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TRB
special report 148

# innovations in construction and maintenance of transportation facilities

transportation research board
national research council

special report 148

# innovations in construction and maintenance of transportation facilities

proceedings of the sixth summer meeting ot the highway research board
in cooperation with the washington department of highways
august 6-8, 1973
olympia, washington

subject areas

15 transportation economics
25 pavement design
26 pavement performance
31 bituminous materials and mixes
32 cement and concrete
33 construction
34 general materials
35 mineral aggregates
40 maintenance, general
41 construction and maintenance equipment
62 foundations (soils)
63 mechanics (earth mass)

transportation research board
national research council
washington, d.c., 1974

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Transportation Research Board Special Report 148
Price $7.40
Edited for TRB by Marjorie Moore

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LIBRARY OF CONGRESS CATALOGING IN PUBLICATION DATA

Main entry under title:

Innovations in construction and maintenance of transportation facilities.

(TRB special report; 148)

1. Pavements–Design and construction–Congresses.
2. Pavements–Maintenance and repair–Congresses.
3. Bridges–Maintenance and repair–Congresses.
4. Bridge construction–Congresses. I. National Research Council. Highway Research Board. II. Washington (State). Dept. of Highways. III. Series: National Research Council. Transportation Research Board. Special report–Transportation Research Board; 148.

TE251.I55 625.8 74-18965
ISBN 0-309-02297-5

contents

# foreword

This Special Report contains the papers presented at the Sixth Summer Meeting of the Highway Research Board, held in Olympia, Washington, August 6-8, 1973. The objective of this meeting was to present timely and useful information on innovations in the construction and maintenance of transportation facilities.

The papers by Alexander and by McManus and Barenberg in part 1 deal with trade-offs in design, construction, and maintenance. They provide an introduction to decision analysis and illustrate some capabilities of this technique.

Part 2 consists of six papers from a session organized by the Committee on Rigid Pavement Construction; they cover the latest developments in concrete pavement construction including full-width paving to a 14-inch depth in a single pass, slip-forming of median barriers, posttensioned pavement, new techniques for airport construction, uses of fiber-reinforced concrete, and construction of a test track for the Ohio Transportation Research Center.

The four papers in part 3 are devoted to construction innovations in the areas of quality control, embankment construction, use of gabions, and hot-mixed asphalt cooling; the four in part 4 discuss systems building of highway bridges. This latter group, which addresses sandwich panels, prestressed concrete panels, and precast bridge deck replacement, is a continuation of Special Report 132.

Skyrocketing energy costs and materials shortages spotlight the desirability and feasibility of using alternate materials and methods, such as Gussasphalt, which is widely used in Europe but new to the United States, and steel-fiber-reinforced concrete, which is now moving from the laboratory testing stage to large-scale field trials. The papers in part 5 give attention to these subjects.

# part 1

# trade-offs and decision analysis

# application of maintainability and expected cost decision analysis to highway design

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There are many trade-offs between construction and maintenance of highways. Conventional engineering economy methods such as the benefit-cost ratio are sometimes used to compare these trade-offs. However, conventional methods often cannot incorporate all the factors that should influence a decision. Two techniques that can extend the capabilities of the conventional engineering economy analysis are maintainability and expected cost decision analysis. These techniques are not new. They have been applied to real problems in the fields of aerospace and business administration for several years. This paper urges that these techniques be used to analyze highway design trade-offs and illustrates their use with the aid of example problems. Maintainability and expected cost decision analysis allow the engineer to use a more systematic approach to certain types of design problems. Application of this approach would lead to better decisions in many situations involving trade-offs between construction and maintenance costs.

•In highway design, as in the design of any facility, there are always trade-offs between initial costs and future costs. The major future costs for a highway are maintenance costs and road user costs.

For any component of a highway, or for the overall highway itself, there are usually options available that offer the engineer the choice of either high initial cost and low maintenance costs or lower initial cost and higher maintenance costs. Road user costs are also affected by the design, and, here also, lower future costs usually must be paid for by higher initial costs.

It is sometimes claimed that there is no significant trade-off between construction costs and maintenance costs. Anyone who has maintained a gravel surface under heavy traffic and then compared the maintenance costs to those of maintaining the same traffic over an adequate pavement knows that there are trade-offs to be considered. This is an exaggerated example but there are less obvious examples in every highway design. Various membranes, seals, and treatments on bridge decks usually have no purpose other than to reduce future maintenance and replacement costs. The same can be said of galvanized or other corrosion-resistant materials.

Balancing these trade-offs is a necessary part of any highway design. The trade-offs are always made. They cannot be avoided. They are made whether the designer consciously considers them or not.

Because reductions in future maintenance costs are usually hard to estimate, the "seat of the pants" method is often used to make the trade-off decision. No cost comparison or formal analysis of costs is attempted. "Seat of the pants" decision-making is sometimes coupled with a strong temptation to design highways as maintenance free as possible. Because it is often possible to spend far more in construction than can ever be recovered in reduced maintenance costs, this may, and sometimes does, lead to extravagant waste of public funds.

Low maintenance is not a virtue in itself and should not be sought to the point where further reduction requires a disproportionate increase in construction cost. At the other extreme, various pressures to limit the cost of construction can result in uneconomical trade-offs if construction cost savings are small compared to the additional required maintenance costs.

Proper consideration of trade-offs involves comparing the costs of competing designs. A valid comparison requires that the costs that occur in different years be properly discounted through use of an appropriate interest rate. This is usually done for highway designs by the familiar benefit-cost method. A straightforward benefit-cost analysis (or other similar engineering economy technique) should probably be the basic method of analyzing the trade-off opportunities in highway design. However, conventional engineering economy studies often cannot incorporate all the factors that should influence a decision. Certain types of easily overlooked constraints and the effect of uncertainty are two such factors that are often missing in conventional analyses. Two techniques that can extend the capabilities of the conventional engineering economy analysis are maintainability and expected cost decision analysis. Use of these techniques could improve the analysis of many trade-offs found in highway design.

The objective of this paper is to show how these techniques can be used in highway design, and the paper is written for the practicing highway engineer. Examples of the application of these techniques have been made as simple as possible to better illustrate the methods involved. As a result of this objective and approach, the paper will probably not be of great interest to those already familiar with these and the more sophisticated methods of decision analysis.

