Vol. 48, No. 3
December 1984

U.S. Department
of Transportation

Federal Highway Administration

Public Roads

A Journal of Highway Research and Development

Public Roads

A Journal of Highway Research and Development
December 1984 Vol. 48, No. 3

COVER: Improved criteria need to be developed to determine the safest and most effective application of pedestrian signal indications.

U.S. Department of Transportation
Elizabeth Hanford Dole, *Secretary*

Federal Highway Administration
R. A. Barnhart, *Administrator*

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

Public Roads is published quarterly by the Offices of Research, Development, and Technology

David K. Phillips, *Associate Administrator*

Technical Editor
C. F. Scheffey

Editorial Staff
Cynthia C. Ebert
Carol H. Wadsworth
William Zaccagnino

Advisory Board
R. J. Betsold, S. R. Byington, R. E. Hay

IN THIS ISSUE

Articles

Departments

Address changes (send both old and new) and requests for removal should be directed to:

Public Roads Magazine, HRD-10
Federal Highway Administration
6300 Georgetown Pike
McLean, Virginia 22101

At present, there are no vacancies on the *FREE* mailing list.

Public Roads, A Journal of Highway Research and Development, is sold by the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402, for $12 per year ($3 additional for foreign mailing) or $3.25 per single copy (81¢ additional for foreign mailing). Subscriptions are available for 1-year periods. Free distribution is limited to public officials actually engaged in planning and constructing highways and to instructors of highway engineering. At present, there are no vacancies on the free mailing list.

The Secretary of Transportation has determined that the publication of this periodical is necessary in the transaction of the public business required by law of this Department. Use of funds for printing this periodical has been approved by the Director of the Office of Management and Budget through March 31, 1985.

The Application of Pedestrian Signals at Traffic-Signalized Intersections[1]

by
H. Douglas Robertson

Introduction

Pedestrian signals have been in use in the United States since the 1920's. (*2*)[2] Although viewed by many engineers as a safety improvement, studies to date have not entirely sustained this premise. (*1*) In some instances, the correlation between pedestrian signal installations and public pressure is far greater than the correlation between pedestrian signal installations and accident reduction or improved operations.

Presently, the major criteria for installing pedestrian signals are based on vehicle and pedestrian volumes, insufficient gaps in traffic, and engineering judgment. (*3*) The operation and timing of the pedestrian signals primarily is a function of traffic demand and, in some cases, pedestrian walking speed and crossing distance. (*4*)

Specifically, the *Manual on Uniform Traffic Control Devices* (MUTCD) contains two warrants based on pedestrian demand for installing *traffic signals*—the minimum pedestrian volume warrant and the school crossing warrant. (*3*) When either of these warrants is applicable, pedestrian indications also are required. In addition, certain special situations are outlined where pedestrian signal indications may be installed. However, no warrant specifically addresses the installation of pedestrian signals at traffic-signalized intersections.

Several studies indicate that other important factors, such as pedestrian behavior, compliance, and delay, should be included as criteria or warrants for installing and operating pedestrian signals. (*1*) It is evident from reviewing the literature that a better means of evaluating both the necessity for pedestrian signals and the appropriate operation of those signals is needed.

In addition to safety and operational considerations, reduced operation and maintenance funds and the need to conserve energy further justify optimal and judicious use of pedestrian signals. Traffic engineers do not have sufficient information to determine where pedestrian signals are needed most and how the signals should be operated to best meet all requirements.

---

[1]This article summarizes the results of a staff study conducted by Dr. Robertson in part when he was a research engineer in the Federal Highway Administration's Traffic Systems Division. A complete documentation of the study is contained in Dr. Robertson's doctoral dissertation "Signalized Intersection Controls for Pedestrians." See reference 1 on page 87.

[2]Italic numbers in parentheses identify references on page 87.

The Federal Highway Administration (FHWA) research described in this article concentrated on the application of pedestrian signals at traffic-signalized intersections. The study was conducted to determine the nature and extent of the problems pedestrians experience at intersections both with and without pedestrian signals, to develop improved criteria and warrants for determining where pedestrian signals should or should not be used based on sound human factors and traffic engineering principles, and to develop practical guidelines that traffic engineers could use in applying the warrants.

Nature and Extent of the Problem

In 1981, 26,700 pedestrians were killed or injured at intersections. (*5*) Evidence indicates that traffic signals improve pedestrian safety. Pedestrian indications, when properly applied to meet specific pedestrian needs, are thought to provide additional safety, the extent of which has not been established. In short, pedestrian indications seem to reduce accidents and/or accident potential at some intersections, have little or no effect at other intersections, and even increase accidents at still other intersections. Clearly, the *conditions* under which pedestrian indications enhance safety must be determined.

The *presence* of pedestrian signal indications does not appear to affect significantly the performance of an intersection as measured by pedestrian and vehicle delay. The *operation* of those indications in conjunction with traffic signals (in terms of phasing and timing), however, has a profound effect on delay. When traffic *and* pedestrian signals are used, they must be properly timed.

Until recent years, the cost of providing and operating traffic and pedestrian signals was not a major problem for most jurisdictions. During the 1950's through the early 1970's, intersection signalization experienced tremendous growth. In the absence of more definitive information many jurisdictions were guided by generally worded warrants and guidelines in the MUTCD and adopted widespread use of pedestrian indications. For example, the City of Los Angeles, California, had pedestrian indications at 89 percent of its traffic-signalized intersections in 1974.[3]

Since 1974, inflation has reduced buying power and the ability of government budgets, further reduced by rising energy costs, to sustain the growth of signal control. The luxury of signal control that does not produce a reasonable and necessary benefit can no longer be afforded.

With no relief to the economic and energy problems in sight, ways must be found to reduce costs. Because pedestrian indications offer a cost-reduction target, it is critical that the conditions for their effective use be determined so that their safety benefits will not be lost in an arbitrary move to cut costs. The development of improved criteria and warrants for the use of pedestrian controls at traffic-signalized intersections is a necessary first step.

The following three existing pedestrian accident data bases containing data on approximately 5,100 intersection accidents and representing 20 different urban areas were used for analysis in the FHWA study:

- 2,685 pedestrian intersection accidents in Washington, D.C., from 1971-1973. Exposure data were available for a portion of this data base in the form of pedestrian volumes and vehicle volumes by turning movement.
- 973 pedestrian intersection accidents from 13 cities. (*6*)
- 1,443 pedestrian intersection accidents from 7 cities. (*7*)

[3]S. E. Rowe, L. P. Nepsund, and M. G. Paetzold, "Traffic Signal Statistics," Department of Traffic, City of Los Angeles, Calif., November 1974. Unpublished report.

An examination of 81 intersections in Washington, D.C., for which pedestrian and vehicle volumes were available revealed that the average number of pedestrians and vehicles per intersection with pedestrian signals was twice that of intersections with traffic signals only, which, in turn, was twice that of nonsignalized intersections. To determine whether the pedestrian accident rate is different at intersections with different kinds of control, 47 of these intersections (23 with pedestrian signals, 19 with traffic signals only, and 5 with no signals) were analyzed. Pedestrian accident rates were calculated for each intersection by dividing the number of pedestrian accidents in 3 years during a 10-hour period by a sample of the pedestrian volume during the same 10-hour period. Mean pedestrian accident rates then were calculated for each kind of control. Tests of significance (Student's t) revealed that the intersections with traffic and/or pedestrian signals had a significantly lower accident rate than nonsignalized intersections. There was no significant difference in mean accident rates between intersections with pedestrian signals and intersections with traffic signals only. These results imply that signalized intersections are safer than nonsignalized intersections and that intersections with pedestrian signals are no safer than intersections with traffic signals only. Caution should be exercised when using these findings, however, because the small samples may not be representative.

More substantial evidence is presented in a recent study that found no significant difference in pedestrian accidents between intersections with standard-timed pedestrian signals compared with intersections with traffic signals only. (*8*) The study was based on data from 1,100 intersections in 15 U.S. cities, and the analysis controlled for both pedestrian and vehicle volumes as well as one-way/two-way operation. Although nonsignalized intersections were not examined, the study offers strong evidence that, in general, intersections with pedestrian signal indications are no safer than intersections with traffic signals alone.

Using the Washington, D.C., data base, a study of the relationship of turning vehicles to pedestrian accidents revealed the following:

- Of the 202 pedestrian accidents that occurred, 29 percent involved turning vehicles.
- The average ratio of turning vehicles to total vehicles entering an intersection was 17 percent.
- Left-turning accidents accounted for 59 percent of the total turning accidents.
- Left turns represented 44 percent of the total turns.

These data indicate that turning vehicles, and in particular left-turning vehicles, were overrepresented in these pedestrian intersection accidents. Overall, left-turning vehicles were almost three times more hazardous to pedestrians than through-moving vehicles. Evidence supports the contention that turning movements, particularly left-turning movements, present a safety problem for pedestrians crossing at intersections and that the problem may be more acute at traffic-signalized intersections. Additionally, the presence of pedestrian indications may create a false sense of security, making pedestrians think they are protected and have no reason to be cautious. (*8*) The absence of pedestrian indications makes pedestrians feel that they must rely on their own senses and judgment and thus exercise more caution, particularly with regard to turning vehicles.

Consideration of the relationships between age and sex and pedestrian accidents led to using population by age and sex groupings and accidents by age and sex categories to obtain exposure. This resulted in a determination of risk versus age and sex. (fig. 1).

Several other factors, including pedestrian accident characteristics, intersection geometrics, vehicle and pedestrian volume and delay, and the benefits and costs of pedestrian signal operation were studied and analyzed to develop criteria and warrants for pedestrian signals at traffic-signalized intersections. (*1*)

Development of Criteria and Warrants

The first step in the development of criteria and warrants for using pedestrian signal indications was to define and understand all of the factors involved in the pedestrian crossing situation and the relationships among those factors. After a careful examination of many kinds of analytical models (for example, risk models, delay models, and gap-acceptance models), it was determined that a comprehensive

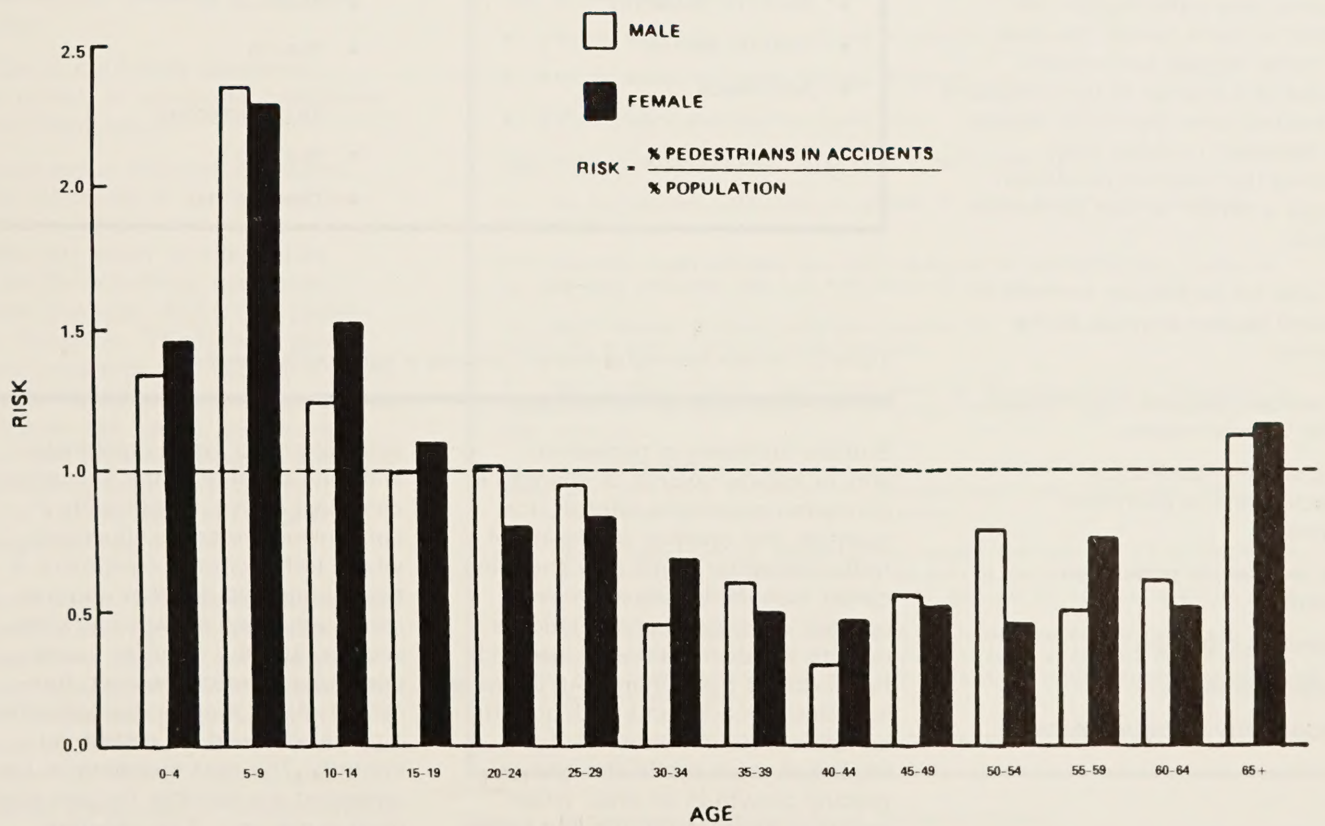


Figure 1.—Pedestrian accident risk by age and sex based on exposure.

behavioral model would best serve the needs of this study. This determination was based, in part, on the fact that of the three basic highway accident causal factors—human, environmental, and vehicular—the human factor was cited as a definite or probable or severity-increasing factor in over 90 percent of 2,258 motor vehicle accidents analyzed between 1972 and 1977 at Indiana University. (*9*) Therefore, a model that focused on the prime accident causal factor appeared to be the most promising. The model developed focused on factors that affect pedestrians crossing at intersections (fig. 2).[4]

Criteria considered in this study addressed *only* the use of pedestrian signal indications and/or pushbuttons at intersections where *traffic signals* either already were in place or presumed to be warranted. The criteria, warrants, and guidelines are offered as an improvement to the guidelines presently stated in the MUTCD.

Pedestrian signals may be installed at the same time traffic signals are installed or, as is usually the case, after traffic signals are installed because of a change in the conditions that existed when the traffic signals were installed. In either case, assessing the need for pedestrian controls is similar to that for traffic controls.

The need for pedestrian controls is indicated by one or more of the following:

- A sudden increase in pedestrian and/or vehicle volume.
- A change in pedestrian characteristics or geometric conditions.
- An increase in pedestrian accidents.
- Citizen complaints.
- Political pressure.
- Opportunity or convenience.

<u>Geometric/physical</u>

- Median islands
- Lighting
- Parking
- Street width
- Sight distance

<u>Human</u>

- Age
- Sex
- Physical disability
- Walking speed
- Compliance
- Risk taking
- Gap acceptance
- Group behavior
- Erratic behavior
- Understanding
- Accidents

<u>Traffic</u>

- Vehicle volume
- Pedestrian volume
- Vehicle speed
- Vehicle mix
- Vehicle directional split
- Vehicle delay
- Pedestrian delay
- Vehicle arrivals
- Pedestrian arrival
- Gap distribution

<u>Controls</u>

- Traffic signals
- Pedestrian signals
- Pedestrian pushbuttons
- Phasing
- Timing

<u>Environmental</u>

- Weather
- Time of day

Figure 2.—Factors affecting pedestrians crossing at signalized intersections.

Sudden increases in pedestrian and/or vehicle volume or changes in geometric conditions refer to, for example, the opening or closing of a traffic generator, such as a shopping center, school, or housing development. Pedestrian characteristics refer to the distributions of age and handicapped pedestrians. An increase in pedestrian accidents and/or citizen complaints normally warns of the impact of more subtle changes or gradual growth in an area, which results in increased traffic and pedestrian demand. Political pressure, a reality the traffic engineer must deal with, often occurs in combination with one or more of the indicators described above. Opportunity or convenience refers to situations where traffic control equipment is being upgraded or other improvements are being made to an intersection, and the need for future pedestrian control has been forecasted. All of these indicators reflect a perceived need for pedestrian controls. The next questions to be answered are whether the perceived need is real and, if so, whether pedestrian controls will meet that need in the most effective and efficient manner.

[4]This model development is discussed fully in reference 1.

Because of their limited safety benefit and cost of installation, operation, and maintenance, pedestrian signal indications should be used only where conditions hazardous to a significant proportion of the crossing pedestrians cannot be mitigated by other suitable means. At many intersections, pedestrian indications simply duplicate traffic signal functions. Traffic signals at intersections help pedestrians cross safely by providing gaps and right-of-way in approaching traffic streams. Current warrants for traffic signals to serve this function are based primarily on pedestrian and vehicle volumes.

The need for pedestrian signal indications, however, is not only a function of pedestrian and vehicle volumes; high volumes may be accommodated by traffic signals alone. Therefore, the criteria to determine the need for pedestrian controls focus on the *additional* aid pedestrian signal indications provide. Pedestrian controls should:

1. Improve pedestrian safety.

2. Accommodate the needs of special pedestrians (young, old, and handicapped) crossing at the intersection.

3. Clarify confusing geometric, operational, or situational conditions at the intersection.

4. Improve or maintain the operational efficiency of the intersection.

Pedestrian safety is reflected by pedestrian accidents, pedestrian/vehicle conflicts, and erratic pedestrian behaviors. The level of pedestrian compliance also may be a safety indicator, because compliance reflects pedestrian risk taking, gap acceptance, and understanding. In addition, pedestrian group behavior often results in a decrease in compliance, prompting unsafe acts.

