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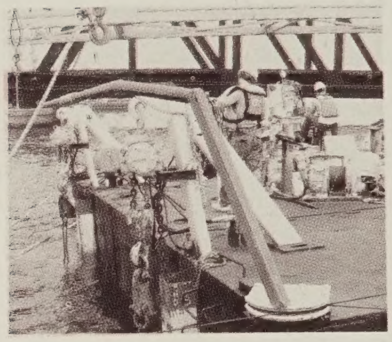

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COVER: Installation of Gabion mattresses to protect bridge piers from scour at the Acosta Bridge in Jacksonville Florida. Gabion mattresses are wire enclosed baskets of small stones that are sometimes used in lieu of large diameter rock riprap as a countermeasure against scour and erosion.

U.S. Department of Transportation
Samuel K. Skinner, *Secretary*

Federal Highway Administration
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U.S. Department of Transportation
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IN THIS ISSUE

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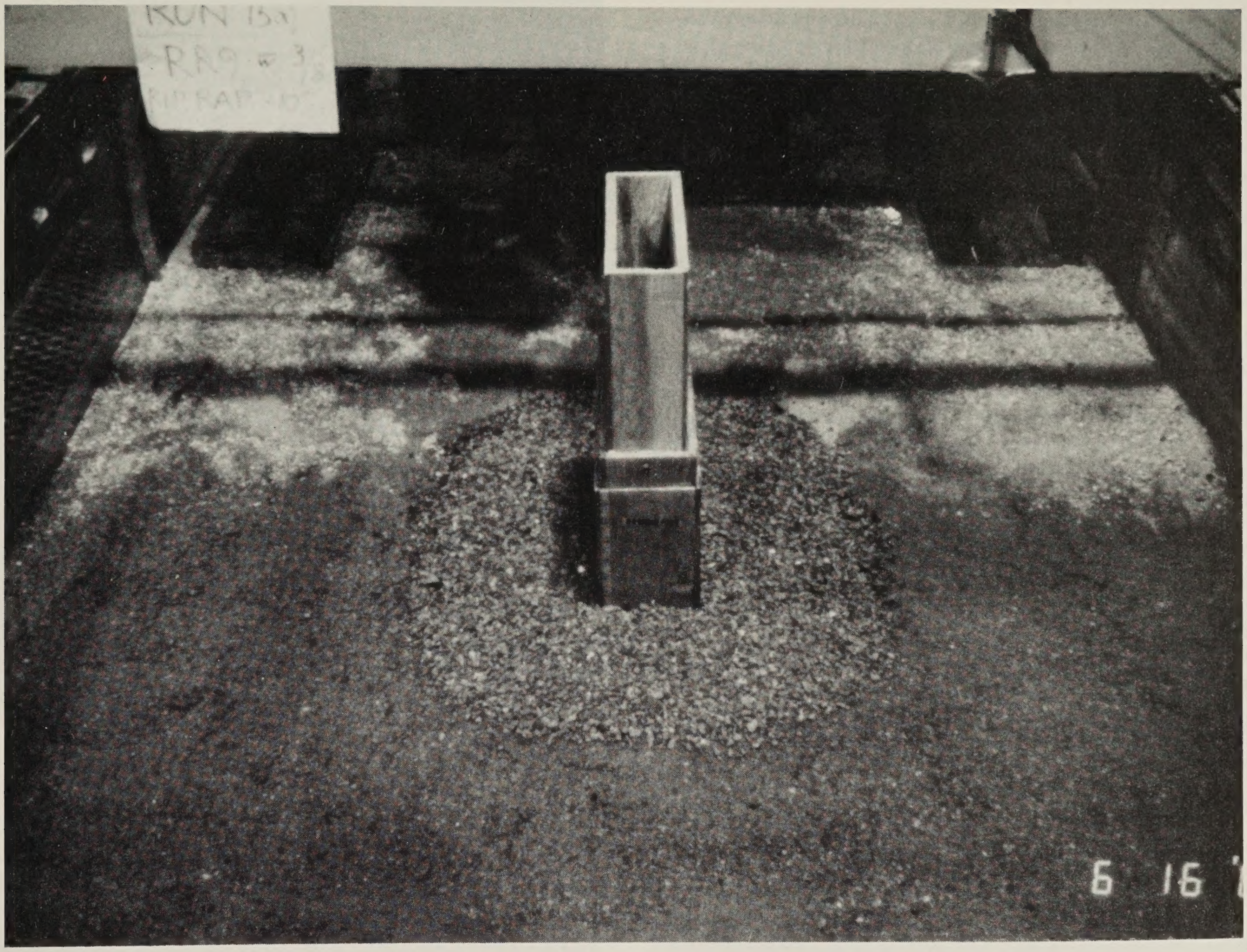


Design of Riprap to Protect Bridge Piers from Local Scour

by Frank Graziano, J. Sterling Jones, and Arthur C. Parola

Introduction

Several recent scour-related bridge catastrophes have sparked an interest in bridge scour and specifically in developing methods of protecting bridge piers from scour. The 1987 collapse of the Schoharie Creek Bridge in New York, the April 1989 collapse of the Hatchie River Bridge in Tennessee, the June 1989 collapse of the temporary bridge in Miamistown, Ohio, and the 1990 closure of the I-95 Bridge over the Altamaha River in Georgia were all—in part—scour related. Nine people were killed in the New York disaster and eight in Tennessee.

Even before these tragedies focused national attention on the scour problem, however, highway engineers had struggled with the problem of protecting bridge piers from scour. Such protection was especially needed where an existing scour hole had to be stabilized before it deepened and became a threat to the bridge foundation.

Although rock riprap has been used for years to protect bridge piers from scour, until quite recently, the Federal Highway Administration (FHWA) had no criteria for designing it. This laboratory study, initiated as part of the FHWA-sponsored Graduate Research Fellowship program, undertook the development of such criteria.[1] Specifically, interim guidelines for designing bridge pier riprap protection were

[1]The Graduate Research Fellowship program is a highly successful enterprise in which university students spend a portion of their academic training at the FHWA's Turner-Fairbank Highway Research Center.

developed during the early phase of this study for FHWA *Hydraulic Engineering Circular 11* (*HEC-11*) "Design of Riprap Revetment," and the draft version of *HEC-18* "Evaluating Scour at Bridges."(*1,2*)[2] This article presents the final results of the study and may be viewed as the basis for revising some of the interim guidelines.

The draft version of *HEC-18* was distributed to the FHWA field offices and to State highway agencies as an attachment to the FHWA *Technical Advisory T5140.20*, "Scour at Bridges."(*3*)

FHWA Interim Procedures

The interim procedures for designing riprap as bridge pier protection are an adaptation of work done approximately 50 years ago by Shields and Isbash. (*4,5*) Shields developed the tractive force theory for the initiation of particle motion. Isbash was a Russian scientist who developed simpler velocity criteria for particle stability based on observations of stones deposited on the crest of a submerged dam. Prior to the present study, there was no comprehensive data set to either support or refute the adaptations of these methods for pier protection.

The Shields theory

Shields' theory on the initiation of riprap particle motion can be expressed as follows:

$$D_r = \frac{U_*^2}{(SP)\,(g)\,(S_s-1)} \qquad (1)$$

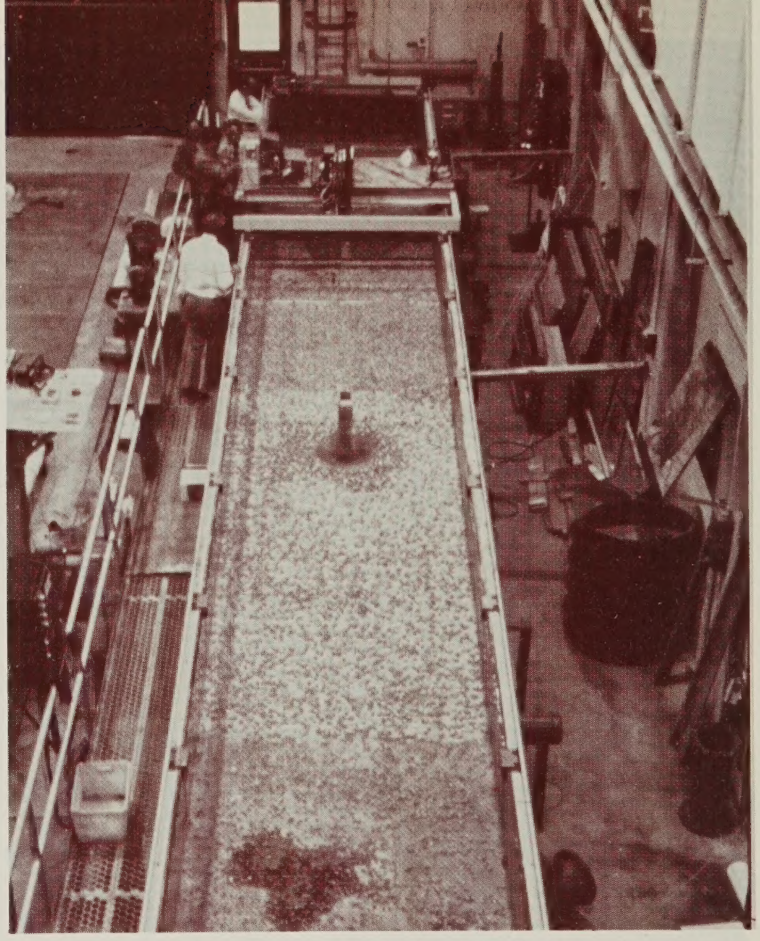
Where:

- D_r = riprap size
- U_* = bed shear velocity
- SP = Shields parameter (usually assumed to be 0.047 for riprap design)
- S_s = specific gravity of the riprap (usually 2.65)
- g = acceleration of gravity

Although his expression is theoretically sound, the bed shear velocity parameter is difficult to determine.

The FHWA's *HEC-11* is based on this expression but was modified to exclude the shear velocity. The modified version in *HEC-11* is:

$$D_r = [0.001\ V_a 3/Y^{0.5}\ K_1 1.5]\ C_{SG} C_{SF} C_{P/A} \qquad (2)$$



A laboratory study was initiated in the FHWA's hydraulics laboratory. The pier model near the center of the flume has a preformed scour hole. This hole was lined with riprap of various sizes, each of which were tested to failure.

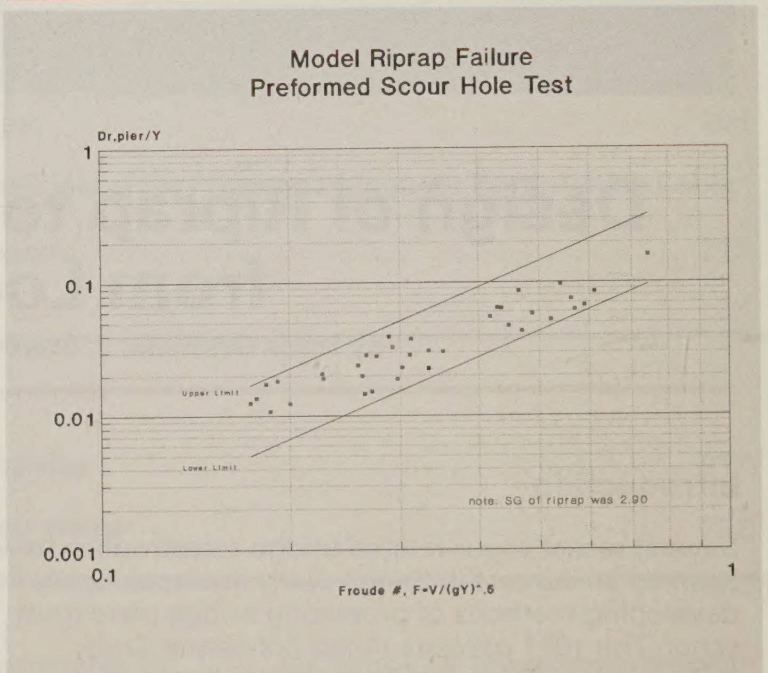


Figure 1.—Prediction limits from equations (5) and (6) compared to experimental data.

[2] Italic numbers in parentheses identify references on page 198.

Where:

V_a = average velocity approaching a pier
Y = depth of flow
K_1 = side slope correction = 1.0 for a flat bed
C_{SG} = specific gravity adjustment = 1.0 for specific gravity of 2.65
C_{SF} = stability factor (SF) adjustment for SF other than 1.2
$C_{P/A}$ = pier adjustment factor (3.38 is recommended)

The pier adjustment factor is simply a velocity multiplier to account for acceleration around the pier. A value of 1.5 was recommended as a multiplier; therefore, $C_{P/A}$ is $(1.5)^3$ or 3.38.

Unfortunately, the *modified* Shields' expression for riprap around piers inherently assumes that the riprap being designed defines the roughness of the flow regime. This assumption, while valid for riprapping a reach of channel, is not quite valid for a very local area of riprap around a bridge pier. This assumption is partly the reason that the velocity is a cubic rather than a squared term; its existence makes use of the modified expression questionable.

The Isbash theory

The Isbash expression for riprap stability can be written as follows:

$$D_r = C_E \frac{V^2}{2g(S_S-1)} \tag{3}$$

Where:

V = flow velocity
C_E = a dimensionless coefficient (Isbash's coefficient, E, inverted and squared)
= 1.38 for loose stones lying on top of a fill that begin to move
= 0.69 for stones lying among others provide some protection and will roll until they find a seat to become stable

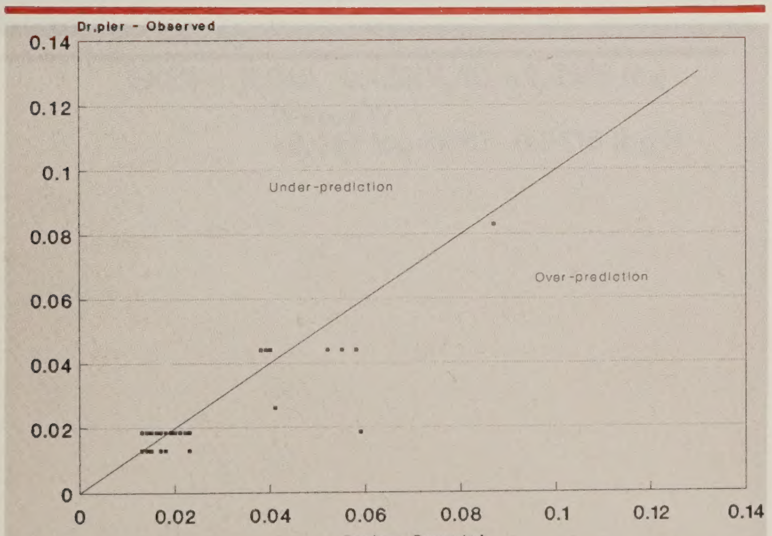


Figure 2.—Riprap size observed versus riprap size predicted from equations (5) and (6) using SF = .03 in equation (6).

The *Technical Advisory* and *HEC-18* essentially adapted this expression for designing riprap for pier protection. In so doing, the authors of these needed to decide which coefficient and what flow velocity to use. The authors of the *Technical Advisory* first recommended using the 1.38 coefficient; after further consideration, however, they decided that some rolling of stones in a blanket of riprap would not jeopardize the integrity of the protection. The November 7, 1988 addendum, thus changed this coefficient from 1.38 to 0.69.