## APPLICATION OF MAINTAINABILITY

The concept of maintainability and most of the existing techniques based on this concept were developed in the electronics and aerospace fields. The systems developed in these fields, like highway systems, often require high operating and maintenance costs. As the electronics and aerospace systems became more complex, the problem of keeping these systems in operation became increasingly difficult. Problems with equipment failure and high maintenance cost became intolerable (1). This led to gradual change in the design philosophy.

The dominant objective of design had been to achieve high levels of performance when the system was functioning properly. This objective was gradually modified so that, in addition to concern about potential performance level, more emphasis was placed on questions relating to how often the system was going to function properly and how much effort would be required to keep it functioning.

Because the systems in question were complex, it was not usually apparent what effect various design options would have on behavior during service life. To answer these questions a variety of systematic methods were developed to assist the designer. Some of these methods are based on the concept of maintainability.

### Definition of Concept

Maintainability is a built-in characteristic of the physical system. It can be defined as a measure of the effort needed to maintain the system. In actual application, maintainability is defined to be most compatible with the analyses being made (2).

The concept of maintainability provides a means of quantifying the expected future maintenance of a system and allows consideration of this maintenance at the design stage along with the more familiar design parameters of performance, reliability, and initial cost.

Maintainability can be used as a design parameter (a) to allow trade-off between future maintenance requirements and other design parameters in order to find the optimum design or (b) to specify a maximum acceptable maintenance effort that the system may require.

The annual maintenance costs for the life of the system can be considered a measure of maintainability. Thus, the use of estimated maintenance costs in a conventional benefit-cost ratio analysis is an example of the use of the maintainability concept. Highway engineers have been using this type of maintainability analysis for many years, under a different name.

The other use of maintainability as a means of specifying the maximum acceptable amount of maintenance may also be useful for highways. This is now done in the electronics and aerospace fields where the required maintainability is routinely specified by the Department of Defense and the National Aeronautics and Space Administration.

If one of the designs being considered for a highway involves a level or type of maintenance that is unrealistic to expect in practice, it should be rejected no matter how low the estimated total cost may be. If a strategy requires heavy maintenance that is not provided, premature failure of the road, high user costs, or both may result. This in turn may cause the actual total cost to be higher than it would be for alternative designs that required only realistic maintenance efforts. This type of situation can be avoided by considering any future limitations on maintenance at the design stage.

Limitations or constraints on the amount or type of maintenance that will be available are real problems in highway design. These constraints may stem from a variety of causes. Future maintenance budgets may be limited for administrative or political reasons; the maintenance organization may not be capable of performing certain types of operations for lack of equipment, materials, or training; or there may be an administrative or political decision to make low maintenance a goal in itself.

Example 1—Use of Maintainability as a Design Constraint

The following example will illustrate how a limitation on the future maintenance available to a system can be specified as a system requirement and how this may lead to improved decision-making.

The example problem involves the selection of a surface for a low-volume highway. Traffic demand, prices for labor, equipment, material, environmental conditions, and other factors needed to define the problem have been estimated. The design is to be selected on the basis of lowest annual cost subject to a minimum maintainability specification.

This maintainability specification is based on knowledge of the local government and its maintenance organization. It is unrealistic to expect that roads in the area will receive more surface maintenance than can be provided by $380 per mile (annual cost) for the analysis period. An arbitrary definition of maintainability, M, for use in this example might be

$$M = \frac{1}{MC} \times 10{,}000$$

where MC is the annual cost of maintenance. This gives an index of maintainability that increases as maintenance costs decrease. A minimum M of 26 is specified to stay within the expected maintenance constraint of \$380 per mile (10,000 ÷ 380 = 26). Maintainability could also have been defined to equal the maintenance cost directly; we then would have specified a maximum limit.

A computer-based simulation model (3) was used to estimate the average costs of providing a surface for the road by a variety of designs. As a result of a series of runs the four surfaces given in Table 1 were selected as the most promising. The best of these four designs can be selected on the basis of lowest annual cost subject to our minimum maintainability constraint of 26. Design A (which specifies a well-maintained gravel surface) is eliminated from consideration because its expected M is less than the specified minimum. Of the remaining three designs, C, which specifies a bituminous surface treatment plus two additional seal coats during the analysis period, results in the lowest total annual cost. Although the expected maintainability of this strategy is above the specified minimum, it is very close to the limit. The degree of uncertainty in selecting the minimum required maintainability and in the accuracy of the model should be considered in the decision. If these estimates involve a high degree of uncertainty, as is likely, design D, which has a much higher maintainability, may be the best choice.

If the design for this project had been selected on the basis of minimum total cost, with no consideration given to the limit on future maintenance, design A would have been selected. But the limit on available maintenance would have resulted in a maintenance policy similar to that of design B. Actual user costs would also have been similar to those of design B, and the total costs of providing the system would have been greater than for either designs C or D.

This example illustrates how an anticipated constraint on future maintenance can be analytically specified as a design constraint by using the concept of maintainability, and how this can lead to a better decision. The construction, maintenance, and road user costs used in this example were estimated by the computer simulation model mentioned earlier. However, the principle involved (use of a maintainability constraint) is valid no matter how the costs are estimated. The problem of limited available maintenance is a reality in highway design. The concept of maintainability allows us to formally incorporate it into the design process.

## APPLICATION OF EXPECTED COST DECISION ANALYSIS

In the usual economic analysis, estimated future costs are taken at face value and no allowance is made for the often highly uncertain nature of these estimates. The uncertain nature of the predicted costs should be considered in the decision process. Expected cost decision analysis is a method of incorporating this uncertainty into the analysis (4).

The expected cost (EC) of a situation that may have any one of n outcomes can be defined as

$$EC = \sum_{i=1}^{i=n} [p(X_i) \times V_i]$$

where

$p(X_i)$ = probability that the outcome will be $X_i$, and
$V_i$ = cost of outcome $X_i$.