Special pedestrians include the young (usually under age 15), the elderly (usually over age 60), and the handicapped (those with either mobility or vision disabilities). The needs of special pedestrians generally are exhibited by slower walking speeds, impaired vision, lack of understanding of signal indication meanings, and poorer judgment with respect to approaching vehicle speeds and distances.

Examples of confusing geometric, operational, or situational conditions are as follows:

- Traffic signal indications that are not visible to pedestrians, such as on one-way streets and at "T" intersections.

- Parked vehicles, inadequate lighting, inclement weather, or "street furniture" that reduce sight distance (the length of intersection approach that drivers can see pedestrians and pedestrians can see approaching vehicles).

- Pedestrian clearance intervals (a function of street widths and walking speeds) that exceed the clearance intervals required by vehicles.

- Multiphase indications (as with split-phase timing) that confuse pedestrians guided only by traffic signal indications.

- Extreme street widths such that pedestrians can cross only part of a street during a particular interval.

Operational efficiency of an intersection usually is measured by vehicle and pedestrian delays. These delays are affected by signal timing, vehicle and pedestrian volumes, and vehicle and pedestrian arrival (or gap) distributions. Delay also may be affected by vehicular and pedestrian noncompliance with the signal.

Based on the results of this study, a recommended Pedestrian Signal Indication Warrant is shown in figure 3.[5] The first condition of the warrant insures that pedestrians are using the intersection crossing. The remaining

[5] A detailed discussion of each of the warrant conditions is contained in reference 1.

Pedestrian signal indications at traffic-signalized intersections are warranted if:

1. The number of pedestrians crossing at the intersection per hour for an average day is:

- 100 or more for each of any 4 hours,
- 140 or more for each of any 2 hours, or
- 170 or more during the peak hour

<u>and</u> one or more of the following conditions exist:

2. An exclusive interval or phase is provided for pedestrian movement.

3. Traffic indications are not visible to pedestrians, such as on one-way streets and at "T" intersections.

4. Multiphase indications are confusing to pedestrians guided only by traffic signal indications.

5. The pedestrian's sight distance to an approaching vehicle is impaired or is less than the average stopping sight distance of the vehicle.

6. The pedestrian clearance interval exceeds the corresponding vehicle clearance interval or pedestrians can cross only part of a street during a particular interval.

7. The proportion of pedestrians under the age of 15 crossing at the intersection is more than 1 standard deviation above the mean proportion of pedestrians under the age of 15 in the control base.

8. The proportion of elderly and/or handicapped pedestrians crossing at the intersection is more than 1 standard deviation above the mean proportion of elderly and/or handicapped pedestrians in the control base.

9. A significant improvement in operational efficiency will result.

Figure 3.—Recommended Pedestrian Signal Indication Warrant.

eight conditions (one or more of which must be met along with the first condition and that cannot be corrected) restrict the use of pedestrian indications to meeting specific pedestrian safety and operational needs. This recommended warrant is intended to reduce the present level of use of pedestrian signal indications and can be applied to remove existing pedestrian signal indications.

Warrant Application Procedure

The guidelines for applying the recommended Pedestrian Signal Indication Warrant are structured to maximize use of existing information and minimize additional data collection. Following the guidelines also insures that other less expensive but effective means to mitigate the particular pedestrian problems experienced at the intersection are considered before pedestrian signal indications are installed. Table 1 summarizes the procedure for applying the Pedestrian Signal Indication Warrant to the installation of pedestrian indications.

Table 1.—Procedure for applying the Pedestrian Signal Indication Warrant (installation)

Question	Response	Action required
1. Is the minimum level of pedestrian usage exceeded?	Yes No	Go to question 2 Do not install pedestrian indications
2. Is an exclusive interval or phase provided for pedestrian movement?	Yes No	Go to question 10 Go to question 3
3. Are traffic indications visible to pedestrians?	Yes No	Go to question 4 Go to question 10
4. Are multiphase indications in use that are confusing to pedestrians?	Yes No	Go to question 10 Go to question 5
5. Is pedestrian sight distance impaired or less than the average vehicle stopping sight distance?	Yes No	Go to question 10 Go to question 6
6. Does the pedestrian clearance interval exceed the vehicle clearance interval, or can pedestrians cross only part of a street during a particular interval?	Yes No	Go to question 10 Go to question 7
7. Does the proportion of pedestrians under the age of 15 crossing exceed the criterion value established by the control base?	Yes No	Go to question 10 Go to question 8
8. Does the proportion of elderly and/or handicapped pedestrians crossing exceed the criterion value established by the control base?	Yes No	Go to question 10 Go to question 9
9. Will a significant improvement in operational efficiency result?	Yes No	Go to question 10 Do not install pedestrian indications
10. Are other cost-effective means available to correct the condition?	Yes No	Do not install pedestrian indications; examine remaining questions Install pedestrian indications

So far this article has addressed the need for pedestrian signal indications at traffic-signalized intersections where they do not presently exist. There are between 57,500 and 69,000 signalized intersections with pedestrian indications in the United States. (*10, 11*) It is estimated that only 15 percent of these pedestrian indications were installed under the MUTCD pedestrian volume or school crossing warrants.[6] Increasing pressure to reduce operations and maintenance costs of traffic control devices and findings that pedestrian indications at many locations offer no significant safety benefit (*8*) raise the issue of whether pedestrian indications should be removed from intersections where they are no longer needed. Assuming that 85 percent of the pedestrian-signalized intersections in the United States did not meet the MUTCD warrants for pedestrian indications, between 48,900 and 58,700 intersections would be candidates for review for removal using the recommended warrant.

Indications of the lack of need for pedestrian signals include reduced pedestrian and/or vehicle volumes, the removal of established school crossings, a change in traffic signal location or operation, and a change in geometric or other intersection features that increases sight distance.

The procedure for applying the Pedestrian Signal Indication Warrant to *remove* pedestrian indications from a signalized intersection is the same as described in table 1, but in a slightly different sequence (table 2). For jurisdictions with limited budgets, pedestrian indications could be removed from locations where they are not needed and then reinstalled at new locations that warrant their use.

Conclusions

The data gathered in this study imply that pedestrian indications offer a safety benefit *at some locations* without a detrimental effect on traffic operations. The exact conditions under which pedestrian indications are most effective are not clearly defined; however, these conditions must be determined so that pedestrian indications can be used most effectively and their benefits are not lost in an arbitrary move to cut costs.

The recommended Pedestrian Signal Indication Warrant developed in this study offers a preferable alternative to arbitrary decisionmaking by identifying the intersections where pedestrian indications should be used based on specific pedestrian needs. The warrant also can be used to determine if existing pedestrian indications need to be removed. The warrant is not meant to replace current MUTCD warrants but to

[6] E. B. Lieberman, G. F. King, and R. B. Goldblatt, "Traffic Signal Warrants," draft report NCHRP 3-20 submitted to the National Cooperative Highway Research Program, Washington, D.C., December 1976, pp. 23-27.

Table 2.—Procedure for applying the Pedestrian Signal Indication Warrant (removal)

Question	Response	Action required
1. Is an exclusive interval or phase provided for pedestrian movement?	Yes No	Go to question 9 Go to question 2
2. Are traffic indications visible to pedestrians?	Yes No	Go to question 3 Go to question 9
3. Are multiphase indications in use that are confusing to pedestrians?	Yes No	Go to question 9 Go to question 4
4. Is pedestrian sight distance impaired or less than the average vehicle stopping sight distance?	Yes No	Go to question 9 Go to question 5
5. Does the pedestrian clearance interval exceed the vehicle clearance interval, or can pedestrians cross only part of a street during a particular interval?	Yes No	Go to question 9 Go to question 6
6. Does the proportion of pedestrians under the age of 15 crossing exceed the criterion value established by the control base?	Yes No	Go to question 9 Go to question 7
7. Does the proportion of elderly and/or handicapped pedestrians crossing exceed the criterion value established by the control base?	Yes No	Go to question 9 Go to question 8
8. Will a significant loss in operational efficiency result?	Yes No	Go to question 9 Remove pedestrian indications
9. Are other cost-effective means available to correct the conditions?	Yes No	Remove pedestrian indications and implement other means [1] Go to question 10
10. Is the minimum level of pedestrian usage present?	Yes No	Do not remove pedestrian indications Remove pedestrian indications [1]

[1] Questions 1-8 must be examined before the pedestrian indications can be removed.

extend them by providing more detailed guidance. The warrant includes several factors not included in the MUTCD, such as sight distance for pedestrians, presence of special pedestrians, signal compliance, erratic behaviors, and vehicle and pedestrian delay. Application of the warrant may require some data collection and analysis, but not as extensive as a cost-benefit type analysis. Following the step-by-step procedure for applying the warrant will eliminate unnecessary data collection.

Evidence that pedestrian indications are overused and thus contribute to unnecessary costs and delays and possibly reduced safety points to the need for more judicious use of pedestrian indications at signalized intersections. The warrant and guidelines described in this article should be useful to traffic engineers in determining the safest and most effective application of pedestrian signal indications.

REFERENCES

(*1*) H. D. Robertson, "Signalized Intersection Controls for Pedestrians," Ph.D. Dissertation, *Department of Civil Engineering, University of Maryland*, College Park, Md., August 1983. (A copy may be obtained by writing University Microfilms International, 300 N. Zeeb Rd., Ann Arbor, Mich. 48106.)

(*2*) G. M. Sessions, "Traffic Devices: Historical Aspects Thereof," *Institute of Traffic Engineers*, Washington, D.C., 1971, p. 134.

(*3*) "Manual on Uniform Traffic Control Devices for Streets and Highways," *U.S. Department of Transportation*, Washington, D.C., 1978, pp. 4C-1—4D-5.

(*4*) "Transportation and Traffic Engineering Handbook," *Institute of Traffic Engineers*, Englewood Cliffs, N.J., 1982.

(*5*) "Accident Facts," *National Safety Council*, Chicago, Ill., 1982, pp. 45-47, 55, and 61.

(*6*) M. B. Snyder and R. L. Knoblauch, "Pedestrian Safety: The Identification of Precipitating Factors and Possible Countermeasures, Volumes I and II, Appendices," Report No. FH-11-7312, *National Highway Safety Bureau*, Washington, D.C., January 1971.

(*7*) R. L. Knoblauch, "Urban Pedestrian Accident Countermeasures Experimental Evaluation, Volume II—Accident Studies," Report No. DOT-HS-801 347, *U.S. Department of Transportation*, Washington, D.C., February 1975.

(*8*) C. V. Zegeer, K. S. Opiela, and M. J. Cynecki, "The Effect of Pedestrian Signals and Signal Timing on Pedestrian Accidents," Transportation Research Record No. 847, *Transportation Research Board*, Washington, D.C., 1982, pp. 62-72.

(*9*) J. R. Treat, "A Study of Precrash Factors Involved in Traffic Accidents," *The HSRI Research Review*, Highway Safety Research Institute, May-June/July-August 1980, p. 6.

(*10*) "Traffic Control, Roadway Lighting, and Parking Equipment Market," *Frost and Sullivan*, New York, N.Y., March 1979.

(*11*) H. D. Robertson, W. G. Berger, and R. F. Pain, "Urban Intersection Improvements for Pedestrian Safety, Vol. II—Identification of Safety and Operational Problems at Intersections," Report No. FHWA-RD-77-143, *Federal Highway Administration*, Washington, D.C., December 1977. (Report may be purchased from the National Technical Information Service, 5285 Port Royal Rd., Springfield, Va. 22161. Stock No. PB 286498.)

H. Douglas Robertson is an Associate Professor of Civil Engineering at the University of North Carolina at Charlotte. Dr. Robertson specializes in highway safety and traffic operations research. He has held responsible positions in the Federal Highway Administration, the National Highway Traffic Safety Administration, and two engineering consulting firms. His research in the area of pedestrian safety and control led to the adoption of symbolic pedestrian signal displays as a national traffic control standard.

Soil Reinforcement for Stabilization of Earth Slopes and Embankments

by
James K. Mitchell, Willem C. B. Villet, and Albert F. DiMillio

This article is based on a paper presented at the March 1984 Federally Coordinated Program of Highway Research, Development, and Technology (FCP) Conference in McLean, Virginia. Federal Highway Administration (FHWA) ground improvement research on soil reinforcement and other methods (for example, dynamic compaction, prefabricated consolidation drains, and stone columns) is included under FCP Project 5P, "Foundations and Earth Structures." The main focus of the ground improvement research is to develop improved design and construction guidelines for new ground improvement techniques that show potential for profitable use on highway projects. Case history examples and cost-effectiveness data also will be developed to help highway engineers become aware of the potential of these relatively new techniques.

Soil reinforcement research under FCP Project 5P began with a recently completed National Cooperative Highway Research Program study (discussed in this article) to identify, describe, and catalog existing soil reinforcement methods. Current research involves laboratory and full-scale field tests to further define the mechanisms and engineering behavior of reinforced soil structures.

Introduction

Soil reinforcement, the inclusion of resistant elements in a soil mass for use as a construction material or for increasing the strength and stability of existing soil masses in situ, has emerged as one of the most significant advances in geotechnical engineering and earthwork construction of the past decade. The result of soil reinforcement is a *composite material* that combines the best features of the soil (compressive and shear strength) and of the reinforcement (tensile strength) and possesses features similar to reinforced concrete, another well known and widely used composite construction material.

Several soil reinforcement systems are available for walls, slopes, and embankments. Some are best suited for low structures, some for high structures. Some are best used in remote, lightly traveled areas, and others must withstand the rigors of high-traffic-volume urban areas. Some systems can be used with poorer quality soils than can others.

This article presents an overview of reinforcement techniques to stabilize earth slopes and embankments. Slopes are stabilized either by constructing reinforced soil retaining structures (fig. 1) or by installing reinforcements directly into a slope (fig. 2). Embankments usually are stabilized by installing reinforcing elements within the embankment (fig. 3), thus enabling the use of steeper slopes and increased total loading on underlying soft foundation soils. Reinforcement techniques are particularly well suited for highway applications.

Evolution of Soil Reinforcement and Current Highway Applications

The use of tensile inclusions in soil for reinforcement dates back several thousand years. (*1*) [1] Modern development of soil reinforcement for construction of retaining walls was pioneered in France, with the first highway use for a wall near Nice (fig. 4). (*2*) The first use of Reinforced Earth walls in the United States was in 1972 to provide support for California State Highway 39 along a steep slope in the San Gabriel Mountains north of Los Angeles (fig. 5). Since then hundreds of reinforced soil walls and abutments have been constructed for highway applications throughout the United States.

Concurrent with these developments has been the increased study and application of geotextiles for reinforcement. Among the early applications of geotextiles was their inclusion between very weak and soft foundation soils and gravel layers used for expedient roads or road bases. The geotextile sheet prevents intrusion of the fine soil into the gravel and serves as a tensile reinforcement to increase the load carrying capacity of the

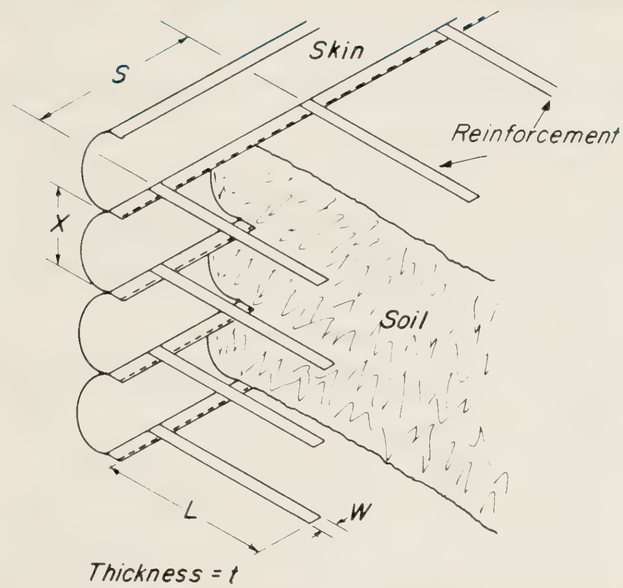


Figure 1. — Reinforced soil retaining structure.

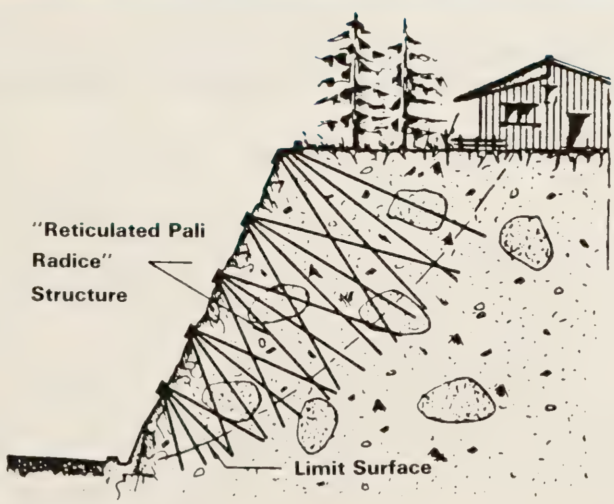


Figure 2. — Reinforced slope.

