent problems in the layout. Black and white glossy photographs of good quality are preferred; however, color photographs and art are acceptable. Send original artwork to the editor when the author is notified that the article has been accepted for publication. Send legible copies with the manuscript. All captions (numbered and unnumbered) should be typed underlined on a separate page.

## Metrication

Under present law and FHWA regulation, *Public Roads* is required to show the English equivalent in parentheses after every metric (SI) unit expressed in the text. In figures and tables, indicate equivalent units in a legend (for example, 1 km=0.6 mile); it is not necessary to show metric or English equivalents for each point plotted or quantity tabulated.

## References

Number references consecutively in the body of the text, enclosed in parentheses and underlined. Copyrighted material referenced and quoted will require copyright releases. Unpublished material referenced in the text will be described in a footnote. Type citations on a separate page under the heading REFERENCES. Number citations in the same manner as in the text and list in the same sequence.

## Galley Proofs

Galley proofs will be sent to authors for their inspection.

For further information, contact the editor: (703) 557-4304.

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# THE NATIONAL SYSTEM OF INTERSTATE AND DEFENSE HIGHWAYS

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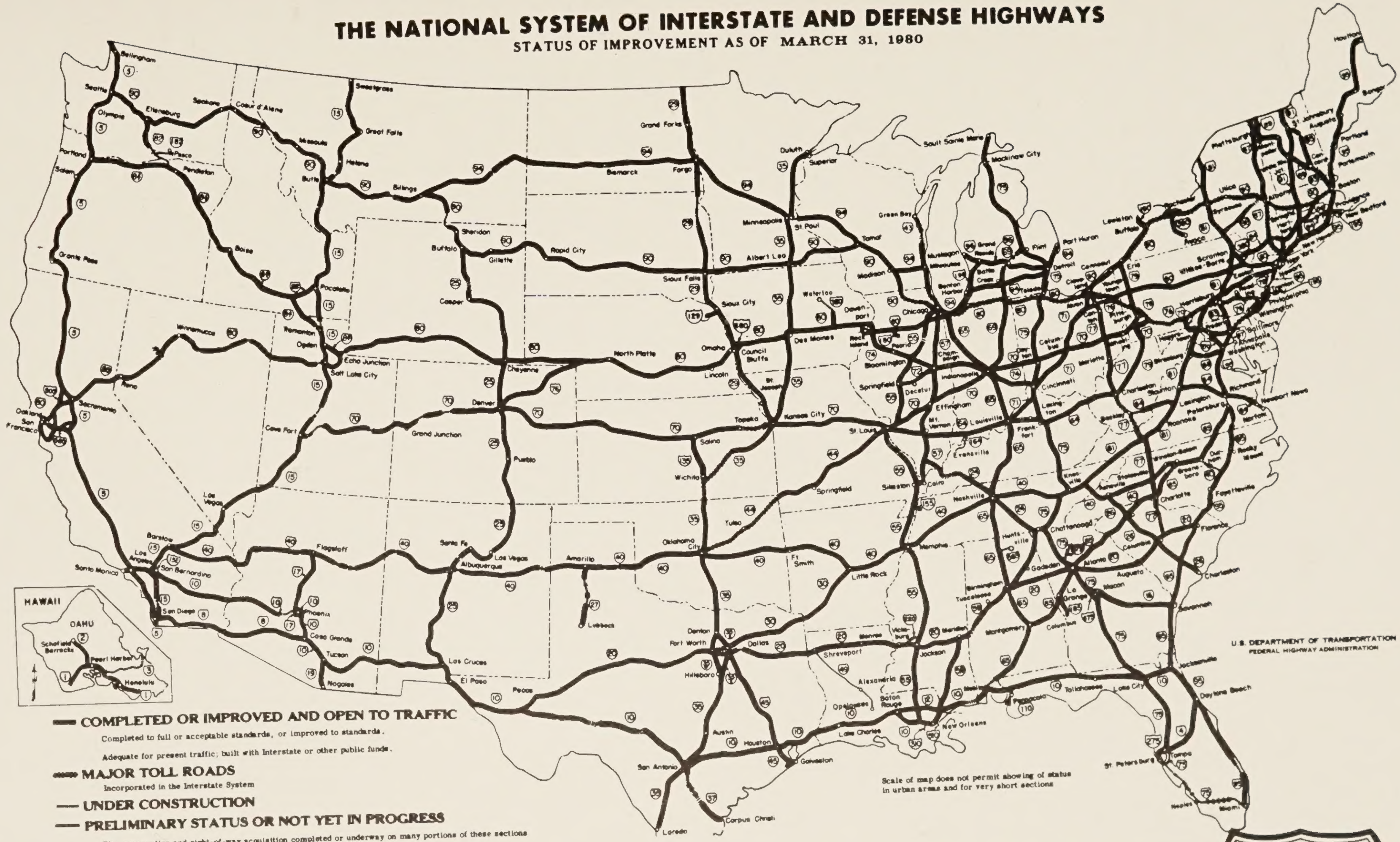



**COMPLETED OR IMPROVED AND OPEN TO TRAFFIC**

Completed to full or acceptable standards, or improved to standards.

Adequate for present traffic; built with Interstate or other public funds.

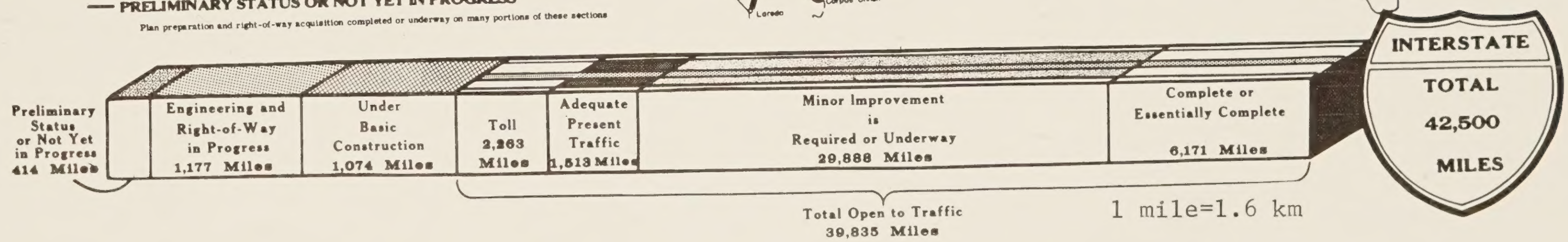
**MAJOR TOLL ROADS**

Incorporated in the Interstate System

**UNDER CONSTRUCTION**

**PRELIMINARY STATUS OR NOT YET IN PROGRESS**

Plan preparation and right-of-way acquisition completed or underway on many portions of these sections

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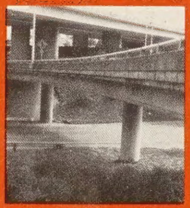
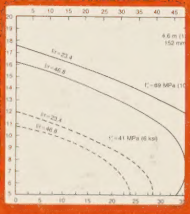
**Process Control for Aggregate Production and Use**



**Bridge Movements and Their Effects**



**Applications of High Strength Concrete for Highway Bridges**

public roads
A JOURNAL OF HIGHWAY RESEARCH AND DEVELOPMENT