Table 1. Annual costs and maintainability index for various surface designs.

| No. | Design | Annual Cost (dollars/mile) Construction | Maintenance | User | Total | M |
|---|---|---|---|---|---|---|
| A | Gravel (1 blading per week) | 2,580 | 610 | 3,460 | 6,650 | 16 |
| B | Gravel (1 blading per 3 weeks) | 2,580 | 320 | 5,320 | 8,220 | 32 |
| C | Surface treatment (+2 seal coats) | 4,050 | 340 | 2,660 | 7,050 | 25 |
| D | Bituminous concrete (2 in.) | 5,200 | 210 | 2,400 | 7,810 | 48 |

Figure 1. Basic decision tree for example 3.

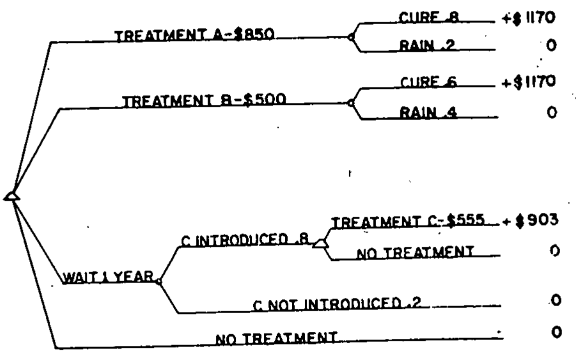
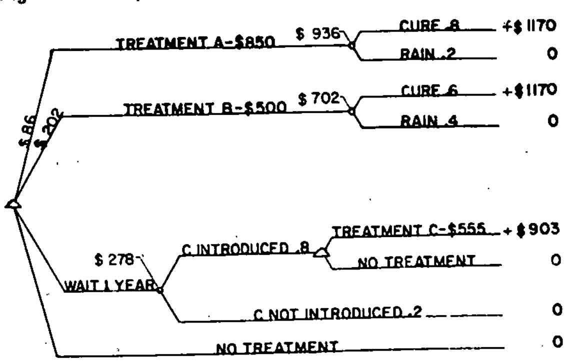
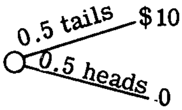



Figure 2. Completed decision tree for example 3.

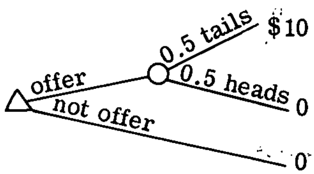


To illustrate this concept, suppose a benevolent gambler offers to pay someone $10 if he gets tails and nothing if he gets heads on one flip of a coin. How much value has he really given away? Certainly not $10 because winning the $10 is not certain. But the offer is of some value. If we assume that there is a 0.5 probability of getting tails, the expected cost to the gambler is $5. The position the gambler has put himself in can be represented by a chance node with two branches.




$$\begin{aligned} 0.5 \times \$10 &= \$5 \\ 0.5 \times 0 &= \underline{\ 0\ } \\ EC &= \$5 \end{aligned}$$

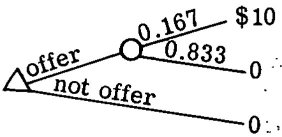
The gambler's choice of whether to offer the gift or not can be diagrammed in decision-tree form in which the decision nodes are marked with a △ and the chance nodes are marked with a ○.




$$\begin{aligned} 0.5 \times \$10 &= \$5 \\ 0.5 \times 0 &= \underline{\ 0\ } \\ EC &= \$5 \end{aligned}$$

The decision-tree diagram is especially useful for systematically representing more complicated decisions.

Similarly an offer to pay $10 for rolling a six with one roll of a die has an expected cost of $1.67, inasmuch as the probability of success is one out of six or 0.167.




$$\begin{aligned} 0.167 \times \$10 &= \$1.67 \\ 0.883 \times 0 &= \underline{\ 0\ } \\ EC &= \$1.67 \end{aligned}$$

The following example will attempt to illustrate the use of expected cost in the analysis of a highway design trade-off.

Example 2—Use of Expected Cost in Trade-Off Analysis

Suppose that present efforts to develop a treatment for preventing ice from bonding to pavements are successful. The treatment consists of a chemical treatment of the pavement surface that costs $1,000 per lane-mile. The treatment has the proven ability to prevent bonding for 5 years if allowed to cure for 5 days without being rained on. If it rains within 5 days of application, the treatment will be of no value.

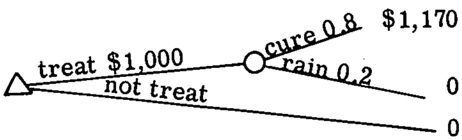
From a pilot test section it has been found that maintenance costs can be reduced by $285 per lane-mile if ice is prevented from bonding.

The benefit-cost ratio of a successful treatment can be computed, based on a 7 percent interest rate, as follows:

$$\begin{aligned} \text{Present worth of cost} &= \$1{,}000 \\ \text{Present worth of benefits} &= \$285 \times 4.10 = \$1{,}170 \\ \text{Benefit-cost ratio} &= \frac{1{,}170}{1{,}000} = 1.2 \end{aligned}$$

The 4.10 is the present worth factor for 5 years and 7 percent interest. This indicates that the treatment is a good investment because the benefit-cost ratio is larger than one. However, this analysis was made on the basis of a successful treatment, and there is some danger that rain will ruin the treatment before it has a chance to cure.

Assume that the probability of a successful treatment is estimated from weather records to be 0.8. At a probability of success of 0.8, the expected present worth of the benefits is $936. Because this is less than the $1,000 present cost of the treatment, it is now obvious that the treatment is not a good gamble. The decision to treat or not to treat can be represented by the following decision tree.




$$
\begin{aligned}
0.8 \times \$1,170 &= \$936 \\
0.2 \times 0 &= \underline{\quad 0} \\
EC &= \$936
\end{aligned}
$$

The benefit-cost ratio for this expected benefit is

$$\text{b-c} = \frac{936}{1,000} = 0.94$$

Because this ratio is smaller than 1.0, the expected cost analysis indicates that the treatment is not a good investment. Stated another way, the odds do not favor paying $1,000 for a treatment that will save $280 per year for 5 years at a probability of 0.8.

Use of expected cost analysis for more complicated problems requires finding the expected costs for all the available alternative designs and then selecting the design by an engineering economy comparison. This process is used in the following example.