A more critical issue was selecting the flow velocity to use in the equation. The *Technical Advisory* authors recommended that a "velocity near the stone" be used—i.e., use the portion of the velocity distribution that is near the bed. For pier protection, the authors of *HEC-18* recommended multiplying the approach velocity by between 1.5 and 2.0 to get the appropriate flow velocity for the equation. The *HEC-18* equation for bridge pier riprap design then can be written:

$$D_{r,pier} = 0.69\,(K_{TA}\,V_a)^2/2g(S_s-1) \tag{4}$$

Where:

$D_{r,pier}$ = riprap size for pier protection
K_{TA} = multiplier to account for acceleration around the pier (values from 1.5 to 2.0 recommended)

This laboratory study compared these procedures against experimental data, as detailed below (figure 3).

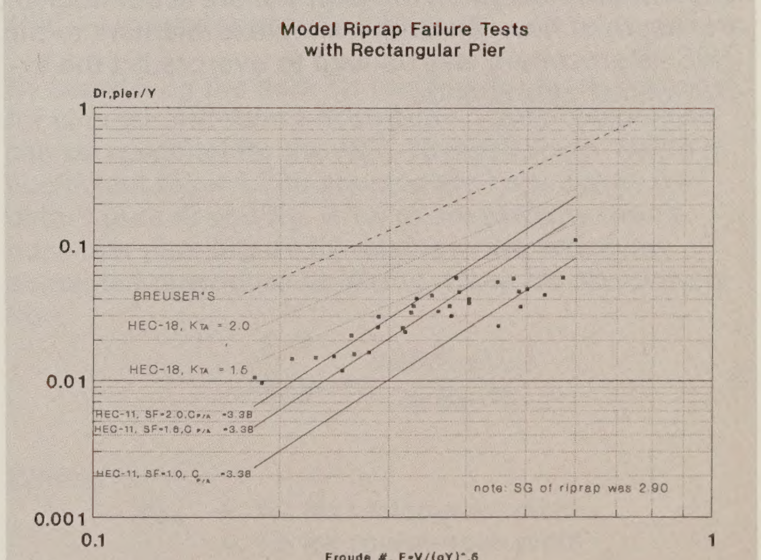


Figure 3.—Comparison of experimental data to existing procedures.

Experimental Setup

The experiments were conducted in a 70-ft (21.3-m) long by 6-ft (1.8-m) wide flume with piers set on a flat bed as well as in preformed scour holes. Three pier widths—0.167, 0.375, and 0.500 ft (0.05, 0.11, and 0.25 m)—were used in the flat bed tests. One rectangular pier 0.375 ft (0.11 m) wide by 0.833 ft (0.25 m) long was used for the preformed scour hole tests. The piers were anchored in a sediment recess located approximately 40 ft (12.2 m) from the beginning of the flume. Preformed scour holes were formed by preliminary runs with no riprap around the pier. The holes were then fixed with a coating of epoxy glue to keep them constant during the riprap tests.

Each experimental run—whether flat bed or preformed scour hole— was conducted in the same way. Model riprap was placed in three layers with the middle layer painted fluorescent orange. A specified flow rate was pumped through the flume with a moveable tailgate that was preset initially to ensure that the riprap was not disturbed. The tailgate was then lowered slowly until the painted layer of riprap was exposed but not displaced.

The riprap was considered to have failed if a patch of the painted layer was visible. Flow characteristics at failure, including depths and velocities, were then measured.

Experimental Results

The experiments yielded the following results; these are presented at two levels of complexity. First, a regression equation was derived. This regression equation accounts for all the variables studied in the experiments including the depth of the scour hole and the depth of flow. Second, a simple refinement to the *HEC-18* procedure was derived to overpredict the experimental data slightly.

The regression equation as presented in equation (1) can be rearranged as follows to solve for bridge pier riprap size: (6)

$$D_{r,\ pier} = \frac{V_a^2\ 10^{EXP}}{(g)\ (S_s-1)} \qquad (5)$$

Where:

- EXP = exponent based on a regression of the experimental data
- EXP = $0.040 - 0.433(Y_s/B) + 0.0566 \log (D_{s,bed}/Y) + 0.142 \log (Y/B) + 0.0116 \log (D_{r,pier}/Y)$
- Y_s = depth of preformed scour hole
- B = pier width
- $D_{s,bed}$ = bed sediment size for approach channel
- $D_{r,pier}$ = riprap size at the pier
- Y = depth of flow

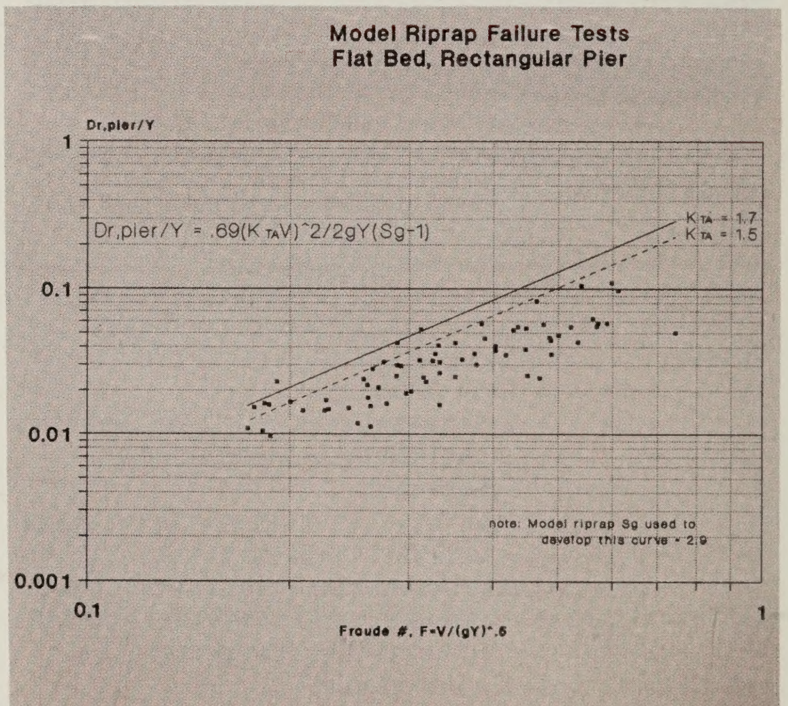


Figure 4.—Riprap failure data for rectangular piers on a flat bed.

The experimental limits of parameters used in the derivation are as follows:

Y_s/B from 0.11 to 0.83
Y/B from 0.86 to 3.77
$D_{s,bed}/Y$ from 0.004 to 0.38
$D_{r,bed}/B$ from 0.035 to 0.22

Equation (5) was derived from the preformed scour hole data and included rectangular pier shape only. It would be reasonable to apply an adjustment for a round-nose pier. The equation can be applied to flat bed conditions without much loss of accuracy because the shallow scour holes had very little effect on the riprap stability.

By inputting critical combinations of the experimental limits in equation (5), a graphic comparison with experimental data can be developed. These combinations yield the following limits for equation (5):

$$\text{upper limit } D_{r,pier} = 1.10 \frac{V_a^2}{g(S_S-1)}$$

$$\text{lower limit } D_{r,pier} = 0.330 \frac{V_a^2}{g(S_S-1)}$$

As shown in figure 1, the experimental data fall within the limits of equation (5) predictions. Also, the *HEC-18* expression, equation (4), falls right in the middle of these limits when a K_{TA} value of 1.5 is selected. Figure 1 does not illustrate whether equation (5) tends to overpredict or underpredict individual data points. Figure 2, however, does show that equation (5) overpredicts most of the data points even without a safety factor applied to the equation.

One criticism of the above procedure is that equation (5) requires a trial and error solution since the riprap size is on both sides of the equation. This problem can be eliminated by mathematically rearranging terms to yield the following:

$$D_{r,pier} = \left[\frac{V_a^2}{(g)(S_S-1)}\right]^{1.012} 10^{1.012\text{EXP2}} \qquad (6)$$

Where:

$$\text{EXP2} = 0.040 - 0.433(Y_s/B) + 0.0566 \log (D_{s,bed}/Y) + 0.142 \log (Y/B) - 0.0116 \log B$$

Parola's original expression from which equation (5) was developed was dimensionless and could be used with either the English or metric system of units. (*6*) Likewise, equation (5) and the expressions developed from it in this article can be used with either system of units—if units are used consistently.

While the above procedure provides a good prediction of riprap size and has a term for downsizing riprap when there is an existing scour hole to be stabilized, the procedure may be too detailed for determining riprap of a flat bed without a preformed scour hole. As noted earlier, the procedure can be used for flat bed conditions, but the regression coefficients were statistically weak due to large scatter in the data for all terms except scour depth. Thus, a simpler procedure—similar to the *HEC-18* interim guidelines—was developed.

Figure 3 illustrates how the existing procedure compares with the experimental data. The various lines for *HEC-11* and *HEC-18* represent different coefficients that a designer might select. The line marked "Brueser's" represents a procedure which was not endorsed by the FHWA but which is in the literature and is used by some designers. (*7*)

Of the three procedures, the *HEC-18* interim procedure compares best with the experimental data. More specific guidelines are needed for selecting the velocity adjustment coefficient, K_{TA}, but the general agreement with the experimental data is good.

Brueser's procedure is overly conservative based on these data and the *HEC-11* special procedure for pier protection using the $C_{P/A}$ factor does not follow the trend of the data. Based on these observations, it is likely that this special procedure will be deleted from *HEC-11* in subsequent revisions.

By separating the data for rectangular piers from that for circular and round-nose piers, tighter guidelines can be specified for the *HEC-18* procedures. Using a coefficient $K_{TA} = 1.7$ to envelop the rectangular pier data (figure 4) and $K_{TA} = 1.5$ to envelop the round-nose pier data (figure 5), results in the following simplified procedure for sizing riprap for pier protection:

$$D_{r,pier} = \frac{0.69(K_{TA}V_a)^2}{2g(S_S-1)} \qquad (7)$$

Where:

K_{TA} = 1.7 for rectangular piers
= 1.5 for round-nose piers

Given the lack of better data, the ratio of riprap size required for rounded piers to the size required for rectangular piers can be assumed to be the same in the detailed procedure as it is in the simplified procedure. Consequently, a reasonable adjustment for the detailed procedure in equation (5) when extended to rounded piers would be to multiply the size by $(1.5/1.7)^2$ or 0.78.

Conclusions

Based on experimental data, two procedures were developed in this study to determine the size of riprap needed to protect bridge piers from local scour. The primary procedure developed is represented by equations (5) and (6). This procedure accounts for the influence of a preexisting scour hole which tends to downsize the riprap requirements. Data used to derive equation (5) were limited to a rectangular pier shape, but other data were used to estimate an adjustment for rounded piers. If equation (5) is used for rounded piers, the estimated adjustment factor for the riprap size is 0.78 (approximately 80 percent). The second procedure, represented by equation (7), is a refinement of the interim guidelines in the FHWA's *HEC-18*. This procedure is simpler and is applicable for most bridge piers where the riprap final placement will be approximately flush with the streambed. The procedure does not, however, account for the influence of a preexisting scour hole around a pier.

Closeup of a rectangular pier with model riprap placed flush with the original streambed elevation.

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(*1*) "Design of Riprap Revetment," *Hydraulic Engineering Circular 11*, Publication No. FHWA-IP-89-016, Federal Highway Administration, Washington, DC, March 1989.

(*2*) "Evaluating Scour at Bridges," *Hydraulic Engineering Circular 18*, Federal Highway Administration, Washington, DC (draft version, unpublished).

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(*4*) A. Shields. *Application of Similarity Principles and Turbulence to Bed-Load Movement*, translated by W.P. Ott and J.C. van Uchelon, Publication 167, California Institute of Technology, 1936.

(*5*) S.V. Isbash. "Construction of Dams by Depositing Rock on Running Water," *Transactions of Second Congress on Large Dams*, Vol. V, pp. 123-136, Washington, DC, 1936.

(*6*) A.C. Parola, Jr. "The Stability of Riprap Used to Protect Bridge Piers," Ph.D. dissertation, Pennsylvania State University, May 1990.

(*7*) A.C. Parola and F. Graziano. "Sizing Riprap to Protect Bridge Pier Foundations," paper presented at the Geotechnical Engineers in Today's Environment Conference, sponsored by the Commonwealth of Pennsylvania Department of Transportation and the Central Pennsylvania Section of the American Society of Civil Engineers, Hershey, PA, April 10-11, 1990.

(*8*) M.E. Quazi and A.W. Peterson. "A Method for Bridge Pier Riprap Design," paper presented at the First Canadian Hydraulics Conference, University of Alberta, Edmonton, Canada, 1973.