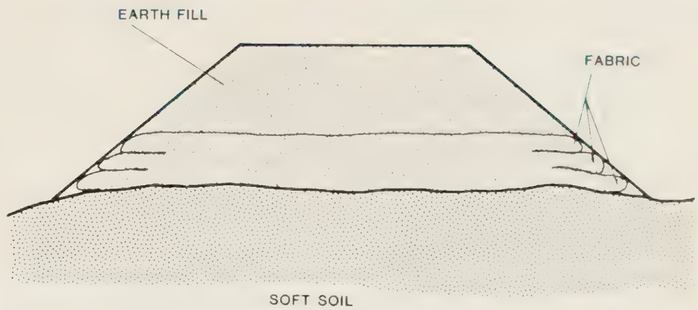


Figure 3. — Reinforced embankment.


Figure 4. — First highway use of modern Reinforced Earth wall, near Nice, France.

[1]Italic numbers in parentheses identify references on page 95.

roadway. High tensile strength, high modulus geotextiles now are used both for construction of fabric-reinforced soil walls (figs. 6 and 7) and for embankment reinforcement (fig. 3).

Laboratory and analytical studies (*3*, *4*) and full-scale field tests (*5*) have been conducted on grid cells for reinforcement. Early work was directed at the use of sand-filled interconnected cells for road base applications (fig. 8). Most recently the concept of earth-filled cells has been extended to embankment reinforcement in which Tensar Geogrids are arranged to form cells (fig. 9). The result is a stiffened layer that provides improved support for embankments over soft foundations.

Many of the current soil reinforcement techniques were developed specifically for highway applications because their use offers some significant advantages including the following:

- Steeper slopes can be used, so the required widths of new rights-of-way are reduced and the traffic corridors in existing rights-of-way can be widened.
- Reinforced soil walls and abutments cost less than reinforced concrete and other wall types over a significant range of wall heights.
- Construction of reinforced soil structures usually is easy and rapid.
- Generally, little site preparation is required, and construction disruptions to traffic and utility lines are less than with conventional techniques.
- Prefabricated components can be used.
- Esthetic structures can be designed.
- Reinforced soil structures are more tolerant of internal and external ground movements.
- Competition among the developers and marketers of different soil reinforcement systems and materials has resulted in product innovation and reduced costs.

Figure 5. – Construction of the first Reinforced Earth wall in the United States (along State Highway 39 in California).

Figure 6. — Fabric-reinforced soil wall.

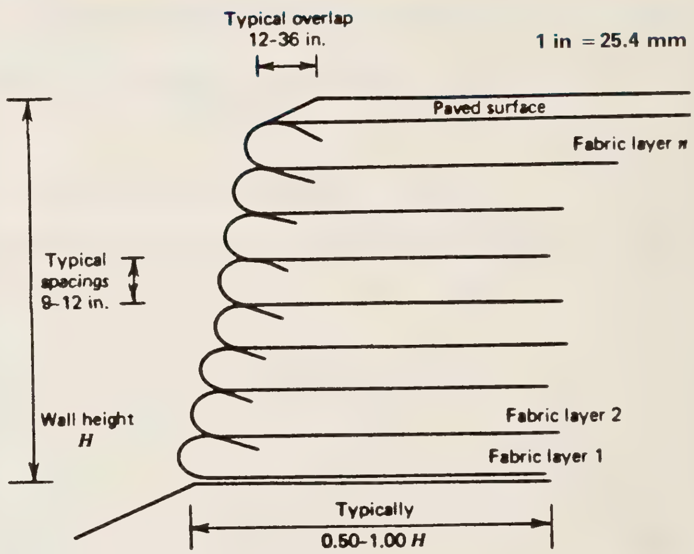


Figure 7. — Schematic diagram of fabric-reinforced soil wall.

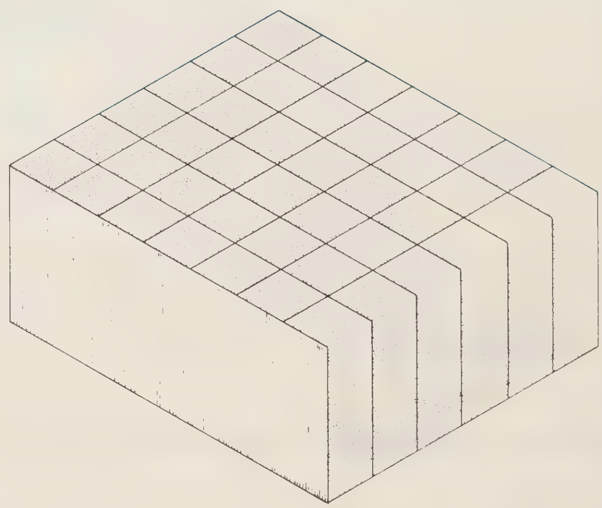


Figure 8. — Concept of grid cell reinforcement for road bases.

Reinforcement Mechanisms

The stability of a reinforced soil system depends on the satisfactory transfer of stress between the soil and reinforcements. Linear, sheet, and grid reinforcements with high strength and tensile stiffness transfer load to the surrounding soil by frictional and passive soil resistance when the reinforcements are loaded in tension. The load that can be transferred by friction (fig. 10) per unit area of reinforcement depends on the interface characteristics of the soil and reinforcing materials and on the normal stress between them, which depends on the stress-deformation behavior of the soil. This behavior, for most soils used in reinforcement applications, is itself stress-dependent. Thus, analysis alone cannot readily estimate the effective friction coefficient. Accordingly, the results of experiments, such as pullout tests, direct shear tests between soil and reinforcement, and instrumented model and full-scale tests, often are used to select appropriate values. (*6*)

Load transfer by passive soil resistance (fig. 11) occurs when a bearing surface normal to the direction of the force to be resisted is pulled into the soil. The classic example of the use of passive resistance in a retaining structure is a deadman anchor. However, a system of tendons and anchors is not at all the same as soil reinforcement. Several embankment wall systems are available that are variants of anchored sheet pile walls or bin walls. They consist of facing elements and ties or tendons of high tensile strength extending to some distance in the soil where they are connected to vertical plates or other anchor types that provide the resisting forces needed for stability of the wall face. These anchor and bin wall systems differ from reinforced soil in that the soil between the face and the anchors does not behave as a composite material and the wall facing must resist, as a minimum, the full active earth pressure of the retained soil. Reinforced soil systems can be sampled to give elements having properties representative of the whole; anchored systems cannot.

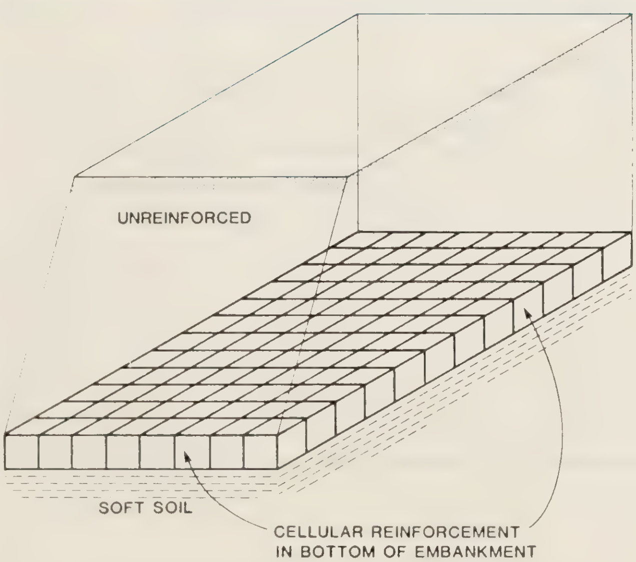


Figure 9.—Embankment reinforcement using grid cells.

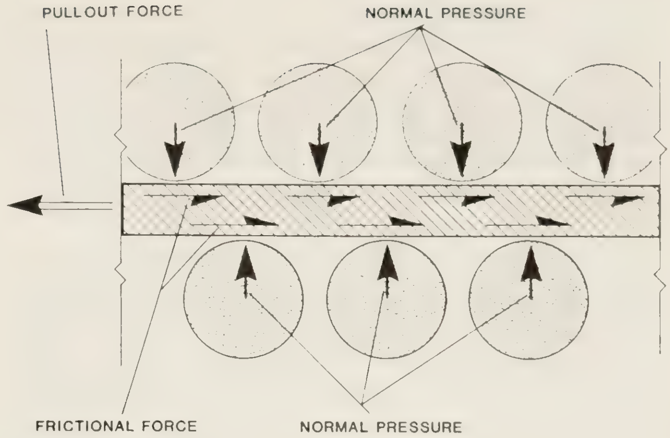


Figure 10.—Frictional transfer of load between reinforcements and soil.

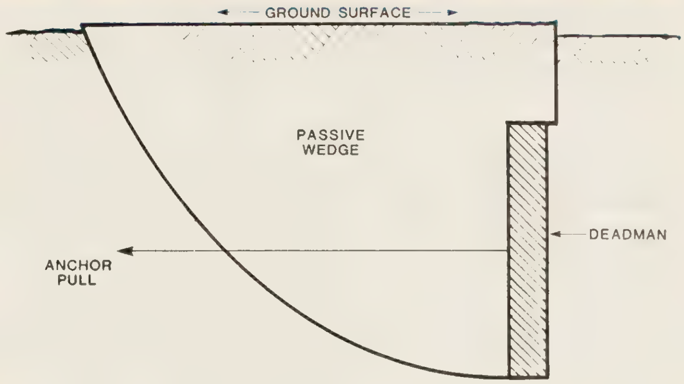


Figure 11.—Load transfer by passive soil resistance.

Most of the presently available soil reinforcement systems involve load transfer by both frictional and passive resistance (fig. 12). Frictional resistance is developed by the soil in contact with the flat surfaces of the ribbed strips used in Reinforced Earth (fig. 12a) and along the bars or wires used in several other systems (fig. 12b). Passive resistance is developed by the front faces of the ribs and transverse bars of the reinforcements shown.

The relative contributions of frictional resistance and passive resistance depend on the sizes and configuration of the reinforcements, on the soil type, on the soil overburden pressures, and on the strain in the system. For example, the experimental data in figure 13 show that ribs increase the apparent friction coefficient very significantly for low fill heights but that the effect decreases with depth. (7)

Little specific data about the relative contributions of the two kinds of resistance or the stress states in the soil between the reinforcements are available. Nevertheless, it is clear that the reinforcement restrains soil deformations and this in turn increases the strength and stability of the soil component of the composite material.

Two additional mechanisms—bending and shear resistance of the reinforcement—may increase overall slope stability when slope dowels or soil nailing are used for reinforcement.

Finally, the grid cell system appears to rely on the inextensible character of the cell walls to prevent lateral spreading of the contained soil, which increases the effective confining pressure resulting in greater soil strength and stiffness.

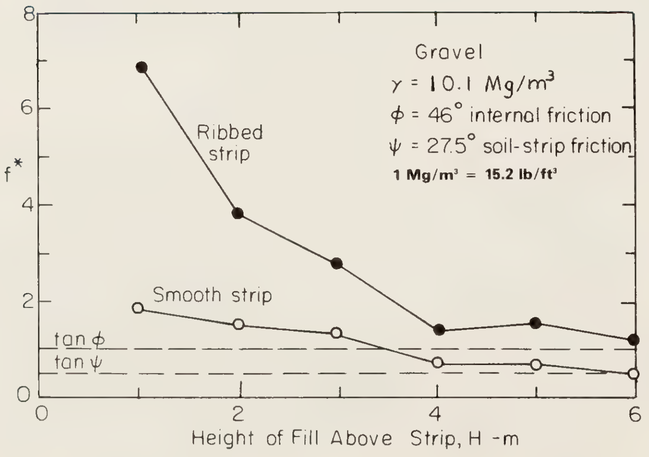
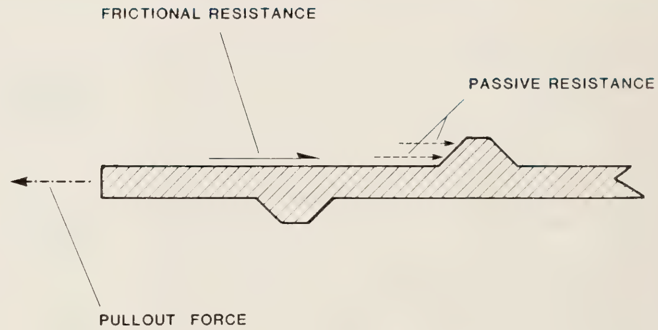


Figure 13.—The apparent coefficient of friction (f) for smooth and ribbed reinforcement strips for different fill heights. (7)*

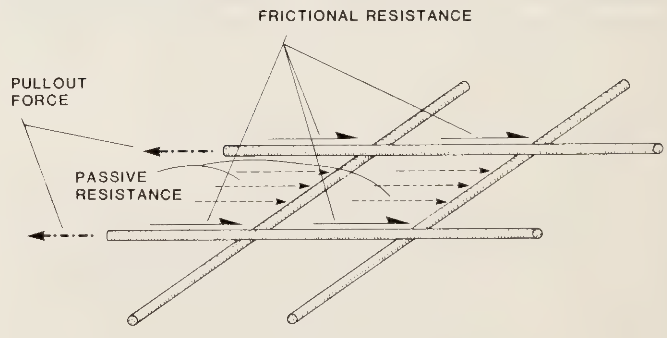
Reinforcement Systems

The use of soil reinforcement has increased rapidly over a relatively short time. As a consequence, several kinds of specialized systems—Reinforced Earth, Tensar Geogrid, VSL Retained Earth, and Hilfiker Welded Wire Wall—now are available as well as a variety of geotextiles that can be used in reinforcing applications. It can be expected that new systems and innovations will continue to appear. It is important, therefore, that the different kinds of soil reinforcement be distinguished to allow for classification of existing systems and also accommodate future systems.

The most suitable classification method would be based on reinforcement geometry or function rather than on reinforcement mechanism or proprietary system name. Mechanism is an unsatisfactory basis for distinction because the effectiveness of most reinforcement systems relies on more than one mechanism. Organizing information according to each specific system is equally unsatisfactory because it would obscure common features of the systems and make it difficult to classify future systems logically. Futhermore, such an approach would be impractical with geotextile reinforcements, because of



(a) Ribbed strips used in Reinforced Earth.



(b) Bar mesh system.

Figure 12.—Reinforcements that develop resistance by a combination of frictional and passive soil resistance.

the large number of materials and kinds available. Finally, a classification of reinforcement systems by function is not without problems because some systems can be used in a variety of applications. Therefore, the most suitable means for distinguishing between reinforcement systems is in terms of the geometry, and the following generic systems can be distinguished:

Fibers: Laboratory studies have shown that various fibers (for example, geotextile and fiberglass) have significant potential for soil reinforcement. However, because it is difficult to mix fibers into soil, fiber reinforcement has not yet been used widely in the field.

Strips: The original Vidal Reinforced Earth Wall developed in France used smooth steel strips. Most of these strips have been replaced by ribbed galvanized steel strips because of their greater pullout resistance. Other materials, including fiberglass, plastics, and composites, have been used to make strip reinforcements.

Sheets: Planar, flexible sheet reinforcement systems are exemplified by geotextiles.

Grids: Welded wire mesh, plastic grid mats (Tensar), and grids formed of interconnected concrete reinforcing bars are examples of grid systems, which are characterized by interconnected longitudinal and transverse elements.

Rods: Soil nailing commonly is done using round rods or dowels that are driven into the soil or grouted into prebored holes.

Cells: Soil reinforcement cells used in laboratory studies, field tests, and field applications have ranged up to a metre across. Materials used have included paper, plastic, aluminum sheet, and geotextiles.

Design Considerations

Reinforced soil slopes and embankments must be externally stable against sliding, overturning, excessive settlement, and foundation bearing failures. Classical methods of soil mechanics usually are suitable for an external stability analysis.

The internal design must insure against rupture of reinforcements, excessive slip or pullout of reinforcements, and loss of reinforcements by corrosion or other forms of deterioration. Analyzing the first two failure modes requires knowledge of soil-reinforcement interactions. As already noted, these interactions depend on soil type, reinforcement type and geometry, and the stress and deformation states of the soil. For reinforced soil walls, the horizontal stress at any depth, the effective friction coefficient between soil and reinforcement, and the stress distribution along the reinforcements must be known or estimated. Many analytical, numerical, model, and full-scale field experiments have been conducted to estimate these factors to provide information needed to select the design parameters.

The distributions of tensile stresses along the strip reinforcement for the Vidal Reinforced Earth Wall are shown in figure 14. The locus of maximum tensile stresses as a function of depth also is shown. This geometry is used with soil property and reinforcement data and suitable factors of safety to select the length, cross sectional area, and vertical and horizontal spacings of reinforcements.

A similar methodology should apply for other kinds of reinforcement. However, there may be differences in the stress distribution along the reinforcements and in the geometry of the active and resistance zones when sheet or grid reinforcements are used. More specific information about these differences is needed for some of the newer systems.

New design methods have been proposed and are beginning to appear for reinforcement of embankments over soft ground. Model tests, theoretical analyses, and field tests at various locations are developing suitable procedures for use with geotextiles and grid cells. One approach is to define a representative modulus for the composite reinforced zone and incorporate the zone as a stiffened layer that supports the embankment above and gives an improved stress distribution to the soft soil below.

A generally accepted method for the seismic design of reinforced soil walls and embankments has not yet been developed. However, field tests at the University of California, Los Angeles, showed that Reinforced Earth could withstand significant dynamic loads without distress.