<u>Example 3</u>

Assume that the engineer making the decision in example 2 has more than one available treatment for preventing ice from bonding. Potential treatments are as follows:

1. Treatment A is the same as treatment in example 2 except that the cost per lane-mile is reduced to $850;
2. Treatment B is similar to treatment A but costs $500 per lane-mile and requires a longer curing period without rain (probability of success = 0.6); and
3. Treatment C must be applied yearly at a cost of $175 per lane-mile and requires no curing period (probability of success = 1.0); this treatment is not now available but will be available next year maybe (probability of introduction next year = 0.8).

Treatments A and B are effective for 5 years. All three treatments are equally effective if successfully applied and save $285 per lane-mile per year. The engineer now has four alternatives:

1. Use treatment A,
2. Use treatment B,
3. Wait 1 year, and then use treatment C for the 4 remaining years in the analysis period, and
4. Use no treatment.

Because the probabilities of success and the costs are different for each alternative (as they always are in actual situations), it is not easy to pick the best alternative.

The problem can be represented by the decision tree shown in Figure 1. Although any one of the three treatments may reduce maintenance costs by $285 per year, treatment

C can reduce these costs only in years 2 through 5. Therefore, the present worth of the benefits shown at the end of that branch of the decision tree is smaller than for the other two treatments. Computations of present worths of costs and benefits were done from standard tables and will not be shown.

To select the best alternative by the expected cost method requires that each branch of the tree be evaluated by starting at the right end and working toward the left. Computations are similar to those shown in example 1. The results of these computations are shown on the completed decision tree in Figure 2.

The expected values of the four alternatives (branches) are as follows:

Treatment A
$$(0.8 \times \$1,170 + 0.2 \times 0) - \$850 = \$86$$

Treatment B
$$(0.6 \times \$1,170 + 0.4 \times 0) - 500 = \$202$$

Treatment C (at year 1)
$$(\$903 - \$555) \times 0.8 + 0.2 \times 0 = \$278$$

No Treatment 0

Treatment C can now be selected as the best of the alternatives on the basis of maximum present worth. Analysis by the benefit-cost method would lead to selection of the same alternative.

Estimates of future costs and benefits always involve some uncertainty. The degree of uncertainty should be considered in trade-off analysis. Expected cost decision analysis provides a simple technique for incorporating this uncertainty into the decision-making process in a quantitative way. Stated another way, expected cost decision analysis provides a technique for determining which choice in a trade-off decision has the best odds.

Thus, the decision does not depend entirely on point estimates of future costs and benefits but may also take into account the uncertainty of these estimates. In many trade-off situations, expected cost decision analysis will lead to a more rational decision than an analysis that ignores the inherent uncertainty of the cost estimates.

## SUMMARY

Examples have been used to illustrate the application of maintainability and expected cost decision analysis to highway design problems. However, the chief value of these techniques involves their application to more complex problems in which it is not so obvious what effect constraints and uncertainties should have on the decision. Illustration of more complex applications is beyond the scope of this paper, but more complete discussion may be found in the references.

The techniques described allow the engineer to use a more systematic approach to certain types of design problems. Application of these techniques would lead to more rational decisions in many situations involving trade-offs between construction costs and maintenance costs.

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# impact of subgrade variability on pavement construction-maintenance cost trade-offs

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A philosophy for determining design values for parameters that display stochastic variation is presented. An illustrative example is given that uses the subgrade CBR of a flexible airfield pavement as the stochastic parameter whose design value is desired. The analysis takes into account several plans for pavement maintenance. Based on the trade-off between construction costs and maintenance costs, the optimal maintenance and reconstruction plan and the optimal design CBR are chosen. It becomes clear from the discussion and example that it is just as possible to arrive at an uneconomical design by using too low a design CBR as it is by using too high a value. The only way to determine the proper design CBR is to consider life-cycle costs and evaluate the construction-maintenance cost trade-offs.

•There is within the pavement engineering profession a growing awareness of an interest in the life-cycle approach to pavement design. Simply stated, the life-cycle approach requires that all costs anticipated over the expected life of the pavement be included in the economic evaluation of alternate pavement systems. With this approach, realistic economic trade-offs can be made between factors affecting the average annual cost of pavements, with specific emphasis on the trade-offs between initial construction costs and maintenance costs.

If entire pavement sections failed uniformly as units, economic evaluation of the trade-offs between initial construction costs and maintenance costs would be straightforward and relatively simple to accomplish. Unfortunately, pavements do not fail uniformly but have service lives that vary over a wide spectrum. Thus, the problem of evaluating costs and benefits becomes a stochastic problem and must be treated as such.

Many factors influence the performance of pavements such as climatic conditions, traffic, and material properties, each of which may cause variability in pavement performance trends. It is generally agreed, however, that one of the more dominant factors affecting pavement behavior and performance is the subgrade support condition. Because subgrade support is a dominant factor controlling pavement performance and because subgrade soils are variable, the question arises of whether it is more economical from a life-cycle costing standpoint to design pavements based on the least expected support condition with little or no expected maintenance or on some more favorable support conditions such as the mean value, to expect some distress, and to repair the pavements as needed to sustain the pavement in a serviceable condition.

The purpose of this paper is to present a philosophy that considers the stochastic nature of paving materials as integral to the economic trade-off between initial construction and maintenance costs of pavement systems. Because subgrade support is a dominant factor in pavement performance the paper is developed around the effect of subgrade variability on the trade-offs between initial construction and maintenance costs. Use of subgrade variability is for illustrative purposes; similar analysis could just as easily be made with other parameters whose variability affects pavement service life.

When considering the stochastic nature of pavement system parameters, one is immediately faced with the problem of the size of area to consider for establishing appropriate statistical parameters. Point-to-point variations in measured subgrade parameters are partially the result of testing errors rather than variation in the materials. Even if these variations over very small areas are due to real variations in the subgrade properties, they will likely have little effect on the performance of the pavement as a whole.

Conversely, variations in the subgrade that occur over relatively large areas may have a significant effect on pavement performance. The areal size that has a significant effect on pavement performance has not been defined and is probably a function of pavement system parameters, including the thickness of the pavement layers and the properties of the paving materials. No attempt will be made here to determine the size of area required to influence pavement performance, but it will be assumed that the variations indicated are characteristic of sufficiently large areas to have an effect on the pavement.