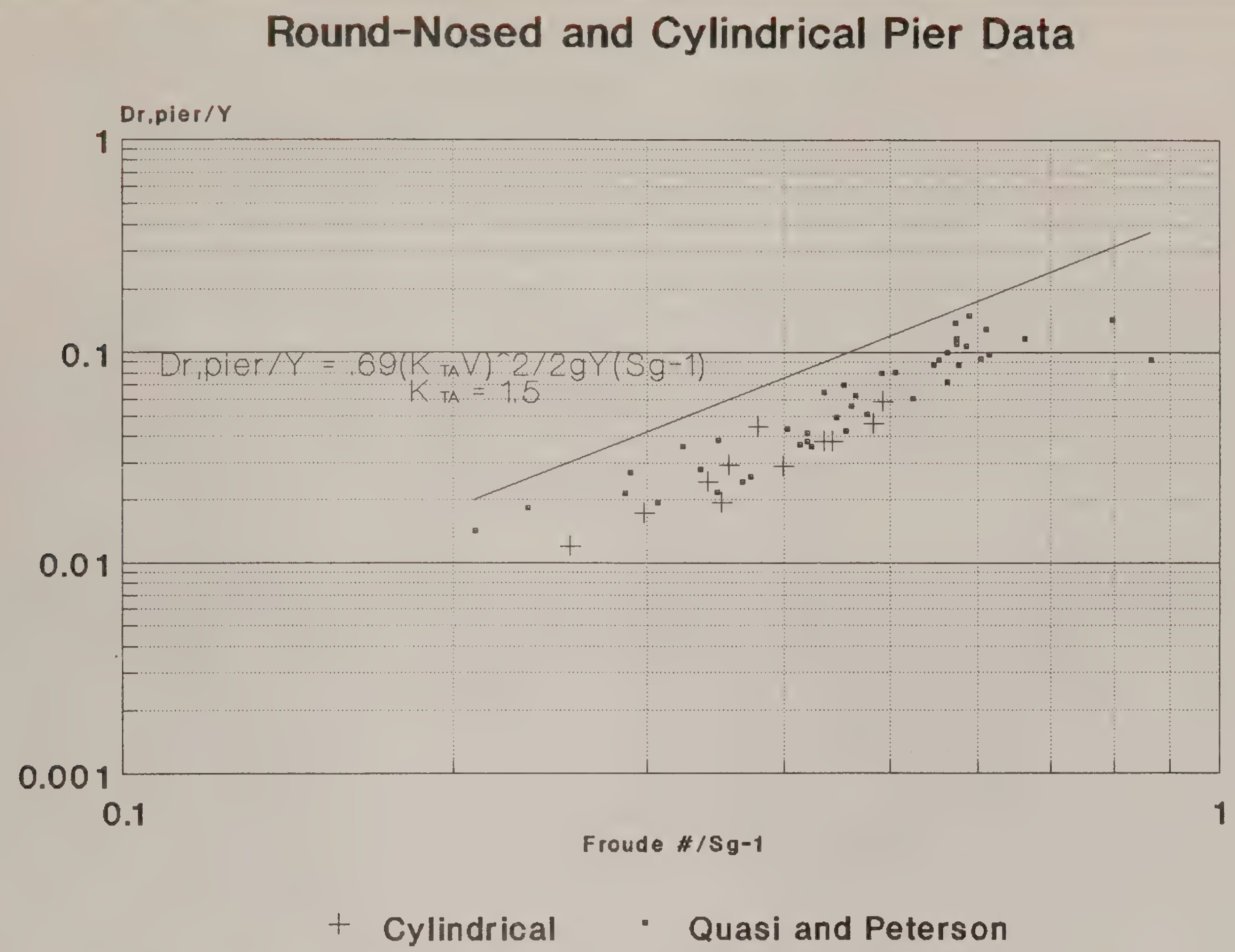
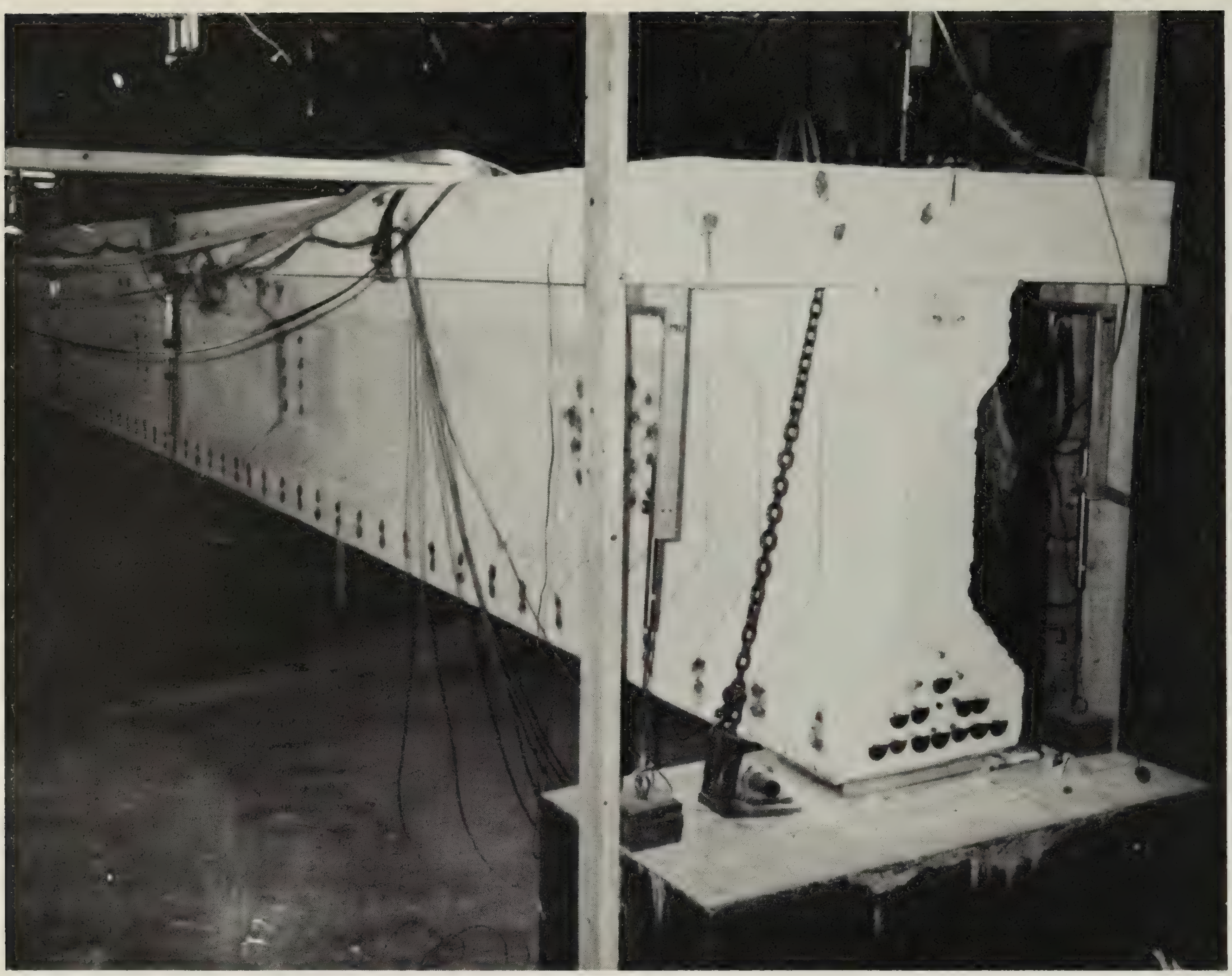


Figure 5.—Riprap failure data for rounded piers on a flat bed. (8)

Frank Graziano is a staff engineer with Ecological Analysis, Inc. in Baltimore, Maryland. At the time of this study, he was a research engineer assigned to the Federal Highway Administration (FHWA) hydraulics laboratory by GKY and Associates in Springfield, Virginia.

J. Sterling Jones is a hydraulics research engineer with the FHWA. He currently manages the FHWA hydraulics laboratory in McLean, Virginia.

Arthur C. Parola is an assistant professor at the University of Louisville in Louisville, Kentucky. At the time he conducted this study, he was a doctoral candidate from Pennsylvania State University participating in the FHWA's Graduate Research Fellowship program at the FHWA hydraulics laboratory.



Development Length of Prestressing Strand

by Susan N. Lane

Introduction

On October 26, 1988, the Federal Highway Administration (FHWA) issued a memorandum concerning the use of prestressing strand in a pretensioned application for prestressed concrete bridges. The memorandum:

Disallowed the use of 0.6-in (15.2-mm) diameter strand in a pretensioned application.

Restricted the minimum strand spacing.

Increased the required development length for fully bonded and debonded strand by 1.6 and 2.0 times the American Association of State Highway and Transportation Officials (AASHTO) equation number 9–32, respectively. (*1*)[1]

The FHWA memorandum indicated that its restrictions were adopted only as an interim measure, until research results indicate otherwise and AASHTO adopts the results.

[1] Italic numbers in parentheses identify references on page 205.

Because of the FHWA memorandum, development length of prestressing strand has become a widely discussed issue. Numerous research studies on this topic are under way or have been recently completed by both the public and private sectors. This article briefly summarizes these studies.

What Is Development Length?

There are two types of prestressing: pretensioning and post-tensioning. When a structural member is prestressed using *pretensioning*, the prestressing strand is first laid in a stressing bed and tensioned. Next, the concrete is cast and cured and the prestressing strands are detensioned. When a structural member is prestressed using *post-tensioning*, empty ducts are typically laid in a stressing bed first. The concrete is then cast and cured, after which the prestressing strands are placed inside of the ducts. After the new concrete has reached a certain strength, the prestressing strands are tensioned and anchored in place.

The prestressing force in a prestressing strand is transferred from the strand to the concrete by bond in a pretensioned member. To transfer the force in the steel at the ultimate load of the member, a certain distance is needed to achieve bonding between the steel strand and the concrete. This distance, measured from the end of the member, is called the development length. (*2,3*) Because the prestressing force in a prestressing strand is transferred from the strand to the concrete through the end anchorages—rather than through bonding—in a post-tensioned member, there is no development length for a post-tensioned member.

The development length is made up of two components: transfer length and flexural bond length. (See figure 1.) The *transfer length* is the distance from the end of the member needed to fully transfer the effective prestressing force by bond from the prestressing strand to the concrete. If the prestressing force is divided by the area of the prestressing strand, the effective prestressing force corresponds to an effective prestressing stress in the strand. As can be seen in figure 1, the transfer length corresponds to the distance needed from the end of the member to develop the effective prestressing stress, f_{se}, in the strand. (*3,4,5*)

The *flexural bond length* is the length needed beyond the transfer length to achieve bonding between the steel strand and the concrete in order to attain the stress in the strand at the ultimate load of the member, f^*_{su}. (*3,4,5*)

In article 9.27 of the *AASHTO Specifications*, development length is given by equation 9-32 as: (*1*)

$$(f^*_{su} - \frac{2}{3}f_{se})\ D \qquad (1)$$

Where:
D = nominal diameter of the strand in inches

If this expression is rewritten, it can be broken into its constituent parts as follows: (*1,2*)

$$L_d = f_{se}\frac{D}{3} + (f^*_{su} - f_{se})D \qquad (2)$$

Where:
L_d = development length
$(1/3f_{se})\ D$ = transfer length
$(f^*_{su} - f_{se})\ D$ = flexural bond length

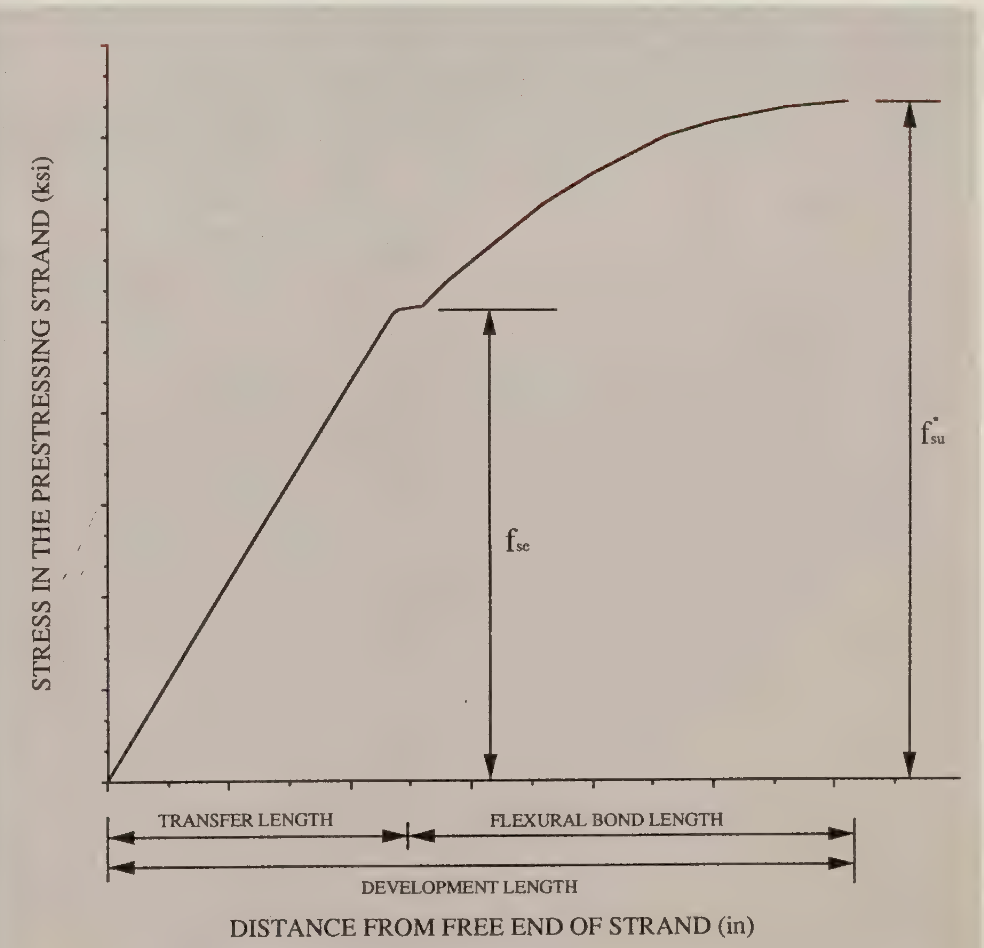


Figure 1.—Development length of prestressing strand.

The *AASHTO Specifications* further state in article 9.20.2.4 that the transfer length component of the development length may be assumed to be equal to 50 times the strand's nominal diameter. (*1*) No expression is given, however, for just the flexural bond length component.

If the full development length is not provided, the ultimate flexural strength of a structural member is greatly reduced. The shear strength of a structural member could also be reduced by an increase in the transfer length component of the development length.

Why Was the Memorandum Issued?

The FHWA issued its memorandum to address two technical incompatibilities. The first incompatibility was between the strand used in the experiments leading to AASHTO equation 9–32 and the strand now in use. The second incompatibility was that the development length values resulting from AASHTO equation 9–32 did not agree with recent research results.

The AASHTO equation, which is used to calculate the development length of prestressing strand, is based on research conducted in the 1950's and 1960's. This research used stress-relieved prestressing strand with an ultimate strength of 250 ksi (1720 MPa); it included very little work on 0.6-in (15.2-mm) diameter strand. For these studies, the stress in the strand immediately after prestress transfer could not exceed 70 percent of the ultimate tensile stress in the steel strand. (*6,7,8*)

In contrast, the current practice is to use low-relaxation strand with an ultimate strength of 270 ksi (1860 MPa). Also the *AASHTO Specifications*, modified in 1986, did not allow a stress in the strand immediately after prestress transfer to exceed 75 percent of the ultimate tensile stress of the steel strand. Therefore, the strand now in use varies greatly from that used to obtain the AASHTO equation 9-32.

Recent research indicated that the development length of uncoated low-relaxation strand with an ultimate strength of 270 ksi (1860 MPa) was greater than that predicted by the AASHTO equation. (*3*) This same study also showed that the development length of grit-impregnated epoxy-coated strand (low-relaxation strand with an ultimate strength of 270 ksi [1860 MPa]) was smaller than the development length calculated using the AASHTO equation.

Research

As a result of the FHWA memorandum many research studies have been undertaken to provide needed information on prestressing strand transfer and development lengths. For some studies, the transfer and development length experimentation is the sole objective of the study; for others, it is an important constituent of a broader study objective. Regardless of the size, all of the studies are providing significant contributions to understanding and mathematically defining prestressing strand transfer and development lengths. These studies are summarized below.

Florida Department of Transportation

A study entitled "Flexural, Shear, and Fatigue Behavior of Precast Prestressed Concrete Beams" is in progress at the Florida Department of Transportation's Structural Research Center. In this research, 33 AASHTO Type II prestressed concrete I-girders with three different uncoated strand diameters—including 0.6-in (15.2-mm) diameter strand—are being investigated to determine transfer and development lengths as well as to evaluate debonding, shear, and fatigue behaviors.

Preliminary results indicate average transfer length values of 80 to 90 times the strand diameter, measured from the end of the girder for fully bonded strands and from the point of debonding for debonded strands.[2]

Work began on this study in June 1989; the final report is scheduled to be completed in June 1991. All study test specimens have been donated by representatives of the Florida Prestressed Concrete Association and by the Florida Wire and Cable Company. Additionally, the Precast/Prestressed Concrete Institute (PCI) is sponsoring a visiting professor from Queens University in Canada to help with the study.

University of Texas at Austin

Work began in September 1988 on a Highway Planning and Research (HP&R) study entitled "Influence of Debonding of Strands on Behavior of Composite Prestressed Concrete Bridge Girders." Transfer length, development length, and fatigue are being investigated in rectangular specimens, I-girders, and box beams containing debonded strand. Two strand diameters—0.5 and 0.6 in (12.7 and 15.2 mm)—and different center-to-center spacings for the uncoated strand are being studied. Preliminary results indicated average transfer length values of 75 to 85 times the strand diameter for the rectangular specimens containing 0.6-in (15.2-mm) diameter strand. The study, jointly sponsored by the FHWA and the State of Texas, is scheduled for completion in August 1991.

[2]Note that the *AASHTO Specifications* state that the transfer length is assumed to be 50 times the strand diameter.