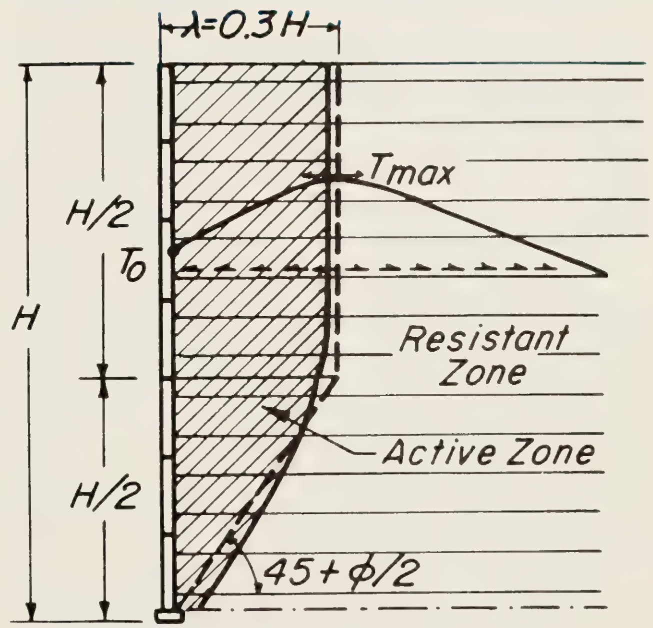


Figure 14. — Stress distribution along reinforcement and locus of maximum tensile stresses in a Reinforced Earth wall.

High-magnitude earthquakes have been considered in the design of walls in areas of known seismicity. The methodology used led to somewhat increased length of reinforcements and a higher density of reinforcements in the upper part of the walls. No internal failures or significant distress to existing reinforced soil slopes or embankments from earthquakes has been reported.

Currently, reinforcement durability is of great concern. The rate of corrosion of metal reinforcement depends on many factors, for example, local chemical concentrations and stray electrical currents, most of which cannot be controlled over the long term in the ground.

Galvanized steel has been used extensively for reinforcement. The byproducts of zinc corrosion cover the base metal and tend to seal affected areas. Corrosion loss over the life of a structure is assumed and added to the cross sectional areas required to carry the anticipated loads. Epoxy-coated steel reinforcements now available provide high durability without requiring additional metal to compensate for corrosion loss. Care is taken during handling and construction to minimize damage to or removal of the coating material.

The nonmetallic reinforcing materials—geotextiles, fiberglass, plastics, and composites—are not susceptible to corrosion but may be susceptible to other chemical and/or biological forms of deterioration. Because many of the materials are new, further study of durability is needed to determine the unknown effects of long term burial and exposure to the elements.

Precast concrete panels, sheet metal facings, geotextiles, and timber can be used for the facing elements of reinforced soil walls and abutments. As well as their esthetic function, the facings prevent loose soil from raveling between reinforcements and provide attachment points for the reinforcements during construction. The facings are designed to resist only small horizontal earth pressures.

Cost Considerations

The cost of soil reinforcement in constant dollars is less today than it was 8 to 10 years ago because the technology is maturing and the emergence of different reinforcement systems makes the market competitive. The total cost for any reinforcement system is composed of materials, construction, and backfill soil costs (if the onsite soil is unsuitable), and any special project features (for example, a special wall facing treatment or restricted access). The 1984 cost of materials for Reinforced Earth walls 10 to 15 ft (3.05 to 4.57 m) high is about \$15 per ft^2 (\$161 per m^2). For walls 15 to 30 ft (4.57 to 9.14 m) high, the cost is \$17 or \$18 per ft^2 (\$183 to \$194 per m^2). In areas where construction equipment is readily available, reinforced concrete may be more economical than reinforced soil for walls up to 10 ft (3.05 m) high. The two materials are competitive for the 10 to 30 ft (3.05 to 9.14 m) height range, and the reinforced soil wall is likely to be less expensive for heights greater than 30 ft (9.14 m).

Cost comparisons are more difficult for slope and embankment reinforcement because each situation is likely to be unique and there may be several options for each situation.

Research Needs

A comprehensive National Cooperative Highway Research Program (NCHRP) study and evaluation of available soil reinforcement techniques applicable to highway embankment and slope stability problems was recently completed. An accompanying report to be published soon will provide insight into existing soil reinforcement methods and their inherent advantages and shortcomings; describe soil reinforcement applicability to specific problems, the internal mechanisms of reinforcement, and the engineering behavior of reinforced soil structures; provide guidance for selecting an appropriate soil reinforcement technique; and provide a practical design methodology (with examples) and cost guidelines.

Also assembled and being documented is information regarding applications, mechanics and behavior, technology, durability and selection of backfill (or evaluation of in situ ground), construction, design methods, case histories, cost comparisons, future developments, and design examples for the following techniques: Anchored Earth, fabric (geotextile), Tensar Geogrid, plastic, Reinforced Earth, soil nailing in embankments, soil nailing in excavations, and Hilfiker Welded Wire Wall.

The NCHRP report should demystify soil reinforcement, increasing its use for highway construction applications. The results also will provide a benchmark for further research to develop improved design and construction procedures.

Although the results of the NCHRP study will document the state-of-the-art in soil reinforcement and should provide a basis for an up-to-date state-of-the-practice, uncertainties will remain and additional research will be required if safe, economical designs are to be assured. The long term durability of reinforcements and the behavior of reinforced soil systems under seismic loadings are such areas requiring additional research.

In recognition of other additional research needs, FHWA is conducting a study to develop comprehensive guidelines for using soil reinforcement techniques to support and/or strengthen retaining walls, bridge foundations, cut slopes, and roadway embankments. The NCHRP study results and other existing information will be reviewed and evaluated, design and construction guidelines will be developed, several reinforcement systems will be investigated in the laboratory and field, and an engineering manual of practice will be prepared.

The laboratory and full-scale field tests should clarify the following in relation to retaining walls, embankments, and spread footing foundation pads:

- Vertical and lateral stress distributions within the reinforced soil mass.

- Stress distributions from application of external loads.
- Vertical stress distribution beneath the reinforced soil mass.
- Appropriate lateral earth pressure coefficient.
- Influence of reinforcement geometry, spacing, orientation, reinforcement material types, epoxy coatings, and backfill soil types.
- Usefulness of reinforcement pullout tests and direct shear tests for evaluating soil-reinforcement interaction.
- Usefulness of models for predicting the behavior of full-scale structures.

The full-scale field tests will be conducted on five fully-instrumented retaining walls and one fully-instrumented embankment section. Three different retaining wall systems and three different backfill types will be used in various combinations to construct the five walls to provide a comparison among wall systems and backfill types. Each wall will be approximately 20 ft (6.1 m) high and 40 ft (12.2 m) long with widths sufficient to mobilize full active earth pressure at the back of the reinforced soil mass.

After gravity and earth pressure loads have been monitored, one wall will be selected for monitoring the effect of spread footing loads (equivalent to typical bridge abutment loads) on top of the retaining wall system. The placement of the footing will be varied along the wall to determine the effect of footing location. All five of the retaining walls also will be used to evaluate the effect of varying the slope of the surcharge above the wall. Two field pullout resistance tests will be conducted after the spread footing tests are completed.

The embankment test section will be 150 ft (45.7 m) long, 25 ft (7.6 m) high, and 20 ft (6.1 m) wide at the top and will be divided into three test sections with varying slopes. One generic system will be used to reinforce the embankment sections to evaluate the effect of reinforcement on steepness of slopes.

Successful completion of this study will reduce significantly the uncertainties about the design and use of some of the soil reinforcement systems now in use.

Soil reinforcement already has assumed a significant role in geotechnical engineering and earthwork construction. Because of the current technical and commercial interest and because soil is our most abundant and least expensive construction material, the development and application of soil reinforcement systems will continue for some time to come.

REFERENCES

(*1*) J. K. Mitchell and J. C. Collin, "Earth Walls," Proceedings of the National Academy of Engineering, 1983 Symposium on Cutting Edge Technologies, *National Academy Press*, Washington, D.C., 1984.

(*2*) H. Vidal, "The Principle of Reinforced Earth," Highway Research Record No. 282, *Highway Research Board*, Washington, D.C., 1969, pp. 1-16.

(*3*) C. Rea and J. K. Mitchell, "Sand Reinforcement Using Paper Grid Cells," Proceedings of the American Society of Civil Engineers Symposium on Earth Reinforcement, Pittsburgh, Penn., Apr. 27, 1978, pp. 644-663.

(*4*) J. K. Mitchell, T. C. Kao, and E. Kavazanjian, Jr., "Analysis of Grid Cell Reinforced Bases," Technical Report GL-79-8, *Geotechnical Engineering Laboratory, U.S. Army Engineer Waterways Experiment Station*, Vicksburg, Miss., July 1979.

(*5*) S. L. Webster and S. J. Alford, "Investigation of Construction Concepts for Pavements Across Soft Ground," Technical Report S-78-6, *U.S. Army Engineer Waterways Experiment Station*, Vicksburg, Miss., July 1978.

(*6*) J. K. Mitchell and F. Schlosser, "Mechanism, Behavior, and Design Methods for Earth Reinforcement," General Report, vol. 3, *Colleague International sur le Reinforcement des Sols, Ecole Nationale des Ponts et Chaussees*, Paris, France, 1979, pp. 25-74.

(*7*) I. Alimi et al., "Etude de l'Adherence Sols-armatures," vol. 1, Proceedings of the 9th International Conference on Soil Mechanics and Foundation Engineering, Tokyo, Japan, 1977, pp. 11-14.

James K. Mitchell is a professor and Chairman of Civil Engineering at the University of California in Berkeley. Over the past 26 years at Berkeley, Dr. Mitchell has conducted research in a variety of geotechnical engineering areas. He has authored or coauthored over 160 geotechnical publications, including several state-of-the-art reports on ground improvement, and often serves as general reporter or moderator at national and international conferences on ground improvement.

Willem C. B. Villet is a senior engineer and associate of Dames & Moore in San Francisco, California. He served as principal investigator on the NCHRP study discussed in this article. Dr. Villet has more than 10 years of diversified geotechnical experience and is a staff consultant on difficult foundation and ground improvement problems.

Albert F. DiMillio is a geotechnical research engineer in the Construction, Maintenance, and Environmental Design Division, Office of Engineering and Highway Operations Research and Development, FHWA. Mr. DiMillio is project manager for FCP Project 5P, "Foundations and Earth Structures." Prior to his present position, he served as an area engineer in FHWA's Indiana Division Office and as the Regional Geotechnical Specialist in Region 5, Homewood, Illinois.

Steel Corrosion in Concrete: pH at Corrosion Sites

by
Yash Paul Virmani, Walter R. Jones, and David H. Jones

Technical Editor's Note

This article should be read by everyone concerned with the bridge deck deterioration problem. Even if you have some difficulty with the chemical equations involved, clear and substantive evidence is presented to show that what seems obvious from observation is not always correct. Elimination of oxygen at the corroding upper layer of steel will not stop the corrosion process, provided that oxygen is available to other elements of reinforcement in the structure. The initial corrosion products formed are not oxides of iron, but more expansive compounds that subsequently are converted to oxides. The corrosion-inhibiting effect of the alkaline constituents in the cement paste is shown to be rapidly lost at local corrosion cells.

All of these findings have significant implications on existing approaches to preserving reinforced concrete structures in a chloride environment.

Introduction

The corrosion of reinforcing steel embedded in concrete is a widespread and serious problem in highway structures located in adverse environments where chloride ions from deicing materials and marine seawater permeate the concrete cover. These chloride ions, in the presence of oxygen and moisture, initiate and continue the corrosion process. This article discusses the chemical reactions that take place during corrosion of the rebars in chloride-contaminated concrete based on observation of the color and the pH of the corrosion products. The article also shows how information derived from pH testing can be used to develop chemical reactions that are taking place in a corrosion cell within a steel-reinforced, chloride-laden concrete member.

Corrosion sites of small magnitude and of little consequence by themselves are formed when a critical amount of chloride penetrates to the first level of steel reinforcement within concrete. This decreases the electrical half-cell potential of the steel at the corrosion sites to values more negative than −0.35 V compared with copper sulfate electrode, thus creating a large potential difference between rebar in the chloride-bearing area and rebar in chloride-free concrete (on both the top and bottom rebar mats). The magnitude of the potential difference is sufficient to overcome the losses in concrete and still allow significant current flow between large amounts of rebar in chloride-free concrete and the corrosion sites. These corrosion currents rapidly reduce the pH to values of 5 to 6 (by iron chloride complexing and hydrolysis, which release hydrogen and chloride ions) at locally corroding sites that are located on paths of least resistance to the macrocathode. This low anode pH is so different from that of nearby portions of the top mat bar also in chloride-bearing concrete (pH of 11.5 to 13.0) that a second powerful macrocathode (which also feeds the original anode spots) is created. The result is rapid, spotty corrosion followed by concrete cracking at total iron losses as low as 0.5 to 2.0 percent of the rebar volume. Further, only a small percentage (1/10 or less) of the total rebar surface area may be visibly corroded when initial cracking occurs.

The pH test is important in determining and confirming the kind of chemical reactions occurring in the concrete and steel system. Portland cement is a complex material containing numerous hydration products at various stages of development in combination with other chemical species. The complexes of iron that form in different environments of moisture, oxygen, and chloride at various stages of development are just as difficult to quantify. No simple chemical reaction equation can be stated. A procedure proposed by the Federal Highway Administration (FHWA) that includes a description of the equipment needed to perform pH testing was used to measure the pH at corroding sites in reinforced concrete beams that had been immersed partially in saturated sodium chloride solution for 5 years and cores containing reinforcing bars taken from various concrete corrosion specimens.

Experiment, Results, and Corrosion Mechanism

Measured pH on or near actively corroding rebar sites (anodes) is acidic (in the range of 4.8 to 6.0) and much lower than that previously hypothesized for a conventional iron to rust in concrete. Concurrent work using rebar anodes in oxygen-devoid limewater solutions showed that no corrosion occurred when limewater without chloride was used. With a 6-V driving voltage, the water underwent hydrolysis and liberated hydrogen gas at the cathode and oxygen gas at the anode. In companion tests, when very small quantities of sodium chloride were added to the limewater, chloride ions were discharged at the anode, and rapid corrosion occurred. White and greenish-yellow corrosion products literally spewed from the anode. The increase in volume of the corrosion products obviously was tremendous.

In this case, the probable anodic corrosion reaction was:

$$Fe^{0} + 2Cl^- \rightarrow (Fe^{++} + 2Cl^-) + 2e^-$$

followed by

$$Fe^{++}(2Cl^-) + 2H_2O \rightarrow Fe(OH)_2 + 2H^+ + 2Cl^- .$$

The chloride facilitates corrosion at the anode (without the presence of molecular oxygen) by iron chloride complexing and hydrolysis. The generated hydrogen ion undoubtedly is the cause of the low anode pH's measured in naturally corroding concrete specimens. The ferrous hydroxide formed by the iron chloride reaction is the same substance initially formed at an anode by the conventional atmospheric corrosion reaction. The iron chloride reaction is self-perpetuating in that the chloride is released for reuse when ferrous hydroxide is formed. White corrosion products of ferrous hydroxide, as well as yellowish-green to greenish-blue products, which may have been ferrous chloride and hydrated ferrous chloride, respectively, were seen in several instances when concrete cores containing corroding rebars were broken open and pH as low as 4.6 was measured. However, upon exposure to open air, these products changed within seconds to black corrosion products. In almost every instance when low pH's were measured, large amounts of black corrosion products also were present initially, indicating that at least small amounts of oxygen had been available. In these instances, the probable full corrosion reaction at the anode, after iron chloride complexing, was:

$$6(Fe^{++} + 2Cl^-) + O_2 + 6H_2O \rightarrow 2Fe_3O_4 + 12H^+ + 12Cl^- .$$

The Fe_3O_4 is a black granular corrosion product. Reddish-yellow brown rust rarely was found at active anode sites when the concrete was broken. Only after exposure in open air (for several hours to as long as 24 hours) did all the corrosion products convert to Fe_2O_3 (reddish-brown rust), probably by the following reaction:

$$4Fe_3O_4 + O_2 + 18H_2O \rightarrow 6Fe_2O_3 \; 3H_2O.$$

The pH measurements at the cathode (noncorroding) sites in reinforced beams and slabs typically were in the range of 11.5 to 13.0, regardless of the chloride content of the concrete.

Conventional theory predicts the primary cathodic reaction in concrete to be oxygen reduction ($1/2\ O_2 + H_2O + 2e^- \rightarrow 2OH^-$). The cathodic sites typically were free of corrosion without even small rust pits, although the rebar had been surrounded for up to 7 years by concrete containing large amounts of chloride. The continuous liberation of hydroxyl ions during oxygen reduction maintains a high pH level.

Thus, based on the above reactions, a maximum of only one-third as much oxygen typically is consumed at a rapidly corroding anode site in chloride-contaminated concrete than at the cathode site. Even if the molecular oxygen is removed at the anode, rapid corrosion can continue through iron chloride complexing and hydrolysis provided that oxygen gas for reduction remains available at the cathode. $FeCl_2$ and $FeCl_3$ are much more expansive than $Fe(OH)_2$, Fe_3O_4, or Fe_2O_3, which further aggravates corrosion-induced concrete cracking.[1] Smaller amounts of parent iron must be consumed for a specific tensile stress to develop.