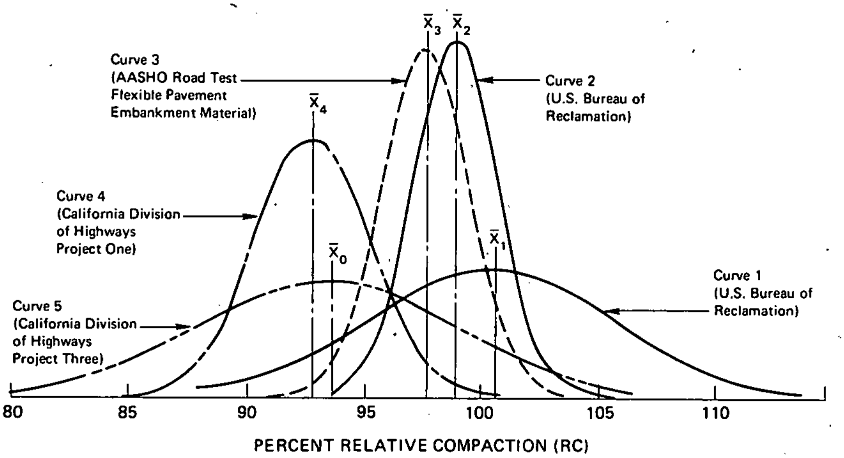
The variability of subgrade soils (Fig. 1) is well known (1, 2, 3). Causes of this variability range from the inherent stochastic nature of natural soils to the effects of nonuniform construction procedures. Minor changes in moisture content during compaction and nonuniform distribution of compactive effort will likely produce significant variations in the support characteristics of the soils. Figure 2 shows the effects of moisture content and density on the CBR values for a typical subgrade soil. For purposes of this paper it is assumed that the effective changes in moisture content and density will occur over portions of the pavement large enough to affect pavement performance (probably several square yards).

Figure 3 shows the variation in the CBR properties of the in situ subgrade for the AASHO Road Test. If we consider only the variation due to changes in the compacted density, and thus eliminate much of the variation due to testing error, the range of values for the CBR can be significantly reduced. Further eliminating the variations that occur over areas too small to influence pavement performance results in an additional reduction in the expected range of CBR values. This reduction is reflected in an assumed distribution of CBR values for the analyses presented (Fig. 4).

The distribution shown in Figure 4 is assumed to represent the entire pavement area. This is intended as an estimate of the distribution expected at the completion of construction and during the life of the pavement. It must be remembered that, if this procedure is used for design, the designer does not know at the time of the design what the ultimate distribution will be and, therefore, must make an estimate based on prior experience with the materials, the specifications used, and the level of quality control during construction. An illustrative evaluation of the sensitivity of the cost trade-off to changes in the standard deviation of the subgrade CBR is made later in the paper.

As an illustration of the points alluded to above, a specific example is presented. The example used is based on the Corps of Engineers design procedure for flexible airfield pavements (4). The philosophy expressed herein applies to the design of any pavement system; it could be illustrated just as well for highway pavements or rigid pavements. Furthermore, other design procedures could serve equally well to illustrate the principles involved, but this procedure was chosen because it has been programmed by

**Figure 1. Variation in subgrade density (1).**




| CURVE NO. | 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|---|
| Minimum Specification Limit, % RC | 98 | 98 | 95 | 90 | 90 |
| Compaction Test Method | Proctor E-11 | Proctor E-11 | AASHO T-99 | Calif. 216 | Calif. 216 |
| Average Compaction | 100.7 | 99.0 | 97.7 | 92.9 | 93.6 |
| Standard Deviation | 5.0 | 1.8 | 1.9 | 2.4 | 5.5 |
| Approximate % Less Than Minimum Specification Limit | 29.5 | 28.9 | 7.8 | 11.3 | 25.6 |

**Figure 2. Effect of density and moisture content on CBR of AASHO Road Test embankment soil (2).**

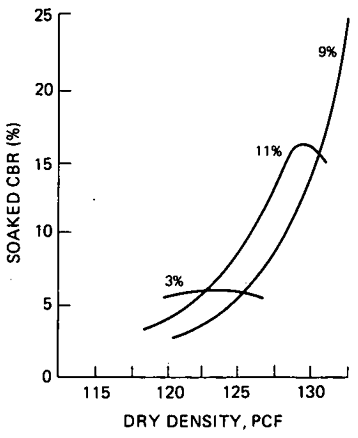
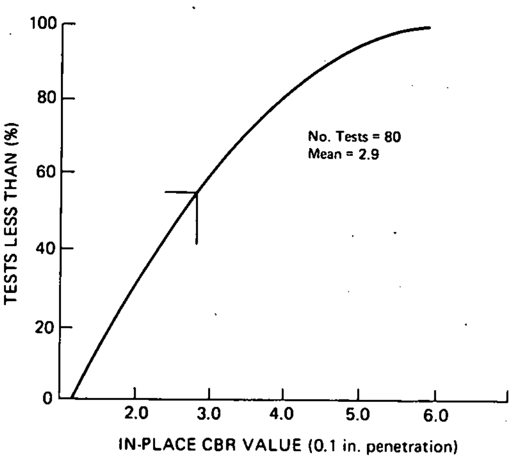



**Figure 3. In-place CBR determination of embankment on flexible tangents at time of paving (3).**



the U.S. Army Engineers CERL staff for life-cycle analysis of pavement systems. Many of the calculations for the development of this paper were made with the computerized program LIFE 1.

## PROCEDURE FOR DETERMINING LIFE-CYCLE COSTS

As has been discussed, the designer is faced with determining a design value for the subgrade CBR expected to be effective at the completion of construction. In addition to determining such a design value, the designer must also select a service life for the pavement. For purposes of this paper, design life is assumed to be an arbitrary period of time over which life-cycle costs are evaluated for various maintenance and reconstruction plans. Service life is the period of time from initial construction until a structural overlay is required. Thus the designer considers a design life, but he could design for a service life that is less than the design life and schedule a structural overlay at that time. Such a plan could lead to a more economical solution for the entire design life. In cases where a structural overlay is projected, the overlaid pavement system must be evaluated for the remainder of the design life in much the same manner as the initial pavement system is evaluated for the proposed service life. All costs incurred during the design life must be considered in comparing alternatives.