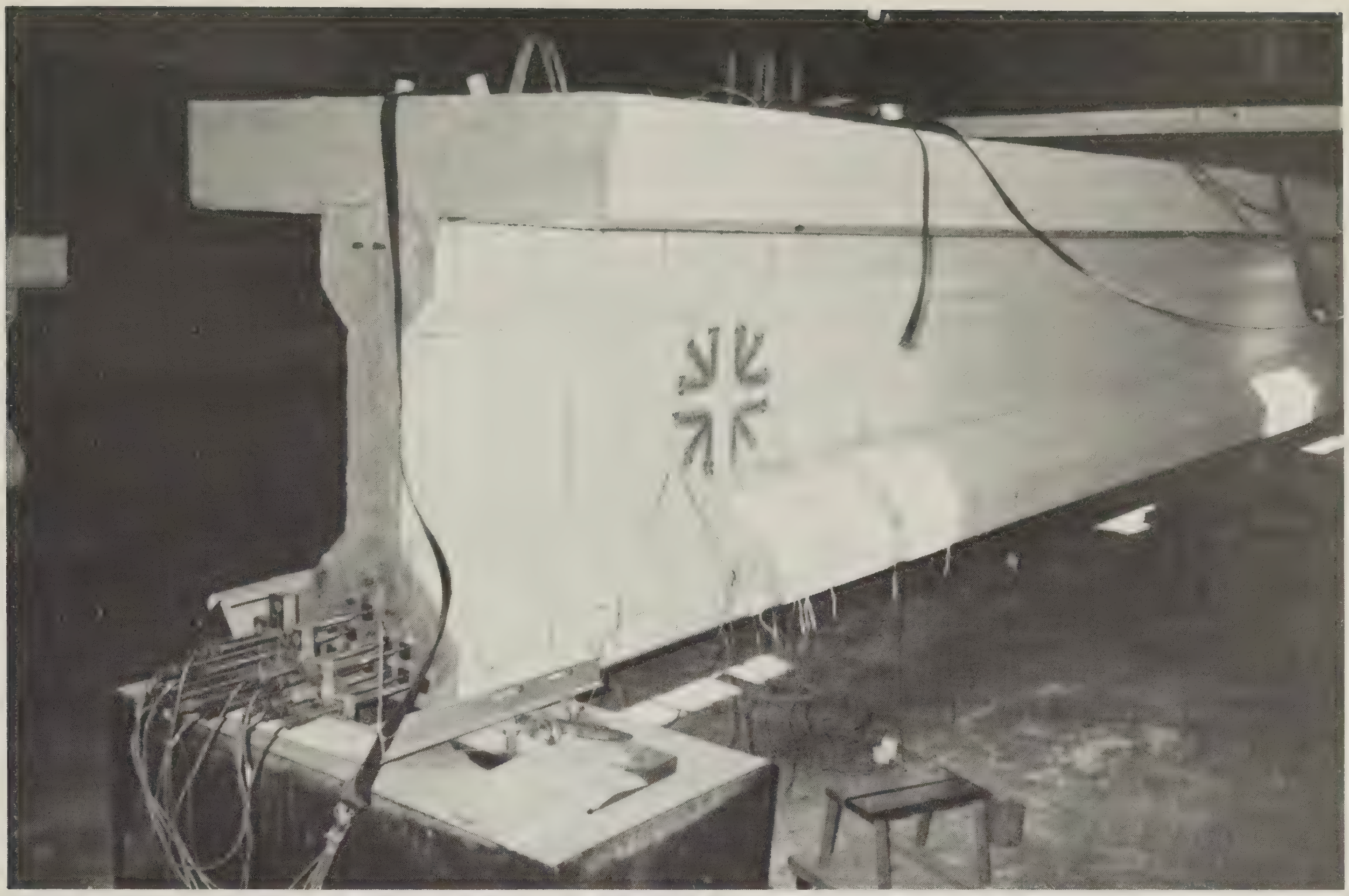


Figure 2.—AASHTO Type II prestressed concrete I-girder prior to the start of development length experimentation.

Figure 3.—Florida Department of Transportation researchers measure strain for AASHTO Type II prestressed concrete development length experimentation.

University of Tennessee

Investigations here focus on the development length of 0.5-in (12.7-mm), 0.5-in special (13.3-mm), and 0.6-in (15.2-mm) diameter uncoated strand in full-size AASHTO Type I prestressed concrete I-girders. Casting of the girders began in April 1989 and the draft final report was issued in April 1990. Preliminary results yielded transfer lengths of approximately 80 times the strand diameter and overall development lengths of approximately 1.1 times that predicted by the AASHTO equation 9-32. These results were all from girders containing uncoated strand at the AASHTO-specified spacings. Other tests were run on girders containing uncoated strand at smaller than the AASHTO-specified spacings, but these results were not available at the time of publication.

The PCI contributed to the funding of this study along with the Florida Wire and Cable Company which funded a separate study to investigate a small number of girders containing epoxy-coated strand.

Purdue University

A study entitled "Strand Debonding in Pretensioned Beams"—an HP&R study for the State of Indiana and the FHWA—concentrates on shear strength and compressive stresses in support regions of two-span continuous I-girders and box beams with debonded strand. Transfer and development lengths are also being measured. This 2½-year study began in June 1989, and the first girders were fabricated in December 1989.

Louisiana State University and the University of New Orleans

These two universities are combining forces for two studies. The first, "Development of Standard Test for Bond Characteristics of Epoxy-Coated and Uncoated Prestressing Strand," centers on developing a standard method of measuring the bond between prestressing strand and concrete. The study, which began in the summer of 1989 and is scheduled to be completed by the summer of 1990, is funded by the two schools, PCI, and local prestressed concrete producers.

The second study, "Spacing and Cover of Epoxy-Coated Strand," is a proposed HP&R study for the State of Louisiana and the FHWA. Transfer lengths will be measured for 40 rectangular specimens containing either epoxy-coated or uncoated 0.5-in (12.7-mm) diameter prestressing strand, with varying strand spacings and concrete cover. This 2-year study began in April 1990.

McGill University

This Montreal, Canada university began a 3-year study in April 1989 for the Canadian government on the detailing and design of pretensioned high-strength concrete double-T beams for buildings. The first phase of this program involves the determination of transfer and development lengths for various diameters of uncoated prestressing strand—including 0.6-in (15.2-mm) diameter strand—in rectangular specimens. A draft report describing the results of the first phase of the program was scheduled to be completed in the spring of 1990.

University of Calgary

A new theory on bond interaction between prestressing strand and concrete is being investigated with funding by the Natural Sciences and Engineering Research Council of Canada. The study, which began in 1987, compares theoretical and transfer and development length results from experimentation on T-shaped specimens containing various strand diameters (including 0.6-in [15.2-mm] diameter strand). Preliminary results indicated that transfer length was inversely proportional to concrete tensile strength, concrete cover, and strand spacing. Development length tests were scheduled for the spring of 1990.

University of Wisconsin at Milwaukee

Investigations are under way here to study the effect of elevated temperatures on the bond strength of epoxy-coated prestressing strand. The project is funded by a fellowship grant from PCI in response to industry concerns about the possibility of bond failure of epoxy-coated strand under fire exposure conditions. Experimentation began in the spring of 1990.

Federal Highway Administration

A research study is in progress at the FHWA's Turner-Fairbank Highway Research Center in McLean, Virginia, entitled "Investigation of Development Length of Uncoated and Epoxy-Coated Prestressing Strand." Fifty prestressed concrete rectangular specimens and 42 AASHTO Type II prestressed concrete I-girders are being investigated to determine transfer and development lengths. Study variables include diameter of the strand (.375-, 0.5-, .5625-, and 0.6-in [9.5-, 12.7-, 14.3-, and 15.2-mm]) strand spacing, concrete strength (f'_c = 5,000, 7,500, and 10,000 psi [35, 52, and 70 CpM]), and the amount of confinement reinforcement. Experimentation began in the spring of 1990. An interim report should be completed in December 1991; the final report is due in early 1993.

Conclusions

Through the research efforts described above, it is hoped that the accuracy of the AASHTO equation 9-32 in determining the development length of currently used uncoated and epoxy-coated prestressing strand can be verified. If this equation is in error, these research efforts should combine to produce a clear and accurate mathematical definition of development length.

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Summary of Lifeline Earthquake Engineering[1]

by James D. Cooper

Background

Lifelines are those utilities, facilities, and structures that must function following an earthquake to facilitate search and rescue, provide emergency services, allow for the movement of goods and materials, and form the needed network required for post-event reconstruction. Electric power and communications, gas and liquid fuel, transportation and water sewer systems form today's modern lifeline network. Each lifeline is comprised of several components, some of which are critically vulnerable to earthquake-induced damage and whose loss of function renders the lifeline useless. For example, a break in a major gas, oil, or water transmission pipeline can cause system failure, while a break in a gas or water service line may eliminate service only to a local area. In either case, failure may induce secondary problems such as fire caused by escaping gas from a broken pipe or the inability to control fires because of a broken water pipe.

Attention was focused on the lifeline problem following the 1971 San Fernando earthquake. Since then, numerous post-earthquake investigations have been made and damage assessment reports prepared documenting the performance of lifelines. The Loma Prieta, California earthquake of October 17, 1989, refocused attention on the importance of functioning lifelines—particularly transportation lifelines—following an event. Failure of a few critical bridges disrupted traffic flow and added untotaled costs to the public. (See figures 1 and 2.)

[1]This article is based on "Summary of Lifeline Farthquake Engineering" from *Proceedings of the 21st Joint Meeting of the U.S.-Japan Cooperative Program in Natural Resources: Panel on Wind and Seismic Effects*, Tsukuba, Japan, May 16-19, 1989.

Basic research is providing engineers with an understanding of how and why lifeline components perform the way they do. Yet little has been published to provide detailed guidance for the design and construction of lifelines to resist strong ground shaking.

The design and construction of integrated lifeline systems involves the application of multidisciplinary topics and experience gained from previous earthquakes where weaknesses in design, construction, system architecture, and management have been highlighted. Lifeline performance and reliability can affect a broad geopolitical area. Because of this, community leaders, public officials, and the private sector must all be involved to mitigate damaging earthquake effects.

Pre-earthquake considerations include the identification of expected variations in earthquake intensity, engineering factors, and policies influencing risk and reliability. The planning process for each lifeline system will significantly influence the expected outcome of the effects of a seismic event. The process includes an understanding of the geological factors that produce the seismic intensity levels that cause structural damage and ground failure. This knowledge supports systematic evaluation of hazards and risks required for planning reliability needed to mitigate earthquake damage.

The easiest, most cost-effective way to mitigate seismically induced damage is to upgrade seismic design and construction procedures for new construction. This approach will take decades to protect the existing inventory, but will ultimately reduce the potential for catastrophic impact in the event of a major earthquake. Only limited seismic design procedures and details have been developed for lifelines over the past 20 years. However, some newer structures incorporating these procedures have been exposed to relatively strong earthquakes and have performed quite satisfactorily. For example, bridges designed and constructed since the mid-1970's functioned well in the 1989 Loma Prieta earthquake. Design and construction costs associated with enhanced seismic resistance for lifelines are still being evaluated. A range of from 1-to 20-percent increased project costs for upgraded seismic resistance has been suggested; the percentage is highly dependent on the level of seismic detailing incorporated into the design. Nonetheless, distributed over time, these costs become an inexpensive investment significantly reducing the sudden economic loss potential from a major earthquake.

A second method to mitigate seismically induced damage is to retrofit existing lifelines. Retrofit is typically *much more costly* than planning in advance for the seismic design of new construction. However, retrofit can be cost effective even in regions outside what is thought to be the traditional seismically active zones. For example, if potentially vulnerable transportation routes are identified, a large number of bridges along these critical routes could be "restrained" at low cost (figure 3). Since this retrofit technique typically costs between $10,000 and $20,000 per bridge, 150 bridges could be retrofitted for the approximate cost of 1 new structure. Retrofit details for bridges are becoming more common in traditional seismically active areas, although retrofit is not yet a generally accepted policy in these areas. For other types of lifelines, retrofit *may not* be cost effective except in unusual circumstances.

This article summarizes recent and ongoing activity that will have important impact on the seismic design and construction of lifeline facilities.

Lifeline Design Methods

The American Society of Civil Engineers (ASCE) established the Technical Council on Lifeline Earthquake Engineering (TCLEE) in 1974 to enable civil engineers to

Figure 1.—Loma Prieta: vital transportation link closed.

Figure 2.—Loma Prieta: transportation routes closed.

play a key role in elevating the state of the art of lifeline earthquake engineering. At that time, no major organization or agency was committed to this problem area. One of the TCLEE's major goals was to develop and assemble guidelines for each lifeline system. Accordingly, the ASCE's "Advisory Notes on Lifeline Earthquake Engineering" presents the most comprehensive overview of the current state of the art of seismic design practice for each lifeline area and provides some insight into how they have performed in past earthquakes. (*1*)[2] Further, it provides information and approaches for developing improved designs for new lifeline facilities. Since the guide's publication, however, significant research has been sponsored by the National Science Foundation (NSF), the Federal Emergency Management Agency (FEMA), the Department of Transportation, and others; the results of this research warrant an update of the "Advisory Notes."

Since 1984, some utilities, privately owned gas and oil companies, and individual agencies have developed their own criteria for seismic design. From these, a general approach for the design of lifeline systems has evolved as follows. These general steps have been applied to the design of electric power and communications facilities, gas and oil transmission systems, and transportation structures, including bridges:

- Perform or conduct site-specific studies which reflect the hazard exposure to the lifeline. Typically, geological and seismological investigations are conducted which identify local and regional geology and historical earthquake activity. In some cases, field exploration studies are conducted.
- Develop criteria based on risk considerations for potential ground motions, including accelerations, velocities and displacements at the site, duration of shaking, and distance of the facility from causative fault.
- Establish structural design parameters including the development of design spectra.
- Factor special structural considerations including estimates of structural damping and ductility into the criteria.
- Develop allowable deformation and force criteria.
- Identify special detailing with emphasis on foundations, anchorages, and connections.

Future Lifeline Design Guides

Congress enacted the Earthquake Hazard Reduction Act of 1977 to reduce risks of life and property from future earthquakes. This act forms the basis of the National Earthquake Hazards Reduction Program (NEHRP) whose specific objectives include:

- Development of seismic design and construction guidelines.
- Development of guides for facilities that are Federally owned, constructed, or financed to ensure serviceability following an earthquake.
- Coordination of the establishment of guides to consider seismic risk in the development of Federal lands.

In 1978, the Interagency Committee on Seismic Safety in Construction (ICSSC) was established as part of the NEHRP to assist Federal departments and agencies involved in construction to develop earthquake hazard reduction measures for incorporation in their ongoing programs. The measures will be based on existing standards when feasible.

Through its subcommittees, the ICSSC cooperates with State and local governments and private organizations in developing nationally applicable earthquake hazard reduction measures. In this context, it is useful to examine the role of one of these subcommittees—the Lifelines Subcommittee.

The mission of the Lifelines Subcommittee is to:

- Identify existing guidelines or standards for seismic design; construction; and retrofit of energy, transportation, water, and telecommunication systems.
- Recommend Federal adoption of such standards when found adequate.
- Encourage development of new standards where there are significant omissions.

[2] Italic numbers in parentheses identify references on page 211.

The subcommittee will also study techniques for evaluating the seismic vulnerability of existing lifelines, for improving the ease of repair, and for strengthening structures to resist seismic effects. (*4*)

The Lifelines Subcommittee is considering strategies that will permit identification of those lifeline facilities that are important to either emergency, immediate recovery, or long-term economic recovery periods, and then provide guidance for appropriate levels of seismic protection for each lifeline type.

Currently, the subcommittee is reviewing technical literature pertaining to seismic resistant design methodology of lifelines. Upon completion of this review, it will develop a formal recommendation to the ICSSC for specific guidelines for Federal agencies.

Lifeline Research

Several Federal agencies and organizations are actively pursuing lifeline earthquake engineering research. Their work will add much to our basic understanding of lifeline systems. The following summarizes key ongoing efforts. Although additional research is being conducted by private organizations and institutes, information on these studies are generally not available to the public.

Department of Transportation research

Early in January 1990, the Secretary of the Department of Transportation (DOT) approved a program to develop additional measures for reducing long-term vulnerability to earthquake hazards in DOT construction projects. An initial three-phased program will address earthquake awareness and current design and retrofit procedures for transportation facilities.