A paragraph in a 1944 *Smithsonian Institution Bulletin* best describes the problem and challenge facing engineers attempting to combat iron chloride complexing in a rigid material:

> ". . . and the ferrous chloride lawrencite ($FeCl_2$), which is reported to have been observed in the solid state but which is deliquescent and usually manifests itself in an exudation of clear greenish drops.
>
> "Lawrencite is not a factor in the structure of meteoric irons except by reason of its destructive action in causing rust and disintegration. By the action of the air it turns to ferric chloride ($FeCl_3$) and ferric hydroxide. The ferric chloride in contact with the still unaltered iron is reduced again to the ferrous chloride. Thus the process is continuous, and as it is accompanied by an increase in volume the iron may become wholly altered and disintegrated unless it is kept under oil. Lawrencite is the bane of curators and collectors. If an iron definitely contains lawrencite, no amount of lacquering seems to protect it; the only recourse is immersion in oil." (*1*) [2]

Based on controlled laboratory experiments, pH measurements, and the observed reaction products, a simplified rebar corrosion mechanism is given below. The following chemical reactions are proposed at the surface of the steel in the presence of chloride ion but no oxygen and in the presence of oxygen.

Corrosion of steel in concrete in the presence of chloride ion but no oxygen:

1. Iron reacts with chloride ions to form an intermediate iron chloride complex at the anode (in the upper layer of steel in a bridge deck).

$$Fe^0 + 2Cl^- \rightarrow (Fe^{++} + 2Cl^-) + 2e$$

2. This complex reacts with moisture to form ferrous hydroxide.

$$(Fe^{++} + 2Cl^-) + 2e + 2H_2O \rightarrow Fe(OH)_2 + 2H^+ + 2Cl^-$$

3. The liberated hydrogen ions combine with electrons to form hydrogen gas at the cathode.

$$2H^+ + 2e \rightarrow H_2 \uparrow$$

Corrosion of steel in concrete in the presence of oxygen:

1. The iron is oxidized to the ferrous state, releasing electrons at the anode.

$$Fe^0 \rightarrow Fe^{++} + 2e$$

2. These electrons combine with oxygen and moisture at the cathode to form hydroxyl ions at both the upper and lower layers of steel.

$$2e + H_2O + \tfrac{1}{2} O_2 \rightarrow 2OH^-$$

3. The ferrous ions combine with hydroxyl ions to again produce ferrous hydroxide, as above.

$$Fe^{++} + 2OH^- \rightarrow Fe(OH)_2$$

Ferrous hydroxide is further oxidized in the presence of moisture to form Fe_2O_3.

$$4Fe(OH)_2 + 2H_2O + O_2 \rightarrow 4Fe(OH)_3$$
$$2Fe(OH)_3 \rightarrow Fe_2O_3 + 3H_2O$$

The significance of these chemical reactions includes the following:

- The expansive corrosion products cause high tensile stresses and can crack the concrete. These cracks would allow chloride ions to enter and cause more iron chloride complexing.
- The corrosion process can be slowed if the cathodic reaction rate can be reduced by cutting the supply of oxygen at the cathodic sites (mostly at the lower level of the steel in a bridge deck).
- The initiation of corrosion can be prohibited by preventing sufficient chlorides from entering the concrete.

pH Measurement Procedure

Corrosion processes can be followed by observing the colors of the products and by measuring the pH of the residue. However, the pH is difficult to measure because of the following:

- The small localized areas on the curved surface of deformed reinforcing bars or their imprint.
- The extremely thin layers of the corrosion products that can be overwhelmed by the effect of the underlying media (steel or concrete).
- The dynamic situation of product conversion when the corrosion cell is opened to the atmosphere.

---

[1]Piling-Bedworth expansion ratios for $FeCl_2$, $FeCl_3$, $Fe(OH)_2$, and Fe_2O_3 are reported by the literature to be 5.98, 8.14, 3.72, and 2.14, respectively. A ratio of 1.0 would indicate no expansion.

[2]Italic number in parentheses identifies reference on page 101.

During this investigation, it was found necessary to define a procedure for measuring pH that provided reproducible results. The procedure describes locating the steel, lifting the concrete cores, processing, exposing the steel, and measuring the pH. A concrete core without rebars was broken vertically into halves to compare three methods for measuring pH—color indicating solutions, pH indicating paper tabs, and pH meter with probe (electrode). The core, exhibiting no corrosion products, was chosen so pH could be determined on large uniform surface areas. A 0.75-in (19-mm) diameter area was used for determining the pH by the three methods. pH was not determined on exactly the same spot but within the chosen 0.75-in (19-mm) diameter, and the results are given in table 1. It appears that the above methods are adequate for determining pH in most instances.

Table 2 illustrates the change in pH value with time measured at three areas with the probe and pH meter on a split core with corrosion. The pH data showed that the probe method was more accurate than the other two but the other two methods complement each other and under certain conditions one or the other may be easier to apply.

The galvanic corrosion in concrete is a process whereby both corroding anodes and noncorroding cathodes often are present side-by-side. The corrosion products at anodes on corroded rebars typically are colored from black, to green, to yellow, to reddish-brown. All of the above colored products may not be present in every specimen. When present, these colored corrosion products often migrate or diffuse from the immediate area around the rebar, which imparts color to the concrete.

Descriptions of corrosion products and pH measurements taken by FHWA on reinforced beams and cores from reinforced slabs are presented in tables 2-7 and in figures 1 and 2. In figure 1 the core,

Table 1.—pH of the hardened concrete as measured by three methods

Position	pH Probe	pH Indicator paper	pH Indicator solution
First area	12.8	12.8 to 13.0	≥ 13.0
Second area	13.0	12.3 to 12.8	≥ 13.0
Third area (on limestone)	12.0	11.8 to 12.1	10.0 to 12.2

Table 2.—Change in pH value with time as measured with probe on the split concrete core

Time	pH First area	pH Second area	pH Third area
Minutes			
1	5.7	5.7	6.1
2	9.4	5.6	
3	10.2	5.5	6.1
4	9.7	5.5	6.0
5	9.6	—	—
6	9.4	—	—
7	8.9	5.5	—
9	10.5	—	—
10	10.0	6.1	6.0
20	—	5.6	—

Additional water added.

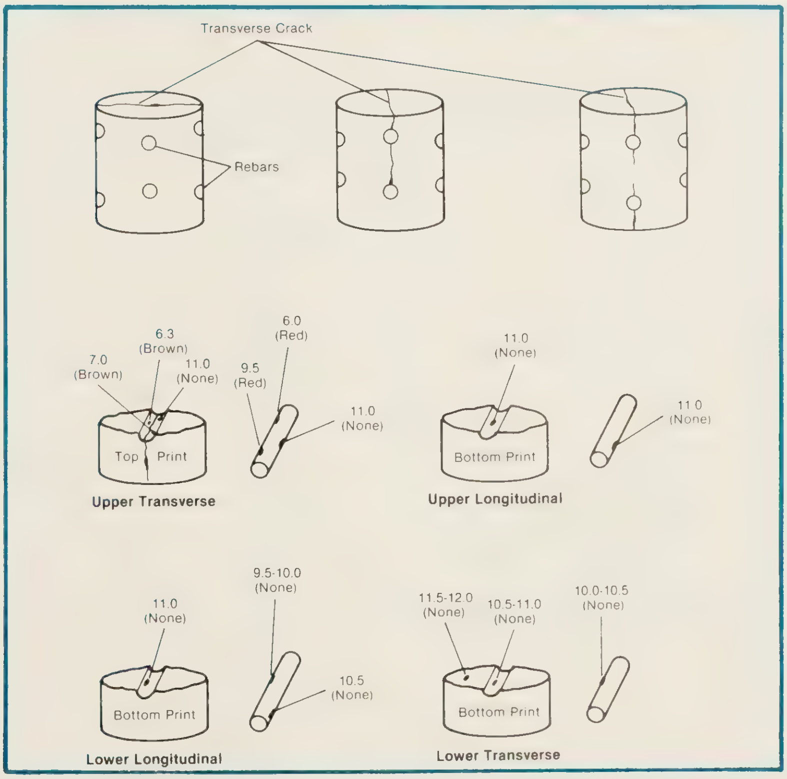


Figure 1.—Locations of the pH measurements on the split 6-in (152-mm) core extracted from the bridge deck section. The pH value and color are indicated.

extracted from a reinforced concrete section of a deteriorated bridge deck, was located in a high electrical half-cell potential area where the transverse and longitudinal rebars crossed each other and had a transverse crack. The pH was measured at various positions in the concrete rebar interface and on the rebar in the split core. In figure 2 the concrete beam (partially immersed in saturated sodium chloride solution for 58 months) contained one longitudinal reinforcing bar and had cracked because of corrosion of the reinforcing steel. The pH was measured at various positions in the concrete rebar interface and on the rebar in the split beam.

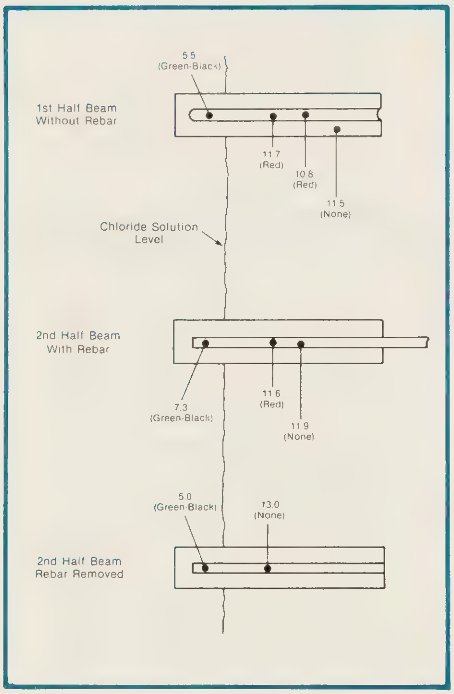


Figure 2. — Locations of the pH measurements on the split concrete beam (3 in x 4 in x 16 in [76 mm x 102 mm x 406 mm]) immersed in saturated sodium chloride solution. The pH value and color are indicated.

Table 3.—Measured pH values on a split concrete core [1]

Position	pH Probe	pH Indicator solution	pH Indicator paper	Description of corrosion product
On concrete away from rebar	10.0	10.2 to 12.2	10	None
On concrete away from rebar (opposite face)	9.9	10.2 to 12.2	10.5	None
On top of rebar	5.6	6.0	6	Black-green
On concrete (rebar reflection)	5.0	5.2 to 6.0	5.5	—
On bottom of rebar	6.0	4.2 to 4.8	6	Black-green
On concrete (rebar reflection)	5.5	5.2 to 6.0	5	Black-green

[1]This 4-in (102-mm) core was taken from a chloride-contaminated reinforced concrete slab through a light surface crack that had corrosion-induced delamination. Upon splitting, the rebar was almost totally black with large patches of aquamarine color products that turned pure green in 30 seconds and brown in about 30 minutes. After 24 hours, all corrosion products on the bar and in the concrete were reddish-brown. The pH was measured at six positions in the split core within 1 hour after splitting.

Table 4.—Measured pH values on split concrete cores [1]

Core No.	Position	pH Probe	pH Indicator solution	pH Indicator paper	Description of corrosion product
1	On upper rebar	5.2	4.2 to 4.8	5	Black
	On concrete next to rebar	4.6	4.2 to 4.8	5	Black with some brown
	On lower rebar	7.9	6.6 to 8.2	7	Red
	On concrete away from rebar; delaminated surface	11.5	10.2 to 12.2	12	Yellow
	On concrete away from stained surface	11.0	10.2 to 12.2	12	None
2[2]	On upper rebar	5.2	4.2 to 4.8	4.5 to 5	Black
	On concrete adjacent to rebar	11.3	12.2 to 13.2	12	Red
	On concrete-delaminated surface	3.9	4.2 to 4.8	4.5	Aquamarine, turned green in 15 seconds
	On sound concrete	11.5	12.2 to 13.2	12	None
3	On concrete under rebar imprint (top surface)	12.2	12.2 to 13.2	12	None
	On sound concrete	11.3	12.2 to 13.2	12	None
	On concrete under rebar imprint (bottom surface)	12.2	10.2 to 12.2	12	None

Table 4 continued—

Three 4-in (102-mm) cores were taken from a different chloride-contaminated reinforced slab. Cores 1 and 2 were taken 3 ft (0.91 m) apart in the slab. They were cracked and delaminated because of corrosion and contained the intersection of crossing bars in the reinforcing mat. Core 3 was taken 10 in (254 mm) away from the second core along the longitudinal bar. It was uncracked and had no visible corrosion on the rebar. pH measurements with the three methods were taken at various positions in the split cores.

Small deposits of white corrosion product were visible under the bar immediately upon splitting. These turned green within 15 seconds.

Table 5.—Measured pH values on the concrete beam cracked by impressed current [1]

Position	pH Indicator solution	pH Indicator paper	Description of corrosion product
On concrete, top imprint of probe	—	6	Black; turned green in 30 seconds
On concrete, top imprint of probe	6.6 to 8.2	5.5 to 6.5	Black-green-dark brown
On concrete, top imprint of probe	6.0 to 6.6	6	Black
On concrete, bottom imprint of probe	6.0 to 6.6	5.5 to 6	Black
On concrete, bottom imprint of probe	6.0 to 6.6	5.5 to 6.5	Black-green-dark brown
On concrete, adhering to probe	—	6 to 6.5	Yellow
On concrete, imprint of copper rod	10.2 to 12.2	12	None

[1] A beam (217 days old in the fog room) containing a rate-of-corrosion probe and a copper rod (cathode) purposely was cracked in 6 days with an impressed current of 14 mA at 6 V. The concrete contained 8 lb Cl^-/yd^3 (4.7 kg Cl^-/m^3) from admixed sodium chloride. The pH was measured at seven positions in the split beam.

Table 6.—Data on various concrete beams mortared to reinforced concrete slabs [1]

Beam No.	Days in fog room before mortared to slab	Cl in slab	Days on slab: Until cracking	Days on slab: Total until splitting for pH	Total current amp hours: Until cracking	Total current amp hours: Total until splitting for pH
278	175	None	67	144	0.66	1.42
243	188	None	60	141	0.87	1.82
303	3	None	105	105	1.21	1.21
283	176	None	No cracks	139	N/A [2]	0.00
282	193	Yes	No cracks	137	N/A [2]	0.41
281	181	Yes	No cracks	121	N/A [2]	0.60
301	12	Yes	> 75 but < 111	111	— [3]	1.76
300	12	Yes	45	45	0.94	0.94

[1] Several concrete beams containing a rate-of-corrosion probe alone or in conjunction with another metal (such as copper tube copper plate, stainless steel plate, or magnesium plate) were cast. All of the beams except beam 283 contained 33 lb Cl^-/yd^3 (19.6 kg Cl^-/m^3) of concrete admixed at the same time of mixing. These beams were mortared to reinforced concrete slabs and exposed to outdoor environment. No more than two beams were mortared at any time to a single reinforced slab. The corrosometer probe in the beam (anode) and the iron rebars (cathode) in the upper mat of the slab were electrically coupled with a separation distance of 2 3/8 in (60.2 mm). (The significance of this unusual test was to demonstrate that concrete brought to uniform moisture conditions in a fog room exhibited no corrosion currents, but when the beams were attached to the slab surface and the rebar steel electrically connected, large currents flowed with the slab steel acting as a large cathode to intensify the corrosion of the small amount of anode steel in the beams.)

[2] Rate of corrosion of probes during fog-room storage was 0 for all beams.

[3] Beam was cracked when removed from slab, but exact data are unknown.

Summary

Laboratory tests confirm that many different corrosion products exist at the concrete-steel interface at a corroding anode. When concrete was split open in room atmosphere, colors and pH's changed rapidly from black/aquamarine, to green, to brown, and finally stabilized at reddish-brown. Each color indicated a change in corrosion product chemistry.

A tentative procedure is proposed for standardizing the three methods evaluated for measuring the pH of the reaction sites on corroding steel rebars. The pH values may be used in conjunction with modeling or simulation of the chemical reactions, which are taking place, to identify and verify the hypothesized reactions.

REFERENCE

(*1*) Stuart H. Perry, "The Metallography of Meteoric Iron," Smithsonian Institution Bulletin No. 184, Washington, D.C., 1944.

Table 7.—Measured pH values on various cracked (split) concrete beams caused by the current flow from dissimilar concentration of chloride ions in the mortared concrete beam and the slab [1]

Beam No.	Position	pH	Description of corrosion product
278	Top of probe	6.0	Black
	Bottom of probe	11.0 to 11.5	None
	On concrete, bottom probe imprint	12.0	None
243	Top of probe	5.5 to 6.0	Black
	On concrete, bottom probe imprint	12.0	None
	Bottom of probe near tip end	5.5 to 6.0	Black
	Around probe at enclosed end	5.5 to 6.0	Green
	On concrete, adjacent to probe	5.5	Black
303	Top of probe	5.5 to 6.0	Black
	Bottom of probe	12.0	None
	On concrete, bottom probe imprint	12.0 to 12.5	None
	On concrete, top probe imprint	6.5 to 7.0	Black-red
283	Measurement at three locations	11.0 to 11.5	All corrosion free
282	Top of probe	6.0	Black
	Bottom of probe near tip end	11.5 to 12.0	None
	On concrete, top probe imprint	9.0 to 9.5	Black
281	On concrete, bottom probe imprint	12.0 to 12.5	None
	Bottom of probe near tip end	10.5	Red
	Top of probe	5.5 to 6.0	Black
	On concrete, top probe imprint	5.5 to 6.0	Black
301	Bottom of probe	5.0 to 5.5	Black
	On concrete, bottom probe imprint	12.5	None
	Around probe at enclosed end	5.5	Black
	On concrete, bottom probe imprint	5.5	Black
300	Top of probe	5.3 to 5.5	Black
	Bottom of probe	11.5 to 12.0	None
	On concrete, top probe imprint	5.9 to 6.2	Black
	Top of probe near tip end	5.3 to 5.7	Black
	Concrete adjacent to probe, bottom half of beam	11.5 to 12.0	None

[1] At the conclusion of the experiment, all the mortared beams, whether uncracked or cracked, were removed from the slabs and split open. The pH was measured at various positions around the probe and concrete in the probe's immediate vicinity.