To make the process clear, an example is presented of the kind of analysis that must be used in order to estimate the effect of design CBR and service life. The example consists of the design of a 12,000- by 150-ft section of airfield. The area of the section of pavement is 200,000 $yd^2$, and it is considered a B traffic area. The same materials will be considered in every design. These materials are described in Figure 5.

The design life considered is 20 years, with the following traffic: 50 passes/day, Boeing 747, and 300 passes/day, Boeing 707.

Three overall plans are considered for maintaining the pavement system for its design life:

1. Initial construction designed to last the design life of 20 years before initial failure;
2. Initial construction designed for a service life of 8 years before initial failure (an AC overlay is scheduled at that time and designed to last through the design life of 20 years); and
3. Initial construction designed to last a service life of 12 years before initial failure (an AC overlay is scheduled for that time and designed to last through the design life of 20 years).

The procedure used is to determine, for each of the plans considered, all the costs expected to be incurred in construction and maintenance over the design life of 20 years. The cost categories to be considered are construction, structural overlays, routine maintenance, and reconstruction of prematurely failed areas. Costs for construction are simply added up from the unit costs shown in Figure 5. Costs of structural overlays are estimated at \$0.60/$yd^2$/in. Routine maintenance costs are taken to be a function of the length of time the pavement must be maintained without benefit of structural overlays. For the purpose of the example, the relationship shown in Figure 6 was assumed.

For plan 1 the routine maintenance cost for the section of pavement considered is $200{,}000 \times \$2.50 = \$500{,}000$. This results in an average annual cost of \$0.125/$yd^2$/yr.

Costs for reconstruction of failed areas are computed as a combination of the service life assumed and the design CBR used. For the sake of simplicity, the procedure was as follows.

Figure 4. Distribution of subgrade CBR.

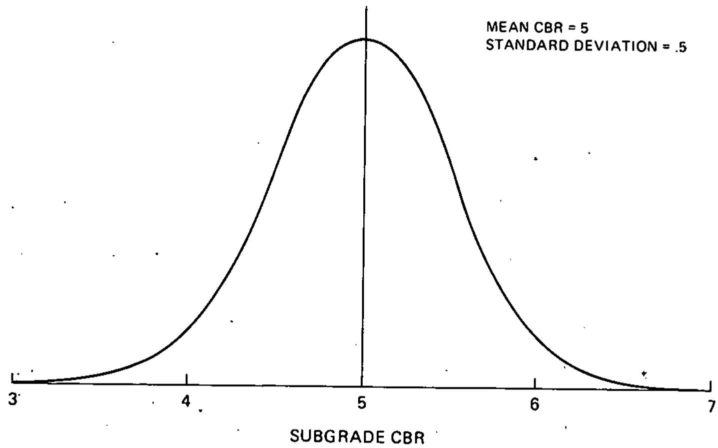



Figure 5. Material properties and unit construction costs.

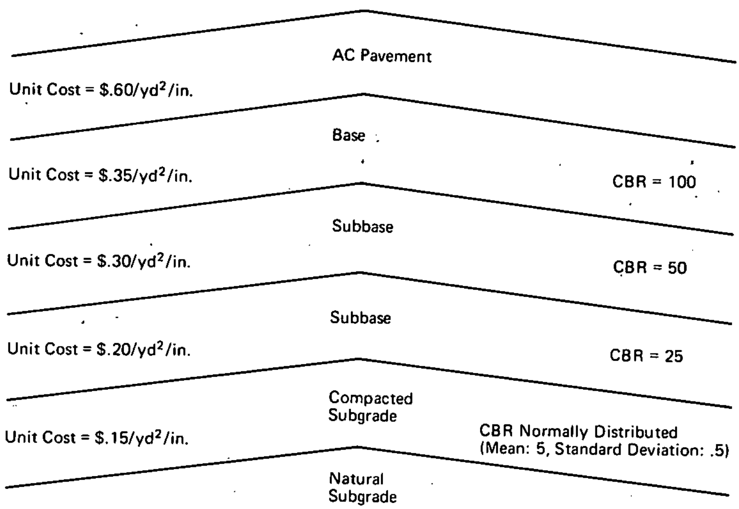
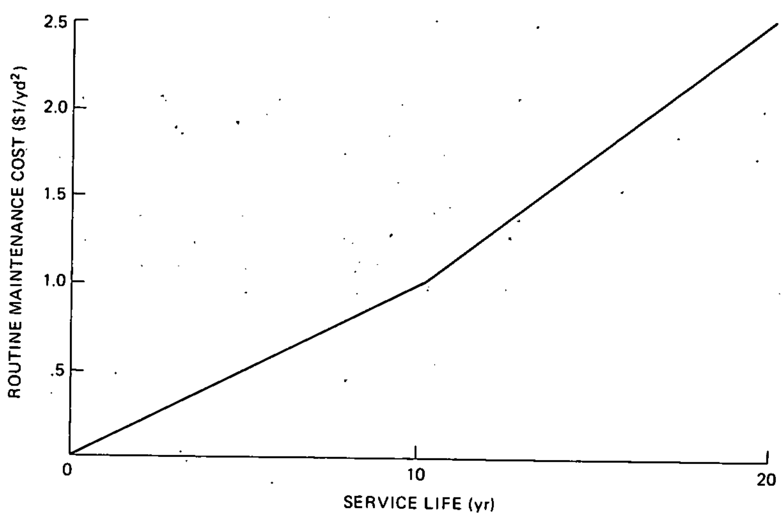



Figure 6. Routine maintenance costs.