Seismic Design Awareness. The DOT will develop a national education program for its constituency and grantees. This program will identify both earthquake hazards and specific measures that can reduce vulnerability. A preliminary effort will summarize applicable research conducted in the last 15 years and document the lessons learned from recent earthquakes. These findings will be publicized and sponsored by the DOT in conference sessions on earthquake design.

Uniform Standards for DOT Programs. Also under its earthquake hazards program, the DOT will establish uniform standards for the seismic design of transportation structures. These standards will provide the basis for both safety regulation and guidance for DOT grantees. The standards will be based on the material contained in the 1988 NEHRP provisions and other available information. Particular attention will be focused on the needs of eastern cities.

Retrofit Policy. The DOT plans to synthesize existing information on retrofitting concepts for buildings and transportation structures in order to provide policy guidance to its modal constituents on where to encourage their use.

National Science Foundation research (*5*)

Geotechnical Research. The NSF is funding several research studies relating to siting and geotechnical systems research. These studies focus on the fundamental engineering issues related to ground shaking and the effects it has on geological-structural interactions. Specific engineering investigations are being conducted to better understand the process of subsurface ground damage to lifeline systems including utility lines and energy transmission systems. These investigations include one by the University of Colorado on the effect of geological layered strata in modifying the strong ground motion experienced by tunnels and buried pipelines and includes amplification of ground motion at the surface. Figure 4 depicts replacement parts for underground pipelines damaged in the Loma Prieta earthquake.

Lifeline System Performance Data. The October 1987 Whittier Narrows, California earthquake and the October 1989 Loma Prieta earthquake offer an opportunity to assess the impact that research conducted since the 1971 San Fernando earthquake has had on lifeline design and performance. Under NSF sponsorship, Dames and Moore, Inc., is collecting, reviewing, and documenting lifeline system performance data from the Whittier Narrows earthquake and developing preliminary response and recovery models for different lifeline systems using data from both the Whittier Narrows earthquake and the San Fernando earthquake. In addition, Dames and Moore, Inc., is assessing the impact that research in the lifelines area conducted after the San Fernando earthquake has had on improving seismic design and emergency response and preparedness procedures for lifelines.

Pipelines Research. The 1987 Whittier Narrows earthquake caused extensive damage to water distribution pipelines. Although theoretical modeling of earthquake-induced damage has been actively pursued, these models are based upon little-understood damage mechanisms. Moreover, there are very little data related to the direct observation of earthquake-induced damage. Such data are needed to validate these models so that they can be used with confidence in deriving seismic design criteria.

An Old Dominion University study sponsored by the NSF involves a thorough field investigation and analysis of the damage to water lifeline systems around Whittier. Predictions of the existing models are being compared with observed behavior; and the models updated accordingly.

Figure 4.—Loma Prieta: underground lifeline utilities heavily damaged in Marina district.

The NSF is also sponsoring a field experiment by Weidlinger Associates in which underground pipelines are placed within the Parkfield segment of the San Andreas Fault Zone. The experiment will help examine the validity of current assumptions regarding the behavior of underground continuously welded steel pipe and jointed ductile iron pipe subjected to near-surface fault offset. Both active and passive instrumentation are included to permit evaluation of important assumptions made in current prediction and design methods.

Finally, the NSF, the State of New York, and various public and private corporations are cosponsoring research by the National Center for Earthquake Engineering Research (NCEER) at Buffalo, New York. (*6,7*) The NCEER research is designed to improve basic knowledge about earthquakes, engineering practices, and the implementation of seismic hazard mitigation procedures to minimize the loss of lives and property. A major focus of the effort is in the area of lifeline system failure, specifically of two lifeline systems—water delivery and crude oil transmission.

The research on water delivery systems addresses issues relating to ground motion studies; system performance, vulnerability and serviceability; and risk assessment and societal impact from loss of function. In addition to ground motion studies—which include space-time correlation and geological, topographical, and ground motion data analyses—numerous detailed studies are being conducted to better understand the hazards associated with liquefaction and large ground deformation, soil-structure interaction, wave propagation, and fault crossings. Detailed studies on system vulnerability, response, and serviceability include evaluating damage and repair of piping systems, establishing system reliability, and enhancing analysis capability.

Research on crude oil transmission systems involves the development of a seismic risk assessment for a crude oil pipeline system which traverses large areas. Factors to be examined will include effects of peak ground accelerations and permanent ground displacements on transmission pipeline response.

Completion of these studies will result in a greater fundamental understanding of how water delivery and crude oil transmission systems respond and interact when subjected to earthquake activity. These results will provide important information for developing detailed seismic design guides for piping systems.

Federal Emergency Management Agency research

Action Plan. In recent years, the FEMA has become a major sponsor of lifeline earthquake engineering studies. For example, the agency sponsored development of a seismic hazard abatement action plan for lifelines. The plan, developed by the Building Seismic Safety Council provides a basis for planning a long-range lifelines research program. The action plan was developed through a workshop consensus process, with expert participants drawn from the diverse areas of lifeline systems design, construction, operation, and maintenance. In all, the action plan recommended 67 priority projects related to lifeline systems. Details of these are contained in "Abatement of Seismic Hazards to Lifelines: An Action Plan" and *Abatement of Seismic Hazards to Lifelines: Proceedings of a Workshop on Development of an Action Plan.* (*2,3*)

Prioritizing the Action Plan. The FEMA also sponsored a followup study to develop expert recommendations and consensus for prioritizing and implementing the action plan. To this end, an ad hoc committee on lifelines was established. The committee observed that earthquakes pose a profound threat to the reliability and continued survivability of all lifeline systems. The committee recommended that a comprehensive, nationally coordinated program be implemented to mitigate the risk to lifelines from seismic and other natural hazards. Further, the committee identified three high-priority project areas in which to concentrate program activities:

- Improve the awareness and education on lifeline hazards mitigation to lifeline service providers, users, and regulators.
- Develop information on vulnerability of lifeline systems to natural hazards and identify procedures for minimizing lifelines vulnerability.
- Develop and recommend regulatory actions for adopting hazard mitigation standards and criteria for lifeline systems.

Model of Earthquake Damage to Lifelines. The FEMA has initiated a study with EQE, Inc., to assess the potential impact of damaging earthquakes on lifeline systems in general and to determine the vulnerability of a water supply system as a case model. This initial effort is expected to contribute a considerable amount of needed information to promote an awareness of the importance of mitigating hazards to lifeline systems.

Conclusion

Although significant attention has been focused on the effects earthquakes have on lifelines, relatively little guidance is available for their seismic-resistant design. The most definitive seismic design criteria available in the United States for a specific set of structures is the American Association of State Highway and Transportation Officials *Guide Specifications for Earthquake Resistant Design of Highway Bridges.* (*7*) More general seismic design guidance for lifelines is documented in the ASCE publication "Advisory Notes on Lifeline Earthquake Engineering." (*1*)

Important research has and is being conducted that will provide information for updating seismic design guidance for lifelines. Based on completed research, minimal fiscal resources could be invested now to update design guidance for selected lifeline structures. For example, much has been learned about the dynamic effects of fluids in tanks, piping behavior, welding and detailing, and anchorage systems. From this information, existing design guidance for new tanks and piping systems could be updated.

Of course, many unanswered questions remain. These are identified in detail in "Abatement of Seismic Hazards to Lifelines: An Action Plan." (*2*) Moreover, many new lifeline facilities will be constructed before the research studies identified in the action plan can be completed. As engineers, we must apply today's knowledge acquired through lessons learned in past earthquakes to the design of tomorrow's lifeline structures.

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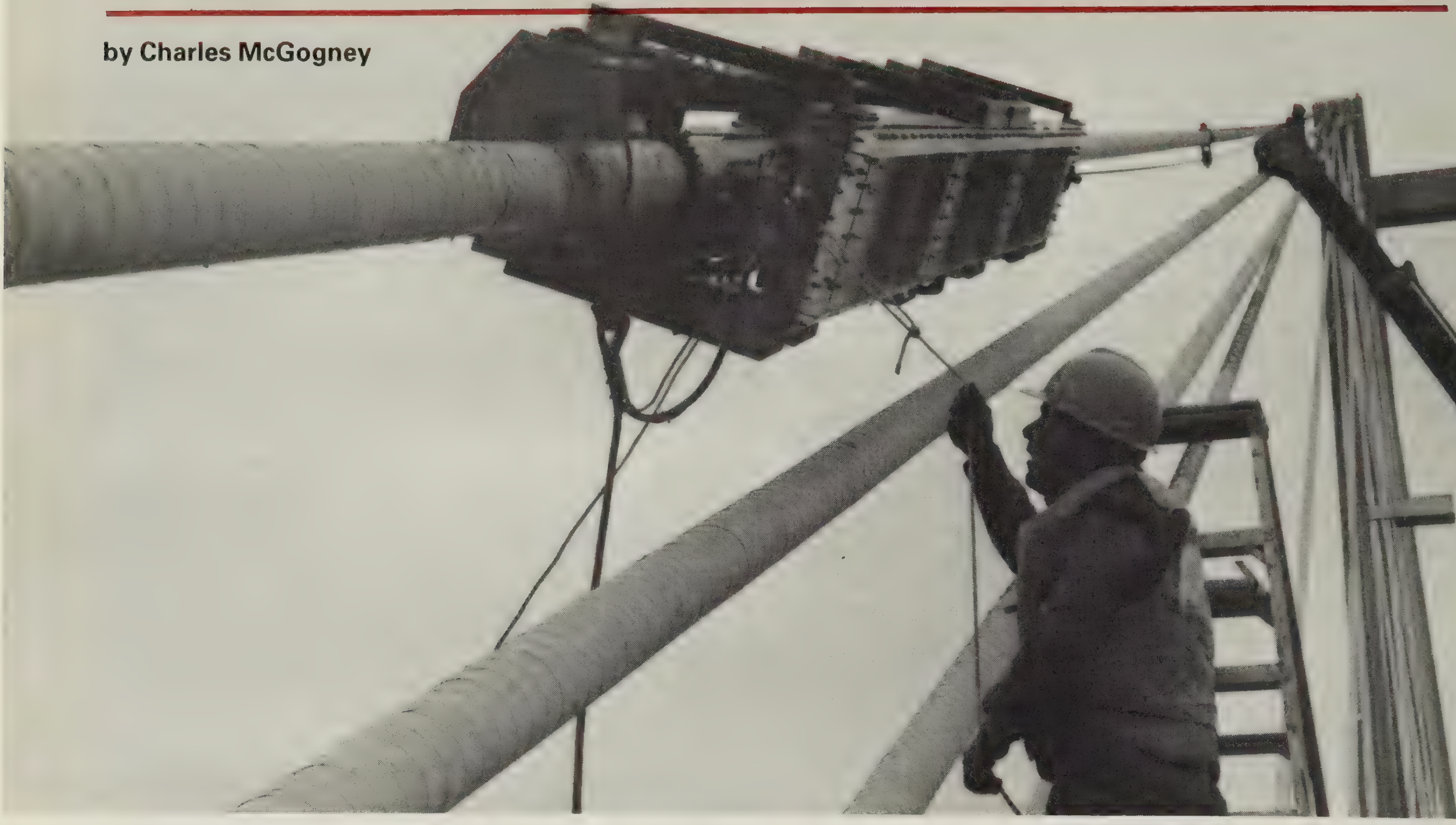
James Cooper serves as the deputy chief of the Structures Division, Office of Engineering and Highway Operations Research and Development, Federal Highway Administration. He is responsible for the administrative leadership and technical guidance of the Division's research programs on bridges, tunnels, hydraulics, and related transportation structures. Mr. Cooper also serves as chairman of the Lifelines Subcommittee of the Federal Government's Interagency Committee on Seismic Safety in Construction.

Laboratory and Field Trials of the Prototype Magnetic Perturbation Cable System

by Charles McGogney



Introduction

Bridges, particularly suspension bridges, pose numerous difficulties for bridge inspectors. Suspension bridge main cables and suspenders constitute one such problem. Although great care has been taken to protect the steel wires and rods that form the bundle, steel corrosion occurs and because of the protective tubing, grouting, and wrapping, it is difficult to pinpoint the source and location of affected areas without performing destructive tests. This is particularly difficult if the bridge is in active service.

Consequently, the Federal Highway Administration (FHWA) has long sponsored research efforts aimed at developing nondestructive bridge inspection and monitoring methodologies. Such a method would be able to detect loss of section or breaks in the steel wires or rods without removing the bridge's protective materials.

To solicit input to this problem, the FHWA issued a request for proposals. In February 1987, an award was made to Texas Research Institute of Austin, Texas, to design, develop, and fabricate a prototype nondestructive inspection and monitoring system for structural cables and strands of suspension bridges. After an indepth study of the available nondestructive methodologies, the researchers determined that magnetic field perturbation was the method most likely to prove successful in satisfying the FHWA objectives. (*1*)[1]

Background

The magnetic field perturbation method is well understood. For its implementation, however, the applied magnetic field must be controlled and signature characteristics must be understood. (See figures 1a, 1b, and 1c for information on the theory and its applications.)

[1]Italic numbers in parentheses identify references on page 216.

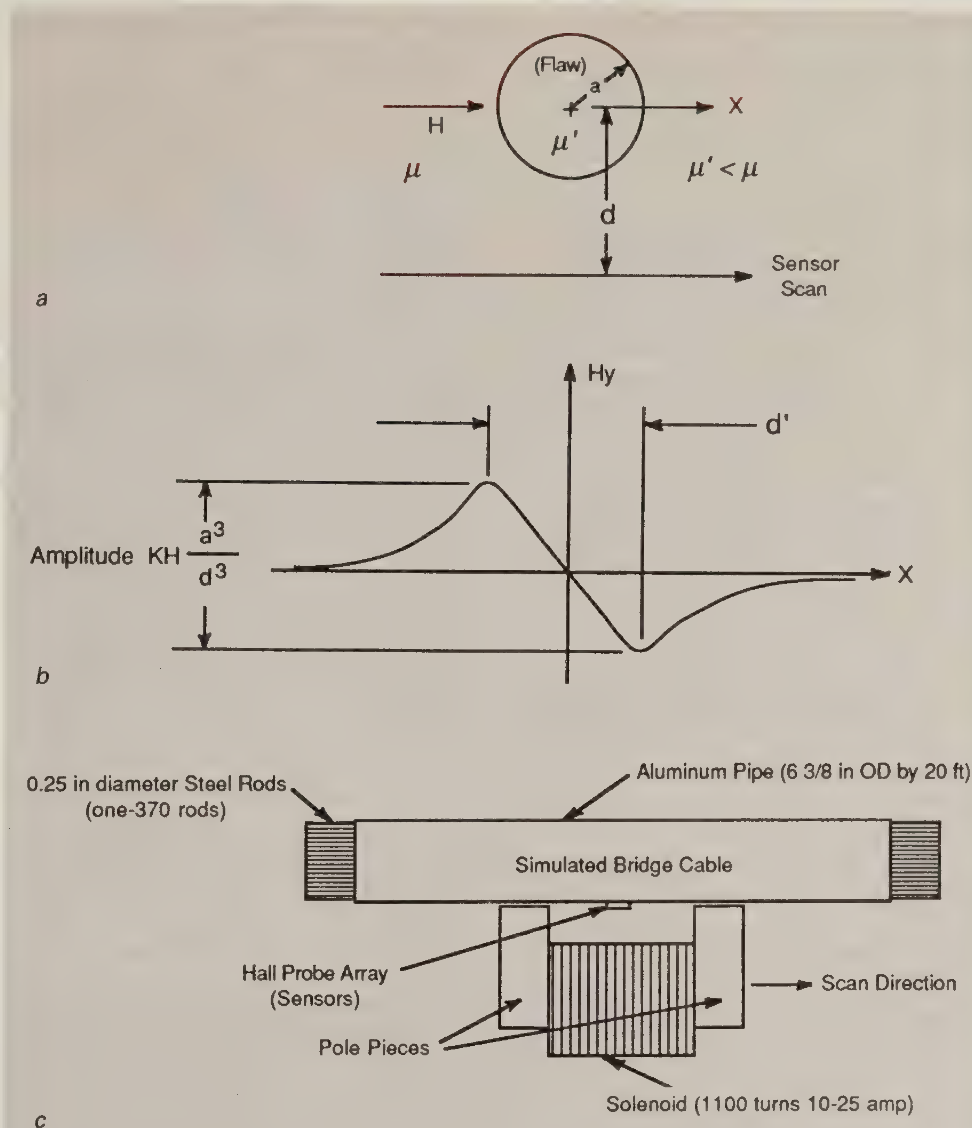
In figures 1a and 1b, a spherical volume (flaw or defect) with a magnetic permeability μ' is embedded in a ferromagnetic material (steel) with a permeability μ and a magnetic field H applied along the X direction. Figure 1c shows the arrangement for magnetizing a cable when the electromagnet and sensor are scanned as a unit over the region containing a flaw, resulting in a magnetic perturbation produced by the flaw. The magnetic field component in the Y direction (Hy) in figure 1b is sensed, and a continuous plot produces the record or signature. Solution of the magnetic field equations produces very useful relationships. For example, for the maximum peak-to-peak amplitude:

$$Hy = \frac{KHa^3}{d^3}$$

Where:

K = constant
H = applied magnetic field
a = flaw radius
d = depth from the scan path to the flaw center

Characteristic features that are useful in relating nondestructive evaluation data to flaw size, depth, location, etc., are listed in figure 1c.