Acknowledgment

Thanks are extended to Mr. Kenneth C. Clear of Kenneth C. Clear, Inc., formerly with FHWA, who initiated the study of measuring pH in chloride-contaminated reinforced concrete specimens.

Yash Paul Virmani is a research chemist in the Materials Technology and Chemistry Division in FHWA's Office of Engineering and Highway Operations Research and Development. Dr. Virmani is task manager for the Federally Coordinated Project 5Q, Task 2, "Cost Effective Rigid Concrete Construction and Rehabilitation in Adverse Environments." He is a co-inventor of conductive polymer concrete, a new material that is the basis of several new cathodic protection systems.

Walter R. Jones is a supervisory civil engineer in the Materials Technology and Chemistry Division. He has over 20 years of experience with FHWA in conducting laboratory studies of concrete and corrosion in concrete.

David H. Jones is a civil engineering technician in the Materials Technology and Chemistry Division. He performed many of the measurements needed to develop confidence in the procedures reported.

Safety Impacts of RRR Projects

by
Julie Anna Cirillo

Introduction

In 1976, Congress, reacting to widespread concern over the deteriorating condition of the Nation's highway system, authorized the use of Federal funds for resurfacing, restoration, and rehabilitation (RRR) projects on existing Federal-aid system highways.[1] Until that time Federal funding for highways had been limited to construction of new Federal-aid highways, and financing of maintenance work was the responsibility of the States.

RRR activities generally fall into two classes—repairs of the road surface (such as resurfacing), reflecting highway deterioration concerns, and improvements in highway geometric characteristics (such as lane and shoulder widening and alinement changes), reflecting safety concerns. More extensive improvements of existing facilities are considered to be "reconstruction" instead of RRR projects.

The Federal-aid highway system constitutes only 20 percent of the Nation's highway mileage but handles 80 percent of all highway travel. Excluding Interstates, which have their own RRR program, the Federal-aid system consists of three administrative systems—primary, secondary, and urban—for which funding is provided on a 75 percent Federal/25 percent State or local matching basis. For 1983, RRR spending from State and Federal sources, excluding Interstate RRR, has been estimated at $2 billion annually. Characteristics of the three administrative systems are summarized in table 1.

Of the three systems, the primary system accounts for 49 percent of vehicle-miles (vehicle-kilometres) traveled and receives about 60 percent of non-Interstate Federal-aid funding of which at least 40 percent must be spent on RRR and reconstruction projects. Because the system is overwhelmingly rural and consists predominately of two-lane roads, considerable attention in the RRR program has been directed at two-lane rural roads.

Table 1.—Non-Interstate Federal-aid system (1981)

Administrative system	Mileage	Location	Functional system	Annual vehicle-miles	Two-lane highways	Divided highways	Lanes ≤10 ft
	Thousands			*Billions*	*Percent*	*Percent*	*Percent*
Primary	225	89% rural	Arterial	457	82	13	16
Secondary	401	100% rural	Collector	143	99	0.5	33
Urban	129	100% urban	58% arterial 42% collector	327	76	10	35

1 mile = 1.609 km
1 ft = 0.305 m

[1]Historical information extracted from documents prepared by staff of the National Academy of Sciences.

Selecting RRR Design Standards

Interim measure

When RRR projects originally became eligible for Federal-aid in 1976, the Federal Highway Administration (FHWA) had not developed RRR design standards. As an interim measure, new construction standards were applied to RRR projects. Exceptions, permitted on a case-by-case basis, became commonplace for RRR projects because upgrading to the geometric standards for new highways often was extraordinarily expensive. In Northeastern States, where highway systems are relatively old and the topography is severe, 75 to 90 percent of RRR projects received exceptions.

Alternative policies

This interim measure lasted far longer than expected because selecting RRR geometric design standards proved to be complex and controversial. The tradeoffs among highway safety, geometric standards, and the need to use Federal RRR funds in the most cost-effective manner were at the center of the controversy, which was fueled by the different perspectives of safety and State highway organizations. FHWA considered several alternative policies including the following:

1. Continuing to use new construction geometric design standards with exceptions permitted on a case-by-case basis.

2. Adopting RRR standards developed by the American Association of State Highway and Transportation Officials (AASHTO). In November 1976, shortly after the initiation of the Federal RRR program, AASHTO recommended geometric standards for RRR projects. Referred to as the "Purple Book," the AASHTO standards are quantitative design *guidelines* concerning pavement and shoulder widths, cross slopes, superelevation, bridge width, and clear zones.

3. Adopting RRR standards developed internally by FHWA and proposed in August 1978. In general, the FHWA standards were more stringent than those in the Purple Book.

4. Adopting a flexible approach under which States could develop and use their own RRR standards subject to FHWA approval.

The first alternative was supported by safety-oriented organizations, such as the Center for Auto Safety, the Insurance Institute for Highway Safety, and the National Transportation Safety Board, who argued that any standards less stringent than those applied to new construction would be detrimental to safety. Despite the large number of exceptions granted by FHWA, this alternative nevertheless required explicit consideration of exceptions to geometric standards and it occasionally stimulated substantial geometric upgrading. Although safety organizations acknowledged the need for exceptions, both the AASHTO Purple Book standards and FHWA-proposed standards were viewed as being far too lenient, permitting the RRR program to focus almost exclusively on road surface improvements and foreclosing the possibility of additional safety requirements. In addition, some argued that reductions in standards for federally assisted RRR projects would violate legislative mandates concerning safety. The fourth alternative, permitting States to develop their own standards, also was not popular with the safety groups who feared that States would opt for, and FHWA would approve, standards similar to those in the AASHTO Purple Book.

State highway organizations, on the other hand, initially supported the AASHTO Purple Book standards but later indicated a general willingness to accept the more stringent RRR standards proposed by FHWA. New construction standards were viewed by the States as inappropriate for RRR projects. If followed rigorously, the standards would increase project costs greatly, thereby requiring available funds to be concentrated on a smaller number of projects. Such a policy, the States argued, would leave the many miles (kilometres) of Federal-aid highways badly in need of pavement repair unattended, meeting neither safety nor repair objectives. On the other hand, if all projects required exceptions, needless administrative costs and delays would be incurred.

Safety concerns

Throughout the debate and discussion of RRR standards, safety was the key issue. Most concern centered on the effect resurfacing only would have on accident rates, a concern probably evolving from two studies commonly referred to as the Arkansas study (*1*) [2] and the British study. (*2*) The conclusions in the Arkansas study indicate that because resurfacing improves ride quality, drivers are provided with a false sense of security, resulting in increased operating speeds and therefore increased accidents as drivers have insufficient time to adjust to critical situations. Although these conclusions provide insight into the observed difference in accident rates, no data are provided to substantiate the conclusions. For example, no speed information is provided nor are sample size and the effect of low volumes on accident rate discussed.

The British study and a recent Swedish study (*3*) appear to lend credibility to the contention that resurfacing increases speed. After resurfacing, typical speed increases of 6 mph (9.7 km/h) are reported in the British study and average speed increases of 2.5 mph (4.0 km/h) are reported in the Swedish study. However, close inspection of these studies reveals that the data were taken from highway sections that previously were gravel and then improved to hard surface.

Other concerns expressed during the RRR standards debate were the nationwide consequences of alternative RRR standards on safety and highway condition when budgetary resources are limited. Although these questions cannot be answered fully, FHWA attempted to provide insights into these problems by conducting an analysis in conjunction with the rulemaking process. The results of

[2] Italic numbers in parentheses identify references on page 107.

this analysis, published as part of the rulemaking process, concluded that improvements less stringent than standards for new construction would be most appropriate for RRR projects. (*4*)

FHWA analysis and RRR report

The purpose of the FHWA analysis was to determine the extent to which rural arterial and collector systems could be affected by RRR improvements. Before the analysis, it was unclear as to the extent of mileage that would be affected, the cost involved, or the maximum safety impacts expected. The methodology selected permitted the use of a 284,208-mile (457 388-km) inventory of highways, alternative funding levels, and three different RRR improvement concepts. The results of the analysis have provided numerous findings directly related to the real-world RRR implementation concerns and policy decisions.

The FHWA RRR report based on this analysis was criticized heavily by the National Transportation Safety Board (NTSB) for possible methodological shortcomings. (*5*) In addition, the NTSB asserted that about one-third of the roads analyzed in the FHWA RRR report are not eligible for Federal-aid RRR projects, and thus the relative safety and cost impact estimates are distorted. This NTSB assertion is incorrect. The inventory used in the study did include the minor collector system highways not usually on the Federal-aid system. These highways were included, however, to obtain the overall National aspects of RRR problems because States cannot ignore such highways. In reevaluating the data in the FHWA report and excluding the minor collector system of highways, the ratio of total savings to construction costs changed very little.

The NTSB asserted that the FHWA RRR report did not use a good approach for computing the maximum potential fatality reductions using a common shoulder improvement width for the lower functional system of highways. The NTSB preferred using narrower shoulder widths for computing the safety impacts, but made no specific shoulder width recommendations.

To address this concern, different shoulder widths for the lower functional systems were established (table 2), and then projected reductions in fatalities were reestimated (table 3).

As can be seen from table 3, the revised shoulder width assumptions do not change the conclusion that case 2 and midcase potential fatality reductions exceed those of case 1. Also, this subanalysis does not account for the additional mileage that could be improved because of less costly shoulder width improvements.[3]

Additional concern was expressed over the use of resurfacing data that was collected in the mid-1970's. The same organizations willing to accept the Arkansas study criticized the resurfacing data even though the data represented 59 highway sections versus 6 sections for the Arkansas study, 406 miles (653 km) versus 48 miles (77 km) for the Arkansas study, and represented data from eight States versus one State. In addition, the resurfacing data were substantiated further by additional data collected and analyzed by Alabama.

To alleviate these concerns, several statistical analyses and a sensitivity analysis were performed on the 1970 resurfacing data. Each failed to provide convincing evidence that resurfacing only resulted in significant increased accidents.

Table 2.—Hypothesized shoulder widths for resurfacing and shoulder improvements

Functional system	Case	Shoulder widths: Before RRR report	To RRR report	Before subanalysis	To subanalysis
		Feet	*Feet*	*Feet*	*Feet*
Other principal arterial	1	4	12	Same	Same
	Midcase & 2	0	8	Same	Same
Minor arterial	1 & midcase	2	10	Same	Same
	2	0	10	0	3
Major collector	1, midcase, & 2	0	8	0	3
Minor collector	1, midcase, & 2	0	8	0	3

Case 1 represents geometric standards for new construction. Case 2 represents geometric standards consistent with the AASHTO Purple Book. Midcase represents geometric standards between new construction and Purple Book levels.

1 ft = 0.305 m

Table 3.—Potential fatality reductions assuming reduced shoulder widths for resurfacing and shoulder improvements

Case	Investment level	Fatality reductions: RRR report	With reduced shoulder widths on resurfacing and shoulder improvements
1	High	14,484	14,368
Midcase	High	17,205	15,729
2	High	17,207	16,161
1	Low	10,247	10,154
Midcase	Low	11,506	10,270
2	Low	11,731	10,968

Based on the following assumptions for resurfacing and shoulder improvements: Case 2 minor arterials shoulders were widened from 0 to 3 ft and not 0 to 10 ft, and case 1, midcase, and case 2 major and minor collectors shoulders were widened from 0 to 3 ft and not 0 to 8 ft. These assumptions do not account for additional mileage that could be improved because of less costly shoulder width improvements.

1 ft = 0.305 m

[3] S. C. Tignor, "Review of NTSB Report, 'Safety Effectiveness Evaluation,' Relative to FHWA RRR Technical Report," Office of Research, Federal Highway Administration, 1981. Unpublished.

Although the FHWA analysis was not perfect, no other comprehensive assessment of RRR has been undertaken, and the work does logically assess various alternatives. The results indicate that large-scale implementation of relatively minor improvements (roadway and shoulder widening) and their associated safety benefits over large amounts of mileage are more cost-effective than completely upgrading a few highway sections to new construction standards.

Finally, in June 1982 FHWA settled on the fourth approach, permitting the States to develop their own RRR standards subject to FHWA approval.

1982 Surface Transportation Assistance Act

In the fall of 1981, the U.S. Congress House Subcommittee on Investigations and Oversights of the Committee on Public Works and Transportation held hearings on the RRR standards. This activity was coupled with Government Accounting Office reviews and several FHWA activities including State reviews and accelerated research activities. The main issue debated was the role that safety should play as an objective of the RRR program. FHWA adopted the position that safety was an essential consideration of the RRR program along with preserving and extending the service life of highways. Safety groups, on the other hand, argued that safety was the primary objective of all Federal-aid highway programs including RRR. The hearings led to a provision in the Surface Transportation Assistance Act (STAA) of 1982 that stated the objective of the RRR program as being ". . . to preserve and extend the service life of highways and *enhance* highway safety." (*6*) Nevertheless, Congressional deliberations show substantial ambiguity about how much of a change, if any, this provision required. Subsequently, FHWA modified its June 1982 rule on RRR standards by adding the words "enhance highway safety" throughout to reflect the STAA legislative change.

In addition, the STAA provides for a study to be conducted by the National Academy of Sciences (NAS) to develop geometric criteria for RRR projects. The NAS currently is undertaking this activity with the assistance of a study panel representing all the interested parties. Of main concern is the relationship between geometrics and accidents. Past work will be examined and some new research will be undertaken.

Summary and Recommendations

Unfortunately, the highway community, including the safety interest groups, is reluctant to come to grips with the limitations of accident analyses and to use instead available information to establish policy and assist in decisionmaking.

Demands for accident studies to identify changes in accidents for small incremental changes in geometric criteria are misplaced and misinformed. Claims that existing data can be used to undertake such studies reflect a lack of understanding of existing data and the requirements for such an undertaking. The response variable (accidents or accident rate) is too gross a measure to detect small differences in a geometric design element (for example, a 1-ft [0.305-m] increase in shoulder width).

Accidents are extremely rare, requiring vast amounts of highway mileage with similar characteristics for study to obtain adequate sample size. Highway sections selected for study must remain constant for the entire study period (often 5 years or more) except for the element being examined. The cost for collecting and analyzing data including site selection often is prohibitive considering the low probability of detecting small differences.

To resolve this conflict, the following are recommended:

1. Accumulate and analyze existing accident research results.

2. Provide information in a format useful to field operations.

3. Undertake research in selected high-payoff areas.

4. Provide guidance on interpretation of safety data to assist decisionmaking on project improvements.

Efforts to accomplish the literature review and analysis include two editions of the Highway Users Federation for Safety and Mobility publication "Traffic Control and Roadway Elements—Their Relationship to Highway Safety" (1963 and 1968) and the recent FHWA publication "Synthesis of Safety Research Related to Traffic Control and Roadway Elements" (December 1982). Unfortunately, these publications suffer in varying degrees from a lack of critical assessment of the research reported and instead present ranges of results from a variety of sources. Thus, the user is required to critically assess the information presented. Additionally, because the synthesis is an FHWA report, its credibility is immediately questioned by various segments of the highway community.

In the interest of objectivity, under the provision of STAA, NAS is critically analyzing existing information in 10 critical areas: Lane width, shoulder width, shoulder type, horizontal alinement, vertical alinement, roadside, sight distance, intersections, pavement dropoff, and bridge width. NAS is undertaking additional research on sight distance, roadside, and pavement dropoff and will attempt to assess the impact on the highway system of various policies, such as new construction standards and exceptions, to overcome the "deficiencies" of the FHWA regulatory analysis. It is unlikely, however, the outcome will be substantially different from the previous work.

FHWA has conducted reviews in all nine FHWA Regions, including at least two States in each Region, to assess the following:

- Activities undertaken by FHWA's Region and Division Offices to manage and monitor the implementation of non-Interstate RRR projects, including all actions taken pursuant to the final 1982 RRR Rule.
- The process used by FHWA's Division Offices to review and approve non-Interstate RRR projects.

• Recently completed non-Interstate RRR projects to observe the general scope of these projects, including the limitations imposed by various constraints and the extent to which safety has been enhanced.

The following were concluded based on the reviews:

• Safety has been improved to some degree on all projects reviewed and has been enhanced significantly on many projects. Although resurfacing using bituminous concrete overlays was the predominant item of work, many of the projects included additional items of work intended to enhance safety. The extent of safety enhancement depended primarily on terrain, available right-of-way, and available funds. Projects involving only resurfacing usually were on highways constructed to present design standards for new construction or on highways that received significant safety upgrading in the past.

Many of the RRR projects reviewed were developed before the issuance of the 1982 RRR Rule and its subsequent revision emphasizing the need to enhance safety on RRR projects. However, it was observed that only 12 percent of the projects reviewed involved resurfacing only.