1. For a given service life and design CBR, find the required total thickness of pavement. This includes rounding thicknesses off and meeting the specified minimum thickness requirements.
2. Using the thickness obtained in step 1, determine the CBR value that will cause that thickness of pavement to fail during the first third of its service life, during the second third of its service life, and during the remaining third of its service life.
3. Using the assumed probability distribution (based on engineering judgment and on all the sources of variation discussed above) and the CBR values from step 2 above, determine the percentage of the total area expected to fail in the designated time periods.
4. Set the cost of reconstruction as follows: equal to the construction costs for areas failing in the first time period; equal to two-thirds of the construction costs for areas failing in the second time period; and equal to one-third the construction costs for areas failing in the third time period. Determine the cost of reconstruction up until the specified service life. (In considering the time period from the time of a structural overlay to the end of the design life, include the thickness and cost of the overlay.)
5. Add the costs incurred during the initial period before a structural overlay to the costs incurred after the structural overlay to determine the total reconstruction costs.

To illustrate this procedure, the results for the design analysis of a pavement with a mean CBR of 4 and a service life of 12 years are given in Table 1 and shown in Figure 7.

To these costs must be added the cost of the overlay (200,000 × $0.60 × 4 = $480,000) and the cost of routine maintenance both before overlay (200,000 × $1.30 = $260,000) and after overlay (200,000 × $0.8 = $160,000), which totals $420,000, to arrive at the total maintenance and reconstruction costs for the 20-year design life. This gives a total cost for maintenance and reconstruction of $924,260. The cost of the initial construction is $3,160,000. The total life-cycle cost thus becomes $4,084,260 or $20.42/yd$^2$.

If this procedure is repeated for a range of CBRs consistent with the variation assumed and for the three maintenance and reconstruction plans, the total costs will provide the information necessary to choose an alternative that is most cost-effective. Naturally, for given soil conditions, as lower CBR values are used for design, maintenance and reconstruction costs become smaller but the construction costs are larger. Conversely as higher CBR values are used for design, maintenance and reconstruction costs become larger and the construction costs smaller. The design CBR will influence the economic ranking of the three plans on a cost-effective basis.

There are limitations in the procedure used in the example. Only one type of overlay plan was used, and only one design type and choice of material and layer combinations were considered. Finally, the calculation was done at discrete points at fairly wide spacing. In spite of these limitations, the results show the trends that can be expected and demonstrate that a cost trade-off exists and should be considered during design.

The designs for the different service lives were calculated by using a computer system LIFE 1, which performs life-cycle design and maintenance calculations for airfield pavements based on the design methods of the Corps of Engineers. The system has more capability for considering overall plans for maintenance and reconstruction for a design life than displayed in the example. However, the stochastic nature of the reconstruction requirements, due to the variability of many design parameters, is not yet taken into account in the LIFE 1 program. When these capabilities are included, this system should allow the designer to make choices such as the ones described herein and, in doing so, to consider a much richer variety of alternatives.

Results from analyses described above for the three proposed plans and for several CBR values in the range from 3.5 to 5.0 (3 standard deviations) are shown in Figure 8. The cost trade-off is quite pronounced in terms of design CBR and is clear for the maintenance and reconstruction plans at reasonable design CBRs. For the example

Table 1. Costs incurred before and after overlay.

| Item | Before Overlay: Service Life | Before Overlay: Second Time Period | Before Overlay: First Time Period | After Overlay: Remaining Life | After Overlay: Second Time Period | After Overlay: First Time Period |
|---|---|---|---|---|---|---|
| Time period, years | 12 | 8 | 4 | 8 | 5.33 | 2.67 |
| Failure CBR[a] | 3.86 | 3.78 | 3.64 | 3.46 | 3.39 | 3.22 |
| Percentage of area failed[b] | 0.40 | 0.40 | 0.33 | 0.1 | 0 | 0 |
| Costs for each area[c], dollars/yd$^2$ | 5.27 | 10.54 | 15.80 | 6.07 | 12.14 | 18.20 |
| Reconstruction cost[c], dollars | 4,210 | 8,450 | 10,400 | 1,200 | 0 | 0 |
| Total cost, dollars | 23,060 | | | 1,200 | | |

Note: Pavement thickness (step 1) is 61 inches; AC overlay is 4 inches. Total reconstruction cost is $24,260.

[a]Step 2. [b]Step 3. [c]Step 4.

Figure 7. Illustration for service life of 12 and design CBR of 4.

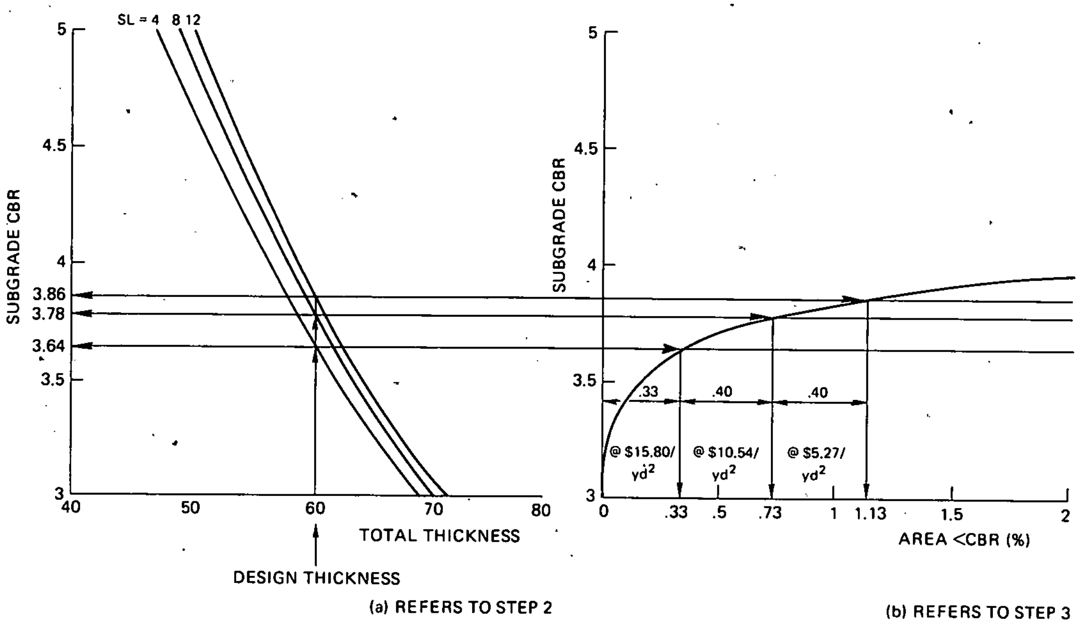


Figure 8. Cost trade-off trends.