SIGNATURE FEATURES:

1. Bipolar and symmetric
2. Polarity as shown for fracture gap or corrosion pit; polarity reversed for soft spot in steel of "ferrule" splice
3. Peak separation d' equals depth to flaw d (independent of size)
4. Amplitude indicates flaw volume (dependent on depth)
5. Maximum amplitude obtained when scan path is directly over flaw
6. Amplitude decreases with flaw depth d (decrease: $1/d^3$)
7. Signal amplitude increases continuously as the magnetic field (H) is increased

Figure 1.—Magnetic perturbation signature features and magnetizing method.

Laboratory Results

Preliminary investigations were conducted using a rudimentary system which consisted of a 200-lb (91-kg) electromagnet, a single Hall probe (sensor), and a powered scanning assembly. The scan assembly was supported by rollers riding on the specimen.

At first the specimen used was a 6.0-in (152-mm) diameter steel pipe 20 ft (6.1 m) long and filled with 370 rods of 0.25 in (6.35 mm) diameter. The 0.28-in (7.1-mm) thick wall was intended to simulate the protective 0.2-in (5.1-mm) diameter soft steel spiral wrap used on some cables. Fracture type flaws were simulated by gaps in the rods. This design was altered, however, because of the high attenuation and drastically altered signatures encountered when using the steel pipe; it was replaced by an aluminum pipe. Additionally, the current in the electromagnet was increased, and a differential Hall probe was incorporated in the system. These changes significantly improved the unit's sensitivity and detection capability.

After reviewing the laboratory results and the program status, the researchers decided to focus equipment design for application on stay-cables, such as those on the Luling Bridge in Luling, Louisiana. Consequently, an integrated electromechanical system was fabricated consisting of:

- A magnetic sensor assembly attached to a moving electromagnet.
- Electrical and hydraulic motors and actuators.
- An electronic control and data acquisition system directed by a remote host computer.
- A self-propelled frame housing for the entire scanning system (see figure 2).

The system is designed to be placed directly onto a bridge cable. It can move itself along the cable to acquire magnetic perturbation data relating to the presence of corrosion damage and broken strands.

Field Demonstration Results

The first field demonstration of the prototype magnetic perturbation cable (MPC) inspection system was held on the Luling Bridge in December 1988. In general, the field demonstration was successful. Approximately 70 scan sequences, or 420 data tracks, each 100 in (2.54 m) long were recorded. Figure 3 depicts all six analog records from computer-reconstructed digital data. Figure 4 shows the same data as figure 3 after digital subtraction of channel 6 data from channels 1 through 5.

The second field demonstration of the prototype MPC inspection system took place October 31, 1989 on the Intercity Bridge over the Columbia River, Kennewick-Pasco, Washington. After check-out, the MPC was taken to the bridge site. Next, it was lifted into place on exterior cable #3–N–18 which is 6 in (152 mm) in diameter and approximately 440 ft (134 m) in length.

Although the system responded to commands from the command module, there were problems with the linear encoder and with one of the motors used to raise and lower the magnet. Once these problems were corrected, the cable inspection was completed.

Figure 5 represents a typical analog recording of data over approximately 40 ft (12.0 m) of cable. The three reference signals from the 0.25-in (6.35-mm) long wires permanently embedded in a stainless steel, half cylinder covering the scanning region on channel 3 are readily recognizable; these are repeated at specific intervals. Figure 6 is a computer reconstruction of figure 5 data after digital subtraction. The negative polarity signal, which occurs at regular intervals of approximately 30 to 40 in (75 to 100 cm), is prominent on all channels and is attributed to the spiral wire wrap. There is a strong signal occurring at approximately the 120-in (300-cm) location on track 2 which masks the signal from the helical wire wrap. This strong, significant signal is due to a metal band on the cable.

Figure 2.—Prototype MPC on stay-cable Pasco-Kennewick Intercity Bridge over the Columbia River, Washington.

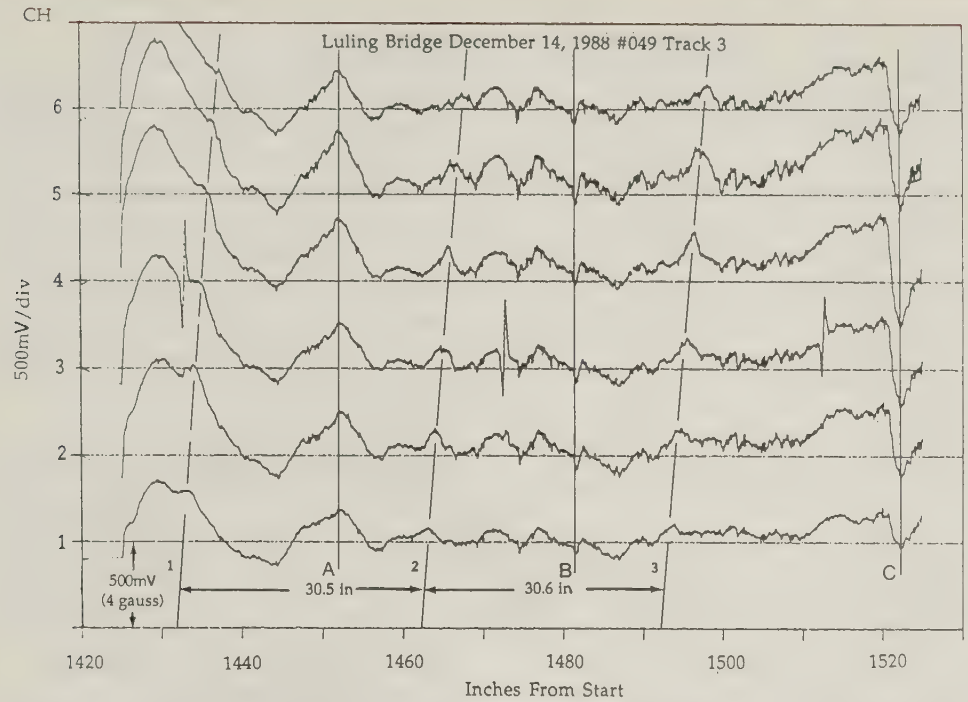


Figure 3.—Typical records from computer "reconstructed" digital data obtained during one scan of the Luling Bridge cable.

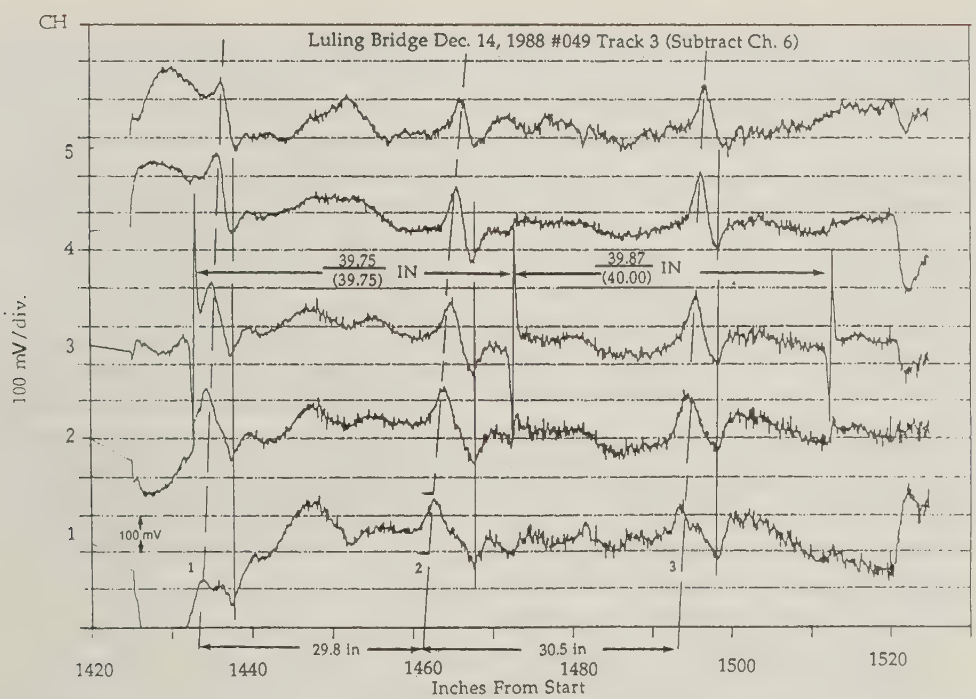


Figure 4.—Same digital data as figure 2 by typical analog records computer "reconstructed" after digital subtraction of channel 6 data from channels 1 through 5.

A third demonstration of the MPC inspection system took place in the Ferguson Structural Engineering Laboratory at the University of Texas in Austin. The objective of this laboratory test was to establish the system's capability in detecting actual fatigue damage in stay-cables. A stay-cable specimen with a 7.62-in (194-mm) diameter and approximately a 17 ft (5.2 m) length, which was comprised of 50 0.60-in (15.2-mm) diameter seven-wire strands had been tested in fatigue (figure 7). This specimen was modeled after a design proposed for the new bridge over the Houston Ship Channel at Baytown, Texas.

The MPC system detected fatigue fractures in single wires as small as 0.2 in (5.1 mm) in diameter in the outer seven-wire strands. These fractures were not suspected during the fatigue testing or after the fatigue testing was completed. They were first detected nondestructively at their precise locations by the MPC system and later confirmed by destructive examination. Other, larger, fatigue failures and tension overload fractures were also detected and located by the MPC system.

Conclusions

- The MPC inspection system has the sensitivity, resolution, repeatability, long-term stability, and adaptability to perform nondestructive inspection and monitoring tests of structural cables and strands of suspension bridges.
- Flux densities higher than that provided by the electromagnet would be required to detect very small flaws (0.01 percent cross-sectional area) such as corrosion pits.

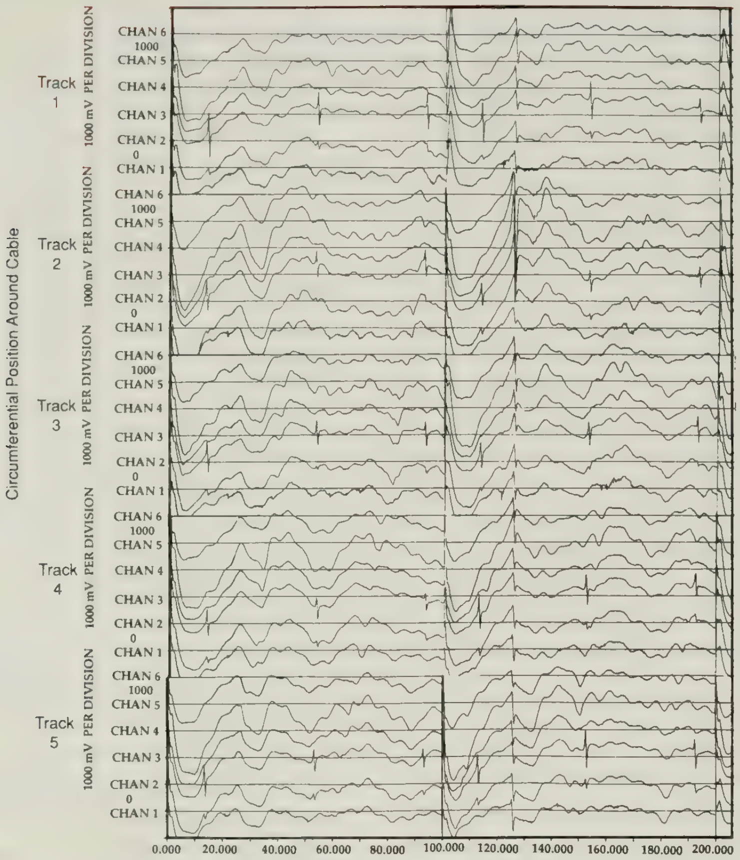


Figure 5.—Unprocessed magnetic signals observed from cable #3-N-18 on the Pasco-Kennewick Bridge.

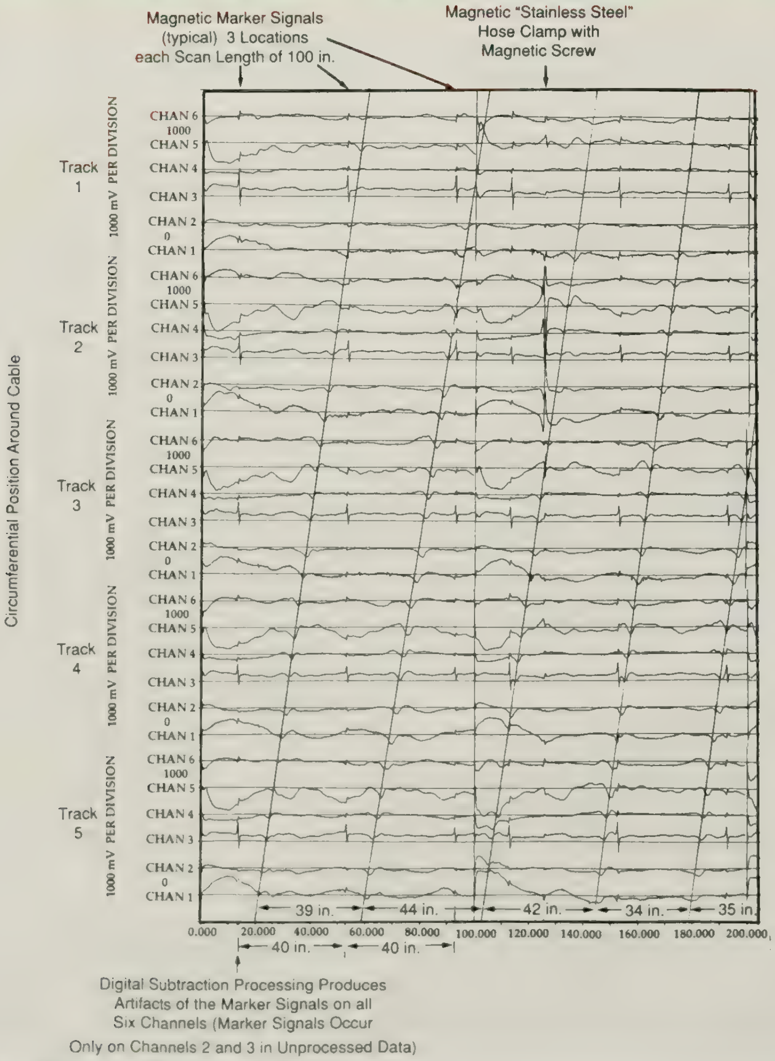


Figure 6.—Processed magnetic signals from cable #3-N-18 on the Pasco-Kennewick Bridge.