States with very limited funds relied on Federal-aid funds to meet critical pavement rehabilitation needs and generally were less receptive to any additional extensive safety improvements, accomplished in conjunction with the pavement improvements, that would reduce the miles (kilometres) of pavement the States could rehabilitate.

The amount of safety enhancement generally was greater when RRR projects were developed through a State's design section rather than the maintenance section and when the FHWA Division actively was involved in the development process.

• The flexibility provided by the 1982 RRR Rule generally has been well received by States and has been relatively easy to implement; however, there is considerable concern over the proper definition of the term "enhance highway safety."

• Available accident data and analyses have not been used to the extent necessary to identify problem areas or potentially hazardous conditions on RRR projects.

• Although most States routinely established an appropriate design speed for use in development of RRR projects, several projects had established design speeds below the posted speed limit, rendering any consideration of curvature, sight distance, and other speed-related features, including exceptions to these items, questionable.

• In most States, the full range of exceptions to the adopted design criteria (usually criteria for new construction) were not being identified and approved. Specifically, exceptions involving less easily measurable geometric design criteria, such as stopping sight distance, horizontal and vertical curvature, and superelevation, frequently were not identified or considered. Another commonly overlooked item was structurally inadequate and/or functionally obsolete bridge rails.

Further, documentation of the disposition of exceptions that were identified is inconsistent and often inadequate.

• Despite the overall enhancement of safety observed on all projects, several potential improvements to further enhance highway safety were observed, including slope flattening, upgrading pavement markings, installing additional signing, and replacing damaged traffic control devices.

These additional improvements, however, present another obstacle to the proponents of using accident studies to determine geometric adequacy. As previously mentioned, accident studies require static conditions except for the element being examined. Thus, only one improvement (for example, lane widening) can be accomplished and analyzed at a time. Including additional safety improvements in RRR projects negates the ability to detect changes of interest, so tradeoffs and analysis at the State and local levels will become increasingly important. Use of existing data for analysis will provide required insights. This coupled with state-of-the-art information will assist in making the RRR program successful.

REFERENCES

(*1*) "Comparison of Accident Rates Before and After Rehabilitation of Narrow Pavements," *Arkansas State Highway Department*, Little Rock, Ark., October 1974.

(*2*) "Research on Road Traffic," *Road Research Laboratory, Great Britain Ministry of Transport*, 1965, pp. 406-470.

(*3*) B. Kolsrud and G. Nilsson, "Relationship Between Road Surface and Journey Speed," Volume 1, Bulletin 277, *National Swedish Road and Traffic Research Institute*, June 1982.

(*4*) "RRR Alternative Evaluations for Non-Interstate Rural Arterial and Collector Highway Systems," *Federal Highway Administration*, Washington, D.C., March 1980.

(*5*) "Safety Effectiveness Evaluation: Federal Highway Administration Non-Interstate Resurfacing, Restoration, and Rehabilitation Program," Report No. NTSB-SEE-81-4, *National Transportation Safety Board*, Washington, D.C., September 1981.

(*6*) Surface Transportation Assistance Act of 1982, sec. 110(a), 97th Cong., 2d sess., December 1982.

Julie Anna Cirillo is Chief of the Safety and Design Division, Office of Safety and Traffic Operations Research and Development, FHWA. Ms. Cirillo has been directly involved in accident research, geometric design, pedestrian safety, and truck size and weight limitations.

Recent Research Reports You Should Know About

The following are brief descriptions of selected reports recently published by the Federal Highway Administration, Offices of Research, Development, and Technology. The Office of Engineering and Highway Operations Research and Development (R&D) includes the Structures Division; Pavement Division; Construction, Maintenance, and Environmental Design Division; and the Materials Technology and Chemistry Division. The Office of Safety and Traffic Operations R&D includes the Systems Technology Division, Safety and Design Division, Traffic Control and Operations Division, and Urban Traffic Management Division. The reports are available from the source noted at the end of each description.

When ordering from the National Technical Information Service (NTIS), use PB number and/or the report number with the report title.

Requests for items available from NTIS should be addressed to:
National Technical Information Service
5285 Port Royal Road
Springfield, Virginia 22161

Safety Modifications of Turned-Down Guardrail Terminals, Volume 1, Executive Summary, and Volume 2, Technical Results, Report Nos. FHWA/RD-84/034 and -84/035

by Safety and Design Division

These reports present the results of a study to develop safer turned-down guardrail terminals. The study used analysis, laboratory testing, and full-scale crash testing to develop a new terminal system, Controlled Releasing Terminal (CRT), for the W-beam guardrail. The CRT was evaluated during 30 full-scale crash tests using 1,800 lb (0.82 Mg), 2,250 lb (1.02 Mg), and 4,500 lb (2.04 Mg) vehicles and meets the requirements of Transportation Research Circular 191 and National Cooperative Highway Research Program Report 230. Results of each full-scale test are presented as well as results from several analytical studies and the laboratory test series.

The reports may be purchased from NTIS.

Development of Strength in Cements, Report No. FHWA/RD-83/049

by Materials Technology and Chemistry Division

This report deals with further studies of phosphate-doped silicates, including thermal analyses and strength testing of α- and β-dicalcium silicate solid solutions. Other reported findings relate to stabilization and strength develop-

ment, chemical makeup of raw mix and fired samples, hydration products, and compressive strengths of paste and mortar samples. Reported tests show that α-C_2S develops C-S-H gel and strength faster than β-C_2S. Also, greater and faster gel and strength development was obtained for samples prepared from reagent grade chemicals rather than natural materials. Additionally, repeated firings increase gel formation and strength in β-C_2S samples.

Further studies of Sorel Cement also are reported, including findings of the condition governing the stability of the major phases of magnesium oxychloride cement. Also, the precipitation of various solid phases, their transformation with time, and their areas of stability were investigated in a broad concentration range of magnesium chloride and sodium hydroxide. Included are experimental data concerning the concentration of reactants, pH values, and analyses of solid phases at various time intervals. In all, four phases, pure or mixed, of magnesium oxychloride cement are identifiable.

The report may be purchased from NTIS.

Low-Cost Bridge Deck Surface Treatment, Report No. FHWA/RD-84/001

by Construction, Maintenance, and Environmental Design Division

This report describes an investigation of low-cost materials for sealing a bridge deck before placing an asphaltic concrete overlay. The objective was to find materials with lower in-place costs and more resistance to water penetration than preformed membrane systems.

A literature search and manufacturer inquiries resulted in 110 candidate materials. This list was reduced to six materials that best met the study's requirements. These six materials then were evaluated in the laboratory to determine their effect on water

absorption, resistance to deicer scaling, and adhesion of asphaltic concrete. The effect of placing hot asphalt on the materials also was evaluated as was the materials' ability to seal a concrete surface after a period of outgassing.

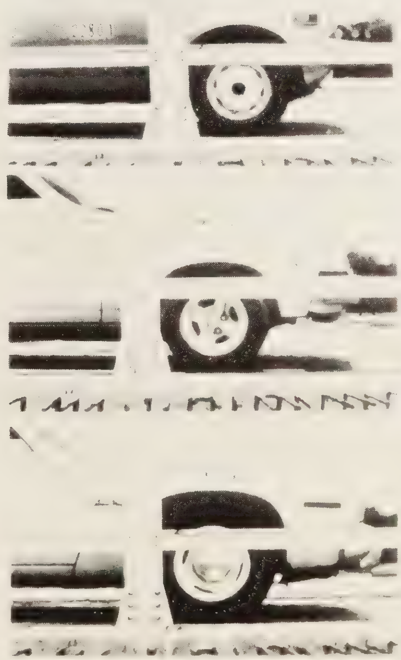
The three best materials were tested on portland cement concrete slabs in outdoor exposure tests. The slabs were overlaid with asphaltic concrete and subjected to salt pondings. Sealer effectiveness was measured by monitoring reinforcing steel corrosion and measuring concrete chloride content at the end of the test. Guidelines for using the three materials were prepared and are contained in the report.

The report may be purchased from NTIS.

Safer Bridge Railings, Report Nos. FHWA/RD-82/072-074.2

by Safety and Design Division

These reports examine the performance of five in-service bridge railing systems intended to meet the current (1976) American Association of State Highway and Transportation Officials specifications. Impact tests were conducted with vehicles ranging in size from an 1,800 lb (0.82 Mg) subcompact automobile to a 32,000 lb (14.5 Mg) intercity bus. Tests also were conducted with an instrumented rigid wall to determine the forces that an unyielding barrier must withstand to redirect these vehicles at selected impact angles, assuming there is no snagging.



Design guidelines to improve the impact performance of bridge railing systems were developed as were suggested improvements to the bridge rail performance standards of National Cooperative Highway Research Program Report 230. The data on which the design guidelines are based are limited and cannot guarantee fully satisfactory impact performance for a given bridge rail design. Thus, while these guidelines should improve impact performance of bridge railings, it is suggested that final acceptance of a design be based on performance demonstrated through full-scale crash tests.

Volume 1 provides a summary of the study, and volume 2 contains elastic and ultimate strength analyses of the tested railings and explains the developments of the design guidelines and suggested revisions to the performance standards. Volumes 3 and 4 contain results of the 30 full-scale crash tests.

The reports may be purchased from NTIS.

PROBAQM—A Probabilistic Air Quality Model for Highways, Report No. FHWA/RD-84/046

by Construction, Maintenance, and Environmental Design Division

This report is based upon and expands earlier research on the near roadway impact of air pollutants. The computer program PROBAQM supersedes PROBCO and SIMCO1, which examine the a priori probability of violations of air pollutant concentrations for 1-hour periods which are not to be exceeded more than once a year for a given single source roadway. In contrast, versatile PROBAQM can model multiple highway section sources and can address any pollution concentration level for any multiple hour period with any allowable violations per year for air contaminant concentrations.

PROBAQM, an efficient user-friendly model, provides realistic air quality impacts from highways for any inert gas substance such as carbon monoxide. PROBAQM avoids subjective assessments commonly used for evaluating near roadway air pollution impacts. It verifies and corrects user impacts, if necessary, incorporates accessible defaults for traffic information, and echoes user inputs and default data. Easily under-

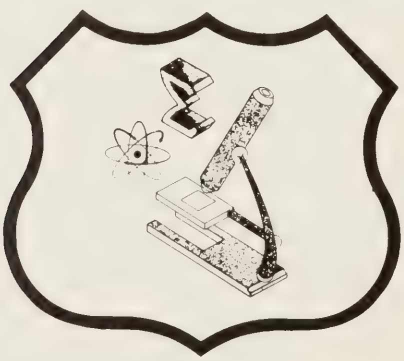
stood informative outputs include histograms and analysis of background conditions.

The report may be purchased from NTIS.

Chemical Composition of Asphalt as Related to Asphalt Durability—State-of-the-Art, Report No. FHWA/RD-84/047

by Materials Technology and Chemistry Division

This report presents a concise, critical review of the literature relating the chemical composition of asphalt to the durability of asphalt mixtures and pavements. Two major chemical factors affecting asphalt durability are compatibility of the interacting asphalt components and resistance to change from oxidative aging. The identification and characterization of the interacting chemical functional types normally present in asphalt or formed on oxidative aging affords a fundamental approach to composition-property-performance relationships of both asphalts and asphalt-aggregate mixtures. In



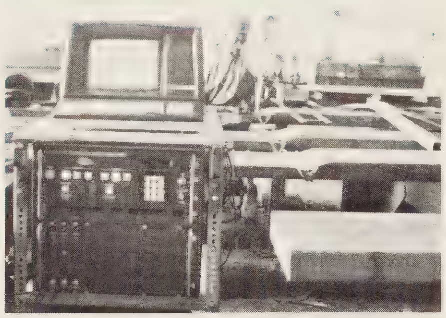
addition to chemical functionality formed on oxidation, asphalt properties are altered significantly by molecular structuring—sometimes called steric hardening. This phenomenon, although highly elusive and difficult to quantify, may be a major factor contributing to pavement embrittlement.

The report will be of interest to research and operations personnel concerned with asphalt pavement distress. It also will provide a convenient reference for asphalt technologists concerned with the chemical properties of these materials.

The report may be purchased from NTIS.

Improved Fabrication and Inspection of Welded Connections in Bridge Structures, Report No. FHWA/RD-83/006

by Structures Division



This report describes a prototype acoustic emission weld quality monitoring system for use during shop fabrication. The first part of the report describes the optimization and application of acoustic emission monitoring to the in-process detection, location, and characterization of flaws in welded connections for highway bridges. The second part of the report presents evaluations of tensile, fracture toughness, and fatigue properties of welded connections fabricated using steels commonly used in bridge construction. Various welding techniques were used in the fabrication.

The report will be of interest to bridge engineers involved in fabrication, maintenance, and inspection.

The report may be purchased from NTIS.

State-of-the-Art in Asphalt Pavement Specifications, Report No. FHWA/RD-84/075

by Construction, Maintenance, and Environmental Design Division

Although substantial progress has been made in the past 20 years in the development of statistically based specifications that account for the variability of highway construction materials and processes, much research still needs to be done. One major concern has been that some testing requirements in the specifications may not be related to the performance of pavements. The optimum quality control and acceptance tests and their frequencies must be established to provide the best balance between quality and cost.

This report is a first step in the development of specifications built around quality criteria that truly are related to pavement performance. The report begins with a brief history of asphalt concrete specifications, then discusses progress made in the development of statistically based specifications and in quality assurance practices. Requirements of current specifications are reviewed and summarized in tables. Significant asphalt concrete pavement performance studies also are reviewed.

Principal distress modes for asphaltic pavements are outlined, and the factors contributing toward each distress type are analyzed. The framework for a system of performance-related specifications, complete with suggested test procedures and construction requirements to support the system, are established. The report should be useful to specifications writers and those responsible for quality assurance.

The report may be purchased from NTIS.

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, Offices of Research, Development, and Technology (RD&T), Federal Highway Administration. Some items by others are included when they have a special interest to highway agencies. Requests for items available from the RD&T Report Center should be addressed to:

Federal Highway Administration
RD&T Report Center, HRD-11
6300 Georgetown Pike
McLean, Virginia 22101

When ordering from the National Technical Information Service (NTIS), use PB number and/or the report number with the report title.

Requests for items available from NTIS should be addressed to:
National Technical Information Service
5285 Port Royal Road
Springfield, Virginia 22161

Handbook on Design of Piles and Drilled Shafts Under Lateral Load, Report No. FHWA-IP-84-11

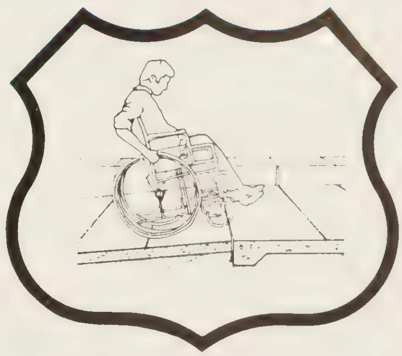
by Office of Implementation

This handbook is designed for geotechnical and structural engineers involved in the design of foundations for highway structures. Several design and analysis methods for piles under lateral loading are presented, including the computer program COM 624, a nondimensional solution, and the Broms method. Step-by-step procedures, including example problems for each method, are presented to help engineers better understand the methods contained in the handbook.

The report may be purchased from NTIS.

Guidelines for Making Pedestrian Crossing Structures Accessible, Report No. FHWA-IP-84-6

by Office of Implementation



Accessibility for handicapped persons is an important consideration in designing and locating pedestrian over- and undercrossing structures. Even if it is impractical to provide full accessibility, as many in the handicapped population as possible should be accommodated on the pedestrian structure.

This report provides recommendations for the design of over- and underpasses to accommodate both handicapped persons in wheelchairs and those with visual impairments. The maximum ramp grades and distances between resting places needed by persons in wheelchairs are provided as are design requirements necessary to accommodate persons with other kinds of ambulatory disabilities. Features necessary to accommodate persons with limited vision and no vision are detailed.

The report may be purchased from NTIS.

Priority Accessible Network for the Elderly and Handicapped in Seattle, Report No. FHWA-TS-84-205, and Priority Accessible Network for the Elderly and Handicapped in New Orleans, Report No. FHWA-TS-84-206

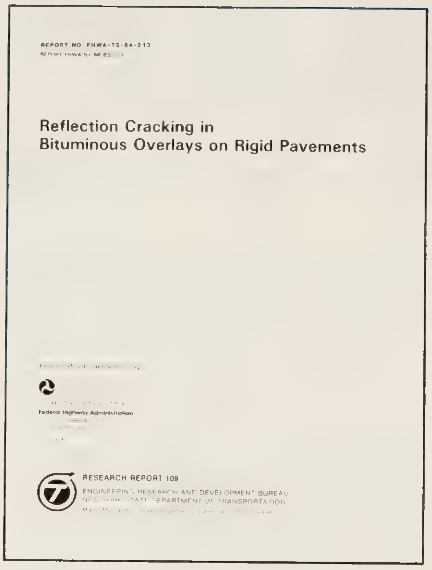
by Office of Implementation

These reports document case studies conducted in Seattle, Washington, and New Orleans, Louisiana, in applying the Federal Highway Administration procedures for assessing the needs of elderly and handicapped pedestrians in urban areas. These procedures are contained in the FHWA report **Development of Priority Accessible Networks—Provisions for the Elderly and Handicapped Pedestrians** (Report No. FHWA-IP-80-8). The information ranges from applying a broad-based planning approach to designing, cost estimating, and evaluating specific improvements along selected routes. The case study reports should be useful to municipal agencies serving elderly and handicapped people and to transportation authorities who are planning to implement such facilities.

The reports may be purchased from NTIS.

Reflection Cracking in Bituminous Overlays on Rigid Pavements, Report No. FHWA-TS-84-213

by New York Department of Transportation and Office of Implementation


REPORT NO. FHWA-TS-84-213

Reflection Cracking in
Bituminous Overlays on Rigid Pavements

Federal Highway Administration

RESEARCH REPORT 109
ENGINEERING RESEARCH AND DEVELOPMENT BUREAU
NEW YORK STATE DEPARTMENT OF TRANSPORTATION

Highway engineers have been concerned about reflection cracking in asphalt overlays on concrete pavements for over 50 years. The reflection cracking problem is neither minor nor confined to any particular geographical or environmental area. This report summarizes the results of over 20 years of testing reflection crack-retarding methods by the New York Department of Transportation and updates the performance of several test pavements. The report also reviews surveys to substantiate the causes and extent of the cracking experienced in New York, as well as the theories behind each preventive method.

The report, originally published by the New York Department of Transportation (Report No. FHWA/NY/RR-83/109), is an excellent summary of various methods tried to reduce reflection cracking and has been reprinted by the Federal Highway Administration to give it a broader national distribution.

The report may be purchased from NTIS.

Methods of Effective Transfer and Implementation of Highway Maintenance Technology, Report No. FHWA/RD-84/501

by Office of Implementation

This report reviews current technology transfer efforts of the Federal Highway Administration and analyzes other selected models of technology transfer. Specific techniques analyzed include circuit riders, conferences/short courses, slide-tapes, state-of-the-art reports, technical notices, trade publications, and changeable add-on notebooks.

The report makes the following recommendations: (1) Technology transfer efforts, where possible, should be carried out by full-time specialists who emphasize face-to-face communication and local feedback; (2) a training program should be developed for technology transfer specialists who work with State and local officials on highway-related problems; (3) a mechanism should be developed to improve the quality and utility of written materials, as well as an effort to control the amount of information that is disseminated to local agencies; (4) the Federal Highway Administration should try to get involved with local user networks such as professional or trade associations to increase peer-to-peer communication; (5) the market for maintenance information should be viewed on a segmented basis; and (6) research use should be viewed as an essential component of all stages of the research process.

The report may be purchased from NTIS.

Pavement and Shoulder Maintenance Performance Guides, Report No. FHWA-TS-84-208

by Office of Implementation

This report summarizes the results of a cooperative study of seven pavement and shoulder maintenance activities by the States of Arkansas, Colorado, Iowa, New Mexico, North Dakota, South Dakota, Utah, and Washington. The participating States exchanged information on each activity and prepared a series of performance guides based on the information exchange and a 1-year evaluation period. These guides outline the procedures, expected performance, equipment, materials, crew sizes, and productivity levels for each maintenance activity.

The guides were field tested over a 12-month period, and final revisions were made based on the testing. The performance guide for each topic is considered to be the state-of-the-art on existing materials, equipment, and procedures. The report should be of interest to State and local maintenance engineers concerned with maintaining pavements and shoulders.

The report may be purchased from NTIS.

Operational and Performance Characteristics of Drum Mix Plants, Report No. FHWA-TS-84-212

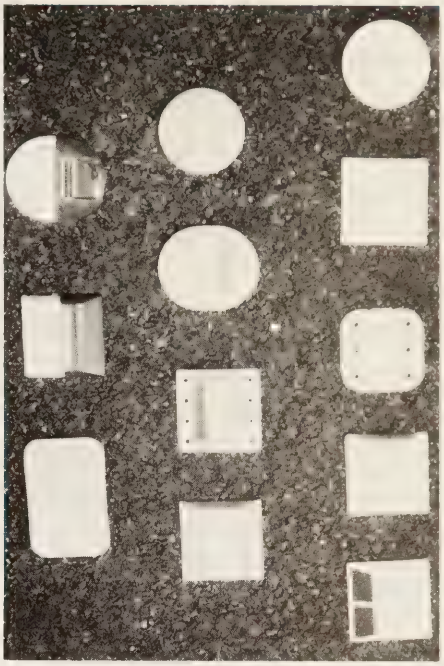
by Office of Implementation

This report compares asphaltic concrete mixtures produced by drum mix and batch plants. Design, production, construction, and performance data were collected on several "project pairs." These data were analyzed and compared to determine if there were any significant differences in mix performance that could be attributed to the production process. Results of the study are presented along with several conclusions and recommendations.

The report may be purchased from NTIS.

Raised Pavement Markers at Hazardous Locations, Report No. FHWA-TS-84-215

by Office of Implementation



Eleven State highway agencies participated in a study to evaluate the effectiveness of raised pavement markers at potentially hazardous locations, which included a variety of roadway conditions.

The consensus was that using raised pavement markers in high-hazard locations enhanced delineation and improved the overall safety of the locations. Using markers to supplement standard striping and signing greatly improves motorist visibility; however, raised pavement markers cannot reduce the potential hazards at all locations.

The report may be purchased from NTIS.

New Research in Progress

The following new research studies reported by FHWA's Offices of Research, Development, and Technology are sponsored in whole or in part with Federal highway funds. For further details on a particular study, please note the kind of study at the end of each description and contact the following: Staff and administrative contract research—*Public Roads* magazine; Highway Planning and Research (HP&R)—performing State highway or transportation department; National Cooperative Highway Research Program (NCHRP)—Program Director, National Cooperative Highway Research Program, Transportation Research Board, 2101 Constitution Avenue, NW., Washington, D.C. 20418.

FCP Category 1—Highway Design and Operation for Safety

FCP Project 1A: Traffic and Safety Control Devices

Title: Evaluation of Manual on Uniform Traffic Control Devices (MUTCD) Criteria. (FCP No. 31A3102)
Objective: Study and evaluate historical data to identify the driver and vehicle characteristics used and considered in the development of traffic control devices in the MUTCD. Identify research needs to support the MUTCD, and design studies to fill the gaps.

Performing Organization: Comsis Corporation, Mountain View, Calif. 94045
Expected Completion Date: March 1986
Estimated Cost: $123,650 (FHWA Administrative Contract)

Title: Wrong-Way Traffic Control at Intersections. (FCP No. 31A3114)
Objective: Conduct a survey to determine current practices and policies for signing and marking intersections of divided highways and one-way streets to properly inform drivers of allowed turning movements and/or warn motorists of wrong-way movements. Collect data at locations where wrong-way problems are frequent to identify possible candidate solutions to improve the situation.
Performing Organization: IFR Applications, Inc., State College, Penn. 16801
Expected Completion Date: December 1985
Estimated Cost: $100,000 (FHWA Administrative Contract)

FCP Project 1S: Design and Corrective Geometrics

Title: Improved Perception-Reaction Time Information for Intersection Sight Distance. (FCP No. 31S1142)
Objective: Develop, through field studies and laboratory experiments, the driving population distribution of perception-reaction times to various operational situations, such as stopping distances at intersections.

Performing Organizations: Institute for Research, State College, Penn. 16801
Expected Completion Date: July 1986
Estimated Cost: $177,500 (FHWA Administrative Contract)

FCP Project 1T: Roadside Safety Hardware

Title: Roadside Safety Library, Data Base Update. (FCP No. 31T4104)
Objective: Debug three computer programs and install them on the FHWA computer system for use in efficiently checking criteria for barrier design and projected vehicular trajectory on impact.
Performing Organization: Analysis Group, Inc., Washington, D.C. 20010
Expected Completion Date: June 1986
Estimated Cost: $309,300 (FHWA Administrative Contract)

Title: Evaluation of the Effectiveness of Crash Cushion Delineation. (FCP No. 31T4213)
Objective: Develop and evaluate, in the laboratory and in the field, crash cushion delineation treatments suitable for day and night use that effectively reduce the number of crash cushion-single vehicle accidents. Based on the evaluation, identify low-cost treatments and associated warrants for their use.
Performing Organization: Comsis Corporation, Wheaton, Md. 20902
Expected Completion Date: June 1986
Estimated Cost: $197,220 (FHWA Administrative Contract)

FCP Project 1U: Large Truck Safety

Title: Development of a Large Truck Safety Data Needs Study Plan. (FCP No. 31U1072)
Objective: Develop a detailed plan that will describe truck safety data needs and how such data can be collected. Include uses of available data, modifications to existing data systems, and collection of new data. Include costs and schedule for acquiring data under various scenarios.
Performing Organization: Bellomo-McGee, Inc., Vienna, Va. 22180
Expected Completion Date: August 1985
Estimated Cost: $94,390 (FHWA Administrative Contract)

Title: The Operation of Larger Trucks on Roads and Streets With Restrictive Geometry. (FCP No. 31U2112)
Objective: Select situations where longer or wider trucks may have difficulty operating or may cause operational problems for other vehicles. Evaluate these situations through field evaluation and off-tracking calculations to determine if such operations might degrade safety on the sections in question.
Performing Organization: Goodell-Grivas, Inc., Southfield, Mich. 48075
Expected Completion Date: June 1986
Estimated Cost: $203,480 (FHWA Administrative Contract)

FCP Project 1W: Vehicle/Surface Interaction Problems

Title: Noncontact, Nondestructive Determination of Pavement Deflection Under a Moving Load. (FCP No. 41W3232)
Objective: Develop specifications and a preliminary design for a mobile, noncontact pavement deflection measuring system. Identify and analyze potential problem areas, and design a laboratory experiment to test selected system components under simulated road conditions.
Performing Organization: Surface Dynamics, Inc., Bloomfield Hills, Mich. 48013
Funding Agency: Michigan Department of Transportation
Expected Completion Date: December 1985
Estimated Cost: $265,130 (HP&R)

FCP Category 3—Highway Operations

FCP Project 3A: Maintenance Management

Title: Cost-Effective Bridge Maintenance Strategies. (FCP No. 33A1212)
Objective: Address bridge maintenance measures such as short-term patching of concrete, spot painting, and maintenance of joints—not major structural inspection or rehabilitation. Use existing data and evaluate current maintenance practices to develop recommendations for maintenance managers on when, how, and where to allocate and prioritize limited maintenance resources to extend the life of bridge structures.
Performing Organization: KRW, Inc., Springfield, Va. 22152
Expected Completion Date: June 1986
Estimated Cost: $158,880 (FHWA Administrative Contract)

FCP Project 3C: Calcium Magnesium Acetate as an Alternate Deicer

Title: Effects of Calcium Magnesium Acetate (CMA) on Pavements and Motor Vehicles. (FCP No. 33C3026)
Objective: Determine the corrosion and chemical effects of CMA on automobile and other transportation vehicle components such as vehicle paint, metallic automobile bodies, tires, chrome, and chassis metals. Quantify chemical effects and corrosion rates and compare them with those rates and effects found for salt (sodium and calcium chlorides).
Performing Organization: Daedalean Associates, Inc., Woodbine, Md. 21797
Expected Completion Date: August 1986
Estimated Cost: $200,000 (FHWA Administrative Contract)

FCP Category 4—Pavement Design, Construction, and Management

FCP Project 4A: Pavement Management Strategies

Title: Evaluation of Increased Pavement Loading. (FCP No. 44A1372)
Objective: Develop a more comprehensive and precise method to predict the number of 18 kip (8.2 Mg) equivalent single axle wheel loads to use in design.
Performing Organization: Austin Research Engineers, Inc., Austin, Tex. 78746
Funding Agency: Arizona Department of Transportation
Expected Completion Date: July 1985
Estimated Cost: $180,000 (HP&R)

FCP Project 4B: Design and Rehabilitation of Rigid Pavements

Title: Mechanistic Design Procedures for Portland Cement Concrete (PCC) Pavements. (FCP No. 44B1182)
Objective: Develop procedures (a computerized analysis program that will be applicable for any set of highway serviceability requirements nationwide) and criteria for the design of PCC pavements. Use the computer program to develop a set of design charts and nomographs to handle most normal design conditions nationwide.
Performing Organization: University of Illinois, Urbana, Ill. 61801
Funding Agency: Illinois Department of Transportation
Expected Completion Date: June 1987
Estimated Cost: $58,000 (HP&R)

FCP Project 4C: Design and Rehabilitation of Flexible Pavements

Title: Development of Asphaltic Concrete Overlay Design Equations. (FCP No. 34C1022)
Objective: Develop overlay equations (like those already developed for Minnesota and Texas) for climatic or geographical areas making use of available usable data and providing some additional falling weight deflectometer testing. Use these regional equations to update the American Association of State Highway and Transportation Officials pavement design guide.
Performing Organization: ERES Consultants, Inc., Champaign, Ill. 41820
Expected Completion Date: July 1986
Estimated Cost: $114,690 (FHWA Administrative Contract)

Title: Development and Implementation of Overlay Design Procedures. (FCP No. 44C1133)
Objective: Develop a mechanistic-based overlay design procedure that can use deflection basin data obtained with the falling weight deflectometer. Develop a fuller understanding of and incorporate the environmental effects on seasonal strength variation of pavement structures in the overlay design procedures.
Performing Organization: Washington State Department of Transportation, Olympia, Wash. 98501
Expected Completion Date: June 1987
Estimated Cost: $99,680 (HP&R)

Title: Flexible Pavement Overlay Design by Dynamic Deflections. (FCP No. 44C1143)
Objective: Review the literature on dynamic deflection and overlay design procedures and determine the advantages and limitations of each. Where possible, visit sites where the actual equipment will be used. Summarize the findings to determine the direction for the State to proceed.
Performing Organization: University of South Carolina, Columbia, S.C. 29202
Funding Agency: South Carolina Department of Highways and Public Transportation
Expected Completion Date: August 1985
Estimated Cost: $61,450 (HP&R)

☆U.S. Government Printing Office:1984—461-830/10001

Title: Investigation of Asphalt Admixtures. (FCP No. 34C5014)
Objective: Identify the most promising asphalt additive(s) or admixture(s) and examine the interaction of the modified asphalt mixtures relating to stability, durability, and flexibility of the asphalt film in aggregate-asphalt hot mixes.
Performing Organization: Texas A&M Research Foundation, College Station, Tex. 77843
Expected Completion Date: August 1986
Estimated Cost: $225,000 (FHWA Administrative Contract)

FCP Project 4Z: Implementation of New Pavement Technology

Title: Evaluation of Alternatives to Improve Pavement Designs. (FCP No. 34ZA098)
Objective: Evaluate the effectiveness of various design features of premium pavements (for example, tied shoulders, effective drainage, thickness, and improved quality control) by comparing the resulting pavement sections with those using the American Association of State Highway and Transportation Officials interim guide and State pavement design methods. Use data from four or five States representing different climatic zones for jointed plain concrete, continuous reinforced concrete, and flexible and composite pavement designs. Conduct an economic analysis to establish the life cycle cost, and evaluate similarities and differences between methods.
Performing Organization: Globetrotters Engineering Corporation, Chicago, Ill. 60606
Expected Completion Date: October 1985
Estimated Cost: $81,075 (FHWA Administrative Contract)

FCP Category 5—Structural Design and Hydraulics

FCP Project 5A: Bridge Loading and Design Criteria

Title: Design of Concrete Bridge Decks. (FCP No. 45A3112)
Objective: Study the influence of different parameters (reinforcement ratio and arrangement of reinforcement) on the design of concrete bridge decks.
Performing Organization: Case Western Reserve University, Cleveland, Ohio 44106
Funding Agency: Ohio Department of Transportation
Expected Completion Date: December 1985
Estimated Cost: $70,000 (HP&R)

FCP Project 5K: Bridge Rehabilitation Technology

Title: Weigh-in-Motion Applied to Bridge Evaluation. (FCP No. 45K2000)
Objective: Test a number of existing bridges in the field with weigh-in-motion equipment to assist in bridge evaluation and rating. Compare bridges that are conventionally rated and record data on truck loads, dynamic impact, member force distribution, and stresses. Incorporate these data in an improved rating analysis. Train State transportation personnel in the use of weigh-in-motion equipment and processing programs for bridge evaluation.
Performing Organization: Case Western Reserve University, Cleveland, Ohio 44106
Funding Agency: Ohio Department of Transportation
Expected Completion Date: July 1985
Estimated Cost: $75,000 (HP&R)

FCP Category 0—Other New Studies

Title: An Evaluation of Tensile Strength Testing Versus Flexural Strength Testing as a Means of Quality Control for Portland Cement Concrete Pavements. (FCP No. 40M4834)
Objective: Determine whether the splitting tensile test (indirect tensile test) or the flexure test is the more reliable indicator of tensile strength of concrete for quality control and design purposes. If the splitting tensile test proves more reliable, investigate the feasibility of converting Texas' quality control for portland cement concrete structures and pavements, including equipment modification, to the splitting tensile test.
Performing Organization: University of Texas, Austin, Tex. 78712
Funding Agency: Texas State Department of Highways and Public Transportation
Expected Completion Date: August 1985
Estimated Cost: $70,000 (HP&R)

U.S. Department
of Transportation

Federal Highway
Administration

400 Seventh St., S.W.
Washington, D.C. 20590

Official Business
Penalty for Private Use $300

Postage and Fees Paid
Federal Highway
Administration
DOT 512

U.S. MAIL

SECOND CLASS
USPS 410-210

in this issue

The Application of Pedestrian Signals at Traffic-Signalized Intersections

Soil Reinforcement for Stabilization of Earth Slopes and Embankments

Steel Corrosion in Concrete: pH at Corrosion Sites

Safety Impacts of RRR Projects