• The overall field operational capability of the prototype MPC system has been proven. Results obtained from scanning laboratory specimens indicate that systems of this type should be capable of providing precise, stable, quantitative, nondestructive evaluation. Such evaluation, which is essential for predicting the structural integrity of suspension bridges' cables has been—up to now—unobtainable.

Reference

(*1*) "Design, Develop, and Fabricate a Prototype Nondestructive Inspection and Monitoring System for Structural Cables and Strands of Suspension Bridges, Vol. I: Final Report." Publication No. FHWA-RD-89-158, Washington, DC, May 1989.

Charles (Chuck) McGogney is a research metallurgist in the Structures Division, Office of Engineering and Highway Operations Research and Development, Federal Highway Administration. Formerly, Mr. McGogney was a research engineer with Kaiser Aluminum and Chemical Corporation; he also served on the research staff at Washington State University. He holds a B.S. degree from the University of Maryland and is a registered professional engineer in California.

Recent Research Reports You Should Know About

The following are brief descriptions of selected reports recently published by the Federal Highway Administration, Office of Research, Development, and Technology (RD&T). The Office of Engineering and Highway Operations Research and Development (R&D) includes the Structures Division, Pavements Division, and Materials Division. The Office of Safety and Traffic Operations R&D includes the Traffic Systems Division, Safety Design Division, and Traffic Safety Research Division. All reports are available from the National Technical Information Service (NTIS). In some cases, limited copies of reports are available from the RD&T Report Center.

When ordering from the NTIS, include the PB number (or the publication number) and the publication title. Address requests to:

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Federal Highway Administration
RD&T Report Center, HRD-11
6300 Georgetown Pike
McLean, Virginia 22101–2296
Telephone: (703) 285–2144

Empirical Bayes Approach to the Estimation of Unsafety: The Multivariate Method, Publication No. FHWA–RD–90–006

by Traffic Safety Research Division

The Empirical Bayes (EB) approach to the estimation of "unsafety" uses historical accident records and traits such as traffic, geometry, age, or gender.

To estimate the unsafety of an entity (e.g., person, vehicle, intersection) using the EB approach, information is needed about the mean and the variance of the unsafety of comparable entities which form its reference population. Although the Method of Sample Moments has been used in the past, it falls short in three ways: a large reference population is required, the choice of a reference population tends to be arbitrary, and the entities in the chosen reference population usually cannot match the traits of the entity of unsafety in question.

This report describes the logical foundations of the Multivariate Method for estimating the mean and variance of unsafety in reference populations. The use of the Multivariate Method makes the EB approach to unsafety estimation applicable to a wider range of circumstances: it makes the decision about what entities to include in the reference population less arbitrary, and it yields better estimates of unsafety. The application of the EB and Multivariate Method in identifying deviant entities and estimating the effect of interventions on unsafety are discussed and illustrated by numerical examples.

Limited copies of this publication are available from the RD&T Report Center. Copies may also be purchased from the NTIS. (PB No. 90–208489/AS, Price code: A04.)

Research and Development Program for Highway Construction Engineering Management, Publication No. FHWA–RD–90–034

by Construction and Maintenance Division

This report recommends a priority program of 16 research and development needs that could result in a significant reduction in the $1.2 billion annual cost of highway construction engineering. The methodology used to generate the needs statements and the formula for the program are explained. Included in the report are the needs statements, a literature review, bibliography, current research, and a comparison with a similar 1979 study.

Limited copies of this publication are available from the RD&T Report Center. Copies may also be purchased from the NTIS. (PB No. 90-208471/AS, Price code: A06.)

Walk Alert National Pedestrian Safety Program 1989: Program Guide, Publication No. FHWA–RD–89–022

by Safety Design Division

A national program on pedestrian safety has been developed which includes engineering, educational, and law enforcement components. This program guide is a user manual for individuals responsible for setting up a comprehensive pedestrian safety program at the local level. Characteristics of pedestrian accidents are discussed. A procedure for setting up a program is presented. Media materials are included as well as a resource guide.

This publication may only be purchased from the NTIS. (PB No. 90-209628/AS, Price code: A08.)

Walk Alert Pedestrian Safety Program, Publication No. FHWA–RD–89–023

by Safety Design Division

The National Safety Council developed a national program on pedestrian safety for the Federal Highway Administration and the National Highway Traffic Safety Administration. This program was titled WALK ALERT. It is a comprehensive program using engineering, law enforcement, and educational countermeasures to reduce the numbers of pedestrian fatalities and injuries.

- Comprehensive set of materials to fill identified gaps in existing information.
- Community level program guide to apply proven techniques.
- State level network to provide guidance and funding of community-level pedestrian safety programs.

The final step was to pilot test the program and its materials and implement the program in the Commonwealth of Virginia. The study also initiated, organized, and implemented the ongoing WALK ALERT programs in 10 States and a major city.

Limited copies of this publication are available from the RD&T Report Center. Copies may also be purchased from the NTIS. (PB No. 90-208919/AS, Price code: A06.)

Calibration of Road Roughness Measuring Equipment Vol. I: Experimental Investigations, Publication No. FHWA–RD–89–077; Vol. II: Calibration Procedures, Publication No. FHWA–RD–89–078

by Pavements Division

Volume I documents the measurements and analyses that were carried out to develop calibration and testing procedures. It describes the extensive series of measurements made on the performance of an inertial road profiling system (IRPS), including evaluation of the noncontact height sensors, the accelerometers used to establish the inertial reference frame, the distance encoder, the associated instrumentation, and the software used to convert the raw data into road elevation profiles.

A field program was carried out which included rod-and-level surveys of several roads which were also profiled using an IRPS, which was also equipped with a commercial response-type road roughness measurement (RTRRM) system, with accelerometers to measure the vertical vibration of both the axle and the body of the vehicle, and with a linear potentiometer to measure the relative displacement between the axle and the body of the vehicle. Separate laboratory measurements were made to characterize the performance of the commercial RTRRM.

Data collected with the RTRRM and with the auxiliary accelerometers and the linear potentiometer were compared with single-number ratings of road roughness as computed from the profiles measured using the IRPS.

Volume II gives calibration and testing procedures developed from the tests described in volume I. These procedures will assist users in assessing the operating performance of the inertial profiling systems and of the response-type road roughness measuring systems.

These publications may only be purchased from the NTIS. Vol I: (PB No. 90–208273/AS, Price code: A05; Vol. II: PB No. 90–208281/AS, Price code: A03.)

Evaluation of the Optimized Policies for Adaptive Control Strategy, Publication No. FHWA–RD–89–135

by Traffic Systems Division

The optimized policies for adaptive control (OPAC) strategy is an online traffic signal control algorithm designed to optimize the performance of traffic signals at isolated intersections using delay as the performance criterion. The OPAC-RT is a traffic control system which implements the OPAC strategy in real time. The system uses traffic data collected from detectors located well upstream (400 to 600 ft [644 to 966 km]) of the stop bar on all approaches to an intersection. Optimum signal timing is determined using minimum and maximum green constraints and does not require a fixed-cycle length. This report describes three field tests of the on-line OPAC strategy. The results indicate that the OPAC performs better than well-timed actuated signals, particularly at greater demand levels.

Limited copies of this publication are available from the RD&T Report Center. Copies may also be purchased from the NTIS. (PB No. 90-208299/AS, Price code: A07.)

Examination of Truck Accidents on Urban Freeways, Publication No. FHWA–RD–89–201

by Safety Design Division

The objective of this study was to determine the nature and extent of urban freeway accidents involving trucks (over 10,000 lb [4563 kg] gross vehicle weight) and their consequences as a function of vehicle type, and traffic and roadway characteristics. The study was limited to urban freeways and expressways with a large total volume (minimum 100,000 average daily traffic) and a significant percentage of large truck traffic (minimum 5 percent).

The primary tasks involved a review of the literature and the analysis of accident and operational data from selected urban freeway sites. A total of 2,221 verified truck accidents were included in the study occurring during 3.75 years on 46.5 mi (75.0 km) of freeway.

The study determined the characteristics of truck accidents, developed comparisons between truck and passenger vehicle accidents, and estimated the operational and economic consequences of truck accidents. An estimate of the total annual cost of urban freeway truck accidents was determined to be $634,000 per freeway mi (1.61 per km). Applying this estimate to the total 2,497 Interstate and freeway miles—with volumes greater than 100,000 vehicles per day—results in a nationwide annual cost of $1.6 billion.

Limited copies of this publication are available from the RD&T Report Center. Copies may also be purchased from the NTIS. (PB No. 90-208901/AS, Price code: A06.)

Recovery of Asphalt from Methylene Chloride and Trichloroethylene by the Abson Method, Publication No. FHWA–RD–89–207

by Pavements Division

The objective of this study was to determine if methylene chloride (CH_2Cl_2) can be used to recover asphalts using the Abson Method (AASHTO T-170 and ASTM D 1856), and to compare its effects on recovered binder properties to those of trichloroethylene (C_2HCl_3).

Current nationally standardized test procedures (AASHTO and ASTM) do not allow methylene chloride in the Abson Method. Virgin paving grade asphalts and hardened asphalts were used in this evaluation. Hardened asphalts consisted of paving grade asphalts aged by the thin film oven procedure, paving grade asphalts extracted from aged loose mixtures and cores, and coating grade roofing asphalts.

The following tests were performed before and after recovering the asphalts from trichloroethylene or methylene chloride: penetration at 25 °C, viscosity at 60 °C, viscosity at 135 °C, high pressure gel permeation chromatography, and infrared spectral analysis. The data indicated that methylene chloride can be used in the Abson Method. Both solvents had some statistically significant effects on some asphalt properties, but neither solvent could clearly be recommended over the other. The properties which were affected indicated hardening or increased molecular structuring. Methylene chloride may be a slightly better solvent because replicate asphalt samples recovered using this solvent provided more consistent data from sample to sample. For practical purposes, both solvents appear suitable for recovering asphalts.

Limited copies of this publication are available from the RD&T Report Center. Copies may also be purchased from the NTIS. (PB No. 90-208497/AS, Price code: A04.)

A Study Series on Office of Traffic Operation Selected Signing Issues, Publication No. FHWA–RD–89–232

by Traffic Systems Division

This report details a series of concurrently conducted laboratory studies on the effectiveness of various standard and/or proposed traffic control devices. In comparisons of bicycle symbol, lane shift symbol, and road-closed barricade designs, no model emerged as superior in terms of driver response.

In a construction zone, relative to the lane channelization split on a three-lane highway, the use of tubes extending from the center lane closure delayed subjects crossing from left to center lanes, and resulted in improved lane placement.

A one-way design which retains the directional cue when the text is obscured proved equal to the current standard, and may be an improvement under conditions of reduced visibility.

The specific arrangement of the elements (i.e., route number, cardinal direction, and directional arrow) of the route shield guide sign appears to affect the speed with which drivers comprehend this information. Subjects responded more rapidly to a design with a vertical arrangement of elements separated by horizontal lines, than to the standard route shield. Three of seven motorcycle warning designs (a word sign, and two symbol designs) performed equally well in a preliminary evaluation, thus providing a basis for further research.

This publication may only be purchased from the NTIS. (PB No. 90-150889/AS, Price code: A10.)

Audit of a Small Sign Support Computer Simulation, Publication No. FHWA–RD–89–233

by Safety Design Division

The objective of this task was to review, analyze, and report on two computer simulations of a small vehicle impacting frontally into a small, base-bending sign support. The theory behind the two simulations is presented in the report titled "Automobile Impact with Small Signs." The review performed and documented here is a combined effort by the Advanced Technology & Research Corp., Vanderbilt University, and the Texas Transportation Institute. Three independent reviews are presented followed by a summary of the overall opinions and combined recommendations. The three independent reviews all agree that additional work is necessary to develop a reliable computer model to be used for designing an advanced bogie test vehicle.

This publication may only be purchased from the NTIS. (PB No. 90-209610/AS, Price code: A08.)

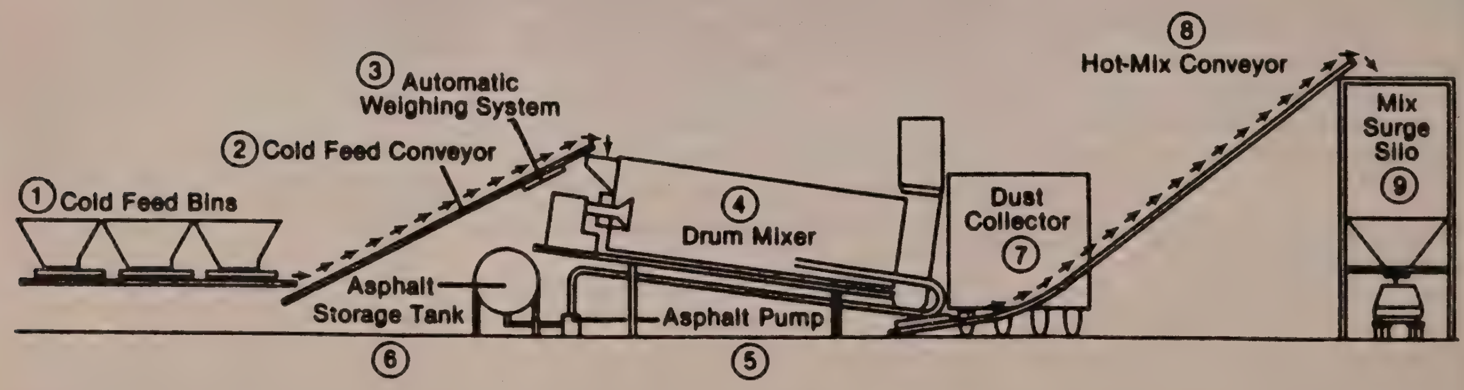
Changes Occurring in Asphalt in Drum-Dryer and Batch (Pug Mill) Mixing Operations, Publication No. FHWA–RD–88–195

by Materials Division

The study was designed to discover whether steam distillation of asphalt takes place in a drum-dryer mixer, compare changes induced by various laboratory conditioning (aging) techniques versus those occurring in drum-dryer mixers, and identify possible differences in asphalts subjected to drum-dryer mixing versus batch (pug mill) mixing operations.

Fifty-five virgin asphalts were subjected to various laboratory conditioning experiments including thin film oven exposure, rolling thin film oven exposure, small steam distillation, forced air distillation, and rolling forced air distillation. Various physical and chemical properties of these conditioned samples were measured. These properties were compared with those of the residues recovered from drum-dryer operations for each asphalt.

By comparing the laboratory conditioned residues to the recovered residues from the drum-dryer operation, similarities of the variously exposed asphalts to asphalt recovered from drum-dryer mixers were ascertained. This demonstrated that steam distillation does not take place in drum-dryer mixers. Eight matched asphalt pairs, 1 used in a drum-dryer mix and 1 in a batch (pug mill) mix, were identified among 24 virgin asphalts from Georgia by statistically comparing various physical, thermal, compositional, and molecular size properties of the virgin asphalts. Asphalts were then recovered from the mixes in which each of the eight drum dryer/batch (pug mill) asphalt pairs were used. The recovered asphalts were analyzed, and the results show the asphalt residues extracted from drum-dryer operations to be slightly harder than



Basic Drum-Mix Plant

those extracted from batch operations.

Limited copies of this publication are available from the RD&T Report Center. Copies may also be purchased from the NTIS. (PB No. 90-208075/AS, Price code: A06.)

Rollover Caused by Concrete Safety Barrier, Vol. I: Technical Report, Publication No. FHWA–RD–88–219

by Safety Design Division

This study was designed to identify the root causes of rollover of vehicles in impacts with concrete safety shaped barriers, determine the extent and severity of overturn collisions with concrete safety shaped barriers, and identify potential countermeasures to reduce shaped concrete barrier rollovers.

The study approach consisted of a critical review of literature, statistical and clinical analysis of four accident data files, and computer simulations. The extent of the rollover problem on concrete safety shaped barriers is found to be less than reported in previous literature. A number of impact conditions were identified from accident studies and confirmed by simulation as potential contributing factors to rollovers.

Three alternate shapes were evaluated as potential countermeasures: F-shape, single constant sloped barrier, and vertical wall. Results of the evaluation show that the F-shape barrier offers little performance improvement. The vertical wall barrier offers the greatest reduction in rollover potential, but also with the greatest increase in lateral accelerations. The single constant sloped barrier with an 80-degree slope may provide the best compromise solution. A benefit/cost analysis is needed and recommended to properly compare the various barrier shapes.

This is volume I of a two-volume final report. The other volume, FHWA–RD–88–209, contains appendixes that are too bulky for inclusion in this technical report.

This publication may only be purchased from the NTIS. (PB No. 90-208067/AS, Price code: A07.)

Luminaire and Sign Supports: Executive Summary, Publication No. FHWA–RD–88–224; Luminaire and Sign Supports: Technical Volume, Publication No. FHWA-RD-88-225

by Safety Design Division

A series of eight full-scale crash tests were conducted on five luminaire and sign supports. The test articles were embedded in S-1 soil as per National Cooperative Highway Research Program (NCHRP) No. 230 recommendations. The test vehicles used for all eight impact tests were 1979 Volkswagen Rabbits with the test mass adjusted to be in the range of 1,800 ± 50 lb (817 ± 23 kg). A triaxial accelerometer was placed near the vehicle center of gravity to record the vehicle decelerations.

Each test event was covered with one real-time and three to four high-speed movie cameras. The data acquired from the tests were processed as per the NCHRP No. 230 procedure. The results were evaluated against the dynamic performance requirements specified in the most recent American Association of State Highway Transportation Officials and NCHRP No. 220 documents. The detailed test and evaluation results and the test reports are presented in this final report.

This report is published in two parts: Executive Summary, Publication No. FHWA-RD-88-224 and Technical Volume, Publication No. FHWA-RD-88-225.

These publications may only be purchased from the NTIS. Executive Summary, Publication No. FHWA-RD-88-224 (PB No. 90-199654/AS, Price code: A03; Technical Volume, Publication No. FHWA-RD-88-225 (PB No. 90-199662/AS, Price code: A11.)

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

The following are brief descriptions of selected items that have been completed recently by State and Federal highway units in cooperation with the Office of Implementation, Office of Research, Development, and Technology (RD&T), Federal Highway Administration. Some items by others are included when the they are of special interest to highway agencies. All reports are available from the National Technical Information Service (NTIS). In some cases, limited copies of reports are available from the RD&T Report Center.

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Evaluation of Asphalt Stripping Tests in Oregon, Publication No. FHWA-TS-90-033

by Office of Implementation

As part of the continuing effort to establish a standard for evaluating moisture damage susceptibility of asphalt concrete design mixes, this study evaluated and compared 4 test methods by testing 15 diverse dense-graded mix designs during Oregon's 1988 construction season. The Index of Retained Strength (IRS) and the Index of Retained Modulus of Resiliency (IRM_R) were the existing Oregon Department of Transportation tests that were compared with the Root-Tunnicliff and the Modified Lottman procedures. None of the four tests predicted the same degree of asphalt stripping across the range of asphalt and aggregate tested.

The IRS test results show the most stripping-susceptible aggregates have a low test index when untreated, but when treated with lime or antistrip additive they have improved test indexes. The IRM_R test results show a greater inconsistency in the effectiveness of the lime-treated aggregate and the antistrip additive, but the test was the most severe of the four evaluated. Neither the Root-Tunnicliff nor Modified Lottman tests show consistently higher stripping indexes for lime-treated aggregate compared to antistrip additive. Additionally, the Root-Tunnicliff and Modified Lottman tests were the most difficult to perform and the least promising evaluated.

Based on the results of this study, the IRS test method continues to be considered a valid and useful stripping test. The IRM_R test appears to have the greatest potential for future improvement with its superior repeatability and apparent greater test severity.

As a result of the study findings, there still is a need for equipment and procedure improvements to increase the accuracy, precision, and correlations of stripping tests.

This publication may only be purchased from the NTIS. (PB No. 90-208851/AS, Price code: A05.)

Evaluation of Asphalt Stripping Tests in Montana, Publication No. FHWA-TS-90-039

by Office of Implementation

The actual moisture damage susceptibility of 10 bituminous mixtures placed in the field was compared to the moisture damage susceptibility that was predicted during the laboratory evaluation of the same mix design. Laboratory mixtures were evaluated using the Modified Lottman procedure and the Root-Tunnicliff procedure, in addition to routine moisture susceptibility testing which includes immersion-compression testing.

After 2 years, cores were taken from the field projects. The condition of the cores and their present susceptibility to moisture damage were determined by performing Modified Lottman and Root-Tunnicliff testing. A feature of the evaluation process was the use of the ACMODAS program to predict the remaining service life of the plant mix cores.

The validity of the process of predicting remaining pavement service using Modified Lottman or Root-Tunnicliff testing will not be known until the pavements in the study reach their terminal distress and require repair or rehabilitation. The comparisons of Modified Lottman, Root-Tunnicliff and immersion- compression testing data of the same mixtures provide some interesting results.

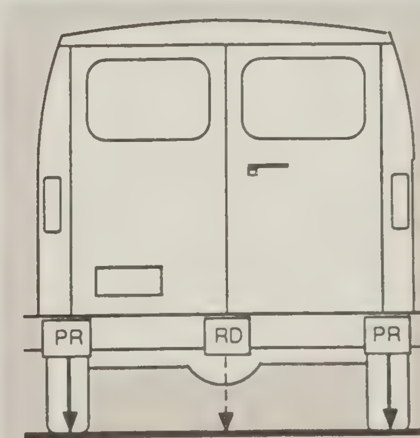
This publication may only be purchased from the NTIS. (PB No. 90-209024/AS, Price code: A05.)

Reducing Runaway Truck Accidents Through Weight-Based Advisory Speeds, Publication No. FHWA–IP–89–023

by Office of Implementation

Trucks are becoming heavier, and the design of more aerodynamic cabs and the use of radial and smaller-diameter tires may increase the potential for brake failure on downgrades. This report gives State transportation officials an overview of the Grade Severity Rating System (GSRS), a program to reduce runaway truck accidents through the use of weight-specific speed (WSS) signs. The report contains adequate information for State transportation officials to decide whether they want to implement GSRS-WSS and also tells potential users where to get the additional information needed for actual implementation.

The report summarizes the five steps required to implement WSS signing. The first step determines the magnitude of the runaway truck problem in a given State and identifies potential WSS sites. The second step analyzes the sites selected to determine percent and length of downgrade and truck braking length—the variables that determine brake temperature and hence safe speeds for different truck weight classes. In step three, analysts enter percent of grade and grade length or truck braking length into a computer program that yields advisory speeds for various truck weight classes. Step four converts the computer program output into the information that will actually be posted on WSS signs. Step five is the actual installation of signs before and preferably along the downgrades selected for WSS.



The report is based on the following studies: *Feasibility of a Grade Severity Rating System*, Publication No. FHWA/RD-79/116, *The Development and Evaluation of a Prototype Grade Severity Rating System*, Publication No. FHWA/RD-81/185, *Field Test of the Grade Severity Rating System (GSRS)*, Publication No. FHWA/RD-86/011, and *Grade Severity Rating System (GSRS)-Users Manual*, Publication No. FHWA-IP-88-015.

Limited copies of this publication are available from the RD&T Report Center. Copies may also be purchased from the NTIS. (PB No. 90-171240/AS, Price code: A03.)

Evaluation of the PRORUT System in Pennsylvania, Publication No. FHWA-TS-89-029

by Office of Implementation

The purpose of this report is to evaluate the usefulness of the profile and rut depth (PRORUT) measuring device to highway departments and to compare the PRORUT against other systems used to measure similar pavement characteristics. Other systems used in the evaluation include Mays Meters, the Portable Universal Roughness Device (PURD), rod-and-level, General Motors Modified Profilometer, and the K.J. Law Profilometer.

Test results show the best correlation is between the adjusted PRORUT (PRORUT minus outliers) and the Aerostar mounted Mays Meters. The PRORUT also has good correlation with all other equipment, except the rod-and-level.

The repeatability of testing with the PRORUT is good both by days (tests done on same day) and over time (different days). Testing done 1 year after the initial testing indicates good repeatability over extended periods of time.

The PRORUT has the potential to be a valuable addition to any highway department pavement management system for production testing. One key factor is having computer equipment in the office compatible with that in the PRORUT. Without the compatible equipment, however, the PRORUT is still applicable for research and calibration testing.

This publication may only be purchased from the NTIS. (PB No. 90-125421/AS, Price code: A06.)

Evaluation of the PRORUT System in Indiana, Publication No. FHWA-TS-89-030

by Office of Implementation

Purdue University and the Indiana Department of Transportation evaluated the performance of a profile and rut depth (PRORUT) device developed by the University of Michigan Transportation Research Institute. Several pavement sections with different characteristics were included in the evaluation. Accurate profiles were determined with manual surveying techniques. Subsequently, the PRORUT device was operated over the same pavement sections and an analysis was made to find the variance of the results. It was found that there was a close agreement between the PRORUT profiles and the rod-and-level profiles in all cases, except for a chip and seal section. Improvements were suggested to enhance the PRORUT operation.

This publication may only be purchased from the NTIS. (PB No. 90-125998/AS, Price code: A04.)

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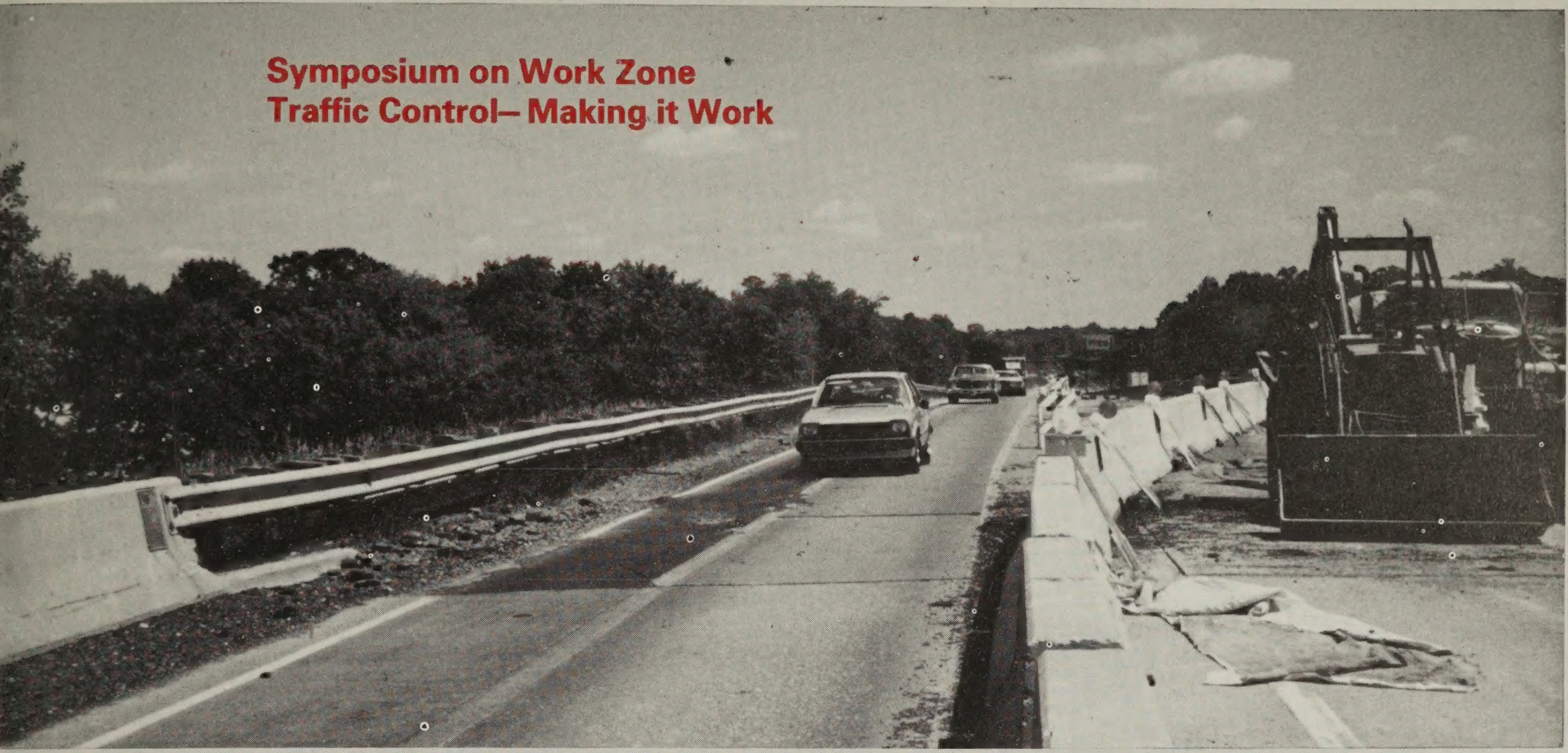
Symposium on Work Zone Traffic Control— Making it Work



Highway construction, maintenance, and utility activity occur everyday on the Nation's highway system. Since these work zones present an unusual situation for the motorist, they are potentially unsafe for both the motorists and the workers. Work zone traffic control can provide a safe environment for work crews, motorists, and pedestrians. Suitable protective devices exist, as do traffic control plans, but they are not always used effectively. Fatalities in highway work zones have increased indicating that we need to do a better job. Traffic control must be correctly deployed and monitored to *make it work*.

As part of its continuing emphasis on work zone safety and technology transfer, the Federal Highway Administration announces a 2-day Symposium on Work Zone Traffic Control in Orlando, Florida on January 18 and 19, 1991. The location and dates provide an opportunity for participants to attend the 21st Annual Convention and Traffic Exposition of the American Traffic Safety Services Association (ATSSA) which is being held January 20 through 22, 1991 at a nearby hotel.

The symposium will follow the structure of the very successful one held in 1985 in San Diego. There will be presentations by representatives of both the public agencies—Federal, State and local—and private contractors and consultants on a variety of timely topics. Also, there will be breakout sessions where the participants can share their experiences, procedures, and concerns.

Those from both the public and private sector who have direct responsibility for the design, implementation, or maintenance of work zone traffic control will benefit from attending this meeting and the ATSSA convention. On the day after the symposium, attenders can tour the ATSSA Traffic Expo where over 100 companies will display work zone traffic control and related products.

For more information contact: Peter Hatzi, the FHWA coordinator (703-285-2517), or Hugh McGee of Bellomo-McGee, Inc., the contractor responsible for organizing the meeting, (703- 847-3071).

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Public Roads

A Journal of Highway Research and Development