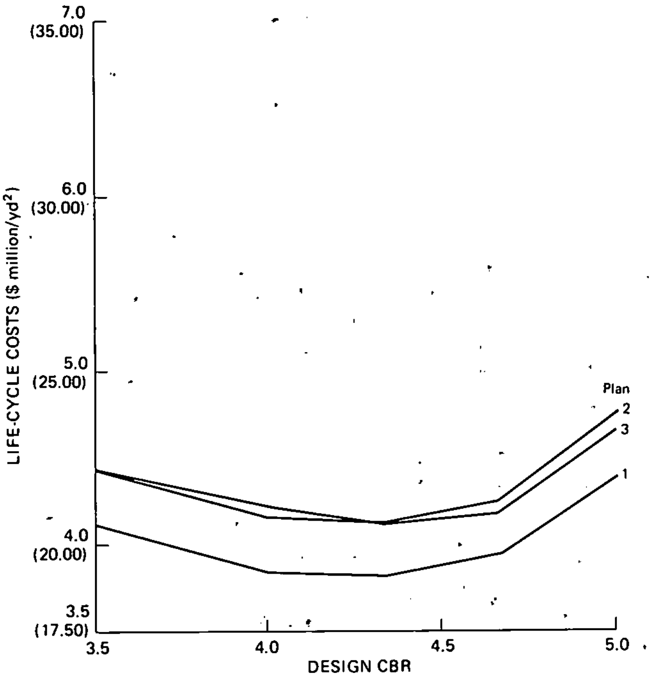
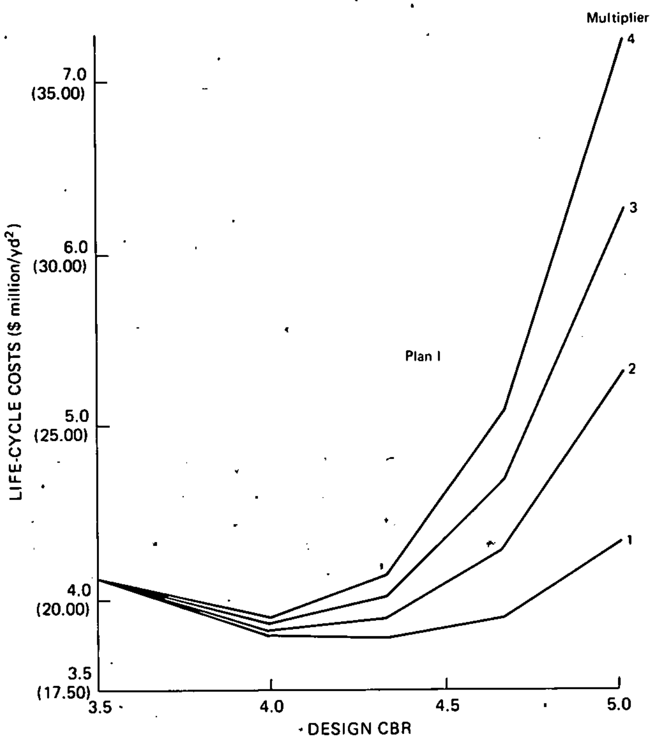



Figure 9. Effect of reconstruction unit costs.



used, the most cost-effective approach is to design for 20 years (plan 1) with a design CBR of approximately 4.25 (1.5 standard deviations below the mean). It is interesting to note that, if one were to seemingly play it safe and use a design CBR of 3.5 (3 standard deviations below the mean), the resulting design would be uneconomical because the increase in initial construction costs more than offsets the estimated maintenance and reconstruction costs. Correspondingly, if the mean value were used as the design CBR, maintenance and reconstruction costs would outweigh savings in the initial construction costs (this assumes that about 50 percent of pavement area will require reconstruction or extensive maintenance during the service life).

The assignment of costs for reconstruction of failed areas is quite arbitrary in the example used. With the idea that reconstruction probably costs considerably more per unit area than initial construction, Figure 9, based on plan 1, shows several cost trade-off curves with multiples of the reconstruction costs used in the example. Naturally, as a higher multiplier is used, the effects of distressed areas are more pronounced and consequently the cost optimal design CBR values are lower. Figure 10 shows all three plans with a reconstruction unit cost multiplier of 4.

To illustrate the sensitivity of the results to the standard deviation of the CBR distribution, assumed to be effective at the time of construction, the results for plan 1, with a multiplier of 1 on the reconstruction unit costs, are shown in Figure 11 for several standard deviations. The mean is assumed to remain at 5, whereas the standard deviation is varied from the 0.5 used in the example to 0.8 and 1.0. It is clear that, as the distribution bands broaden (higher standard deviations), the cost-effective design CBR decreases. However, if the design CBR is expressed in terms of the mean value minus a multiplier of the standard deviation, it is evident that in each of the cases shown the most cost-effective design CBR is approximately equal to the mean minus 1.5 standard deviations.

Naturally the results of such an analysis as presented herein only account for part of the interactions of parameters that exist in a pavement design situation. In addition, assumptions have been tacitly made that

1. The design procedures used are perfectly adequate for determining pavement structural life.
2. An accurate cost estimate can be arrived at by using as elementary an approach as is used in the example (note, however, that the analysis used need only be accurate enough to distinguish between alternatives).
3. The design engineer, looking ahead to the anticipated conditions of construction and operation of the pavement system, can adequately estimate such things as the pattern of routine maintenance expenditures required to keep the system functional and the distribution of CBRs expected to be effective at completion of construction.

## CONCLUSION

The philosophy presented can be expressed very simply as "look at the consequences of design decisions." Often designers try to design pavements that are inexpensive and that are expected to "last forever," without consideration of the resulting maintenance and reconstruction costs. Clearly pavements have finite and widely varying service lives that will affect these costs. Thus, designers should consider the consequences of the variability of design parameters in choosing design values. In addition, it is wise to schedule structural overlays ahead if they are a part of the economic life-cycle cost for the pavement system.

It is evident from discussion and example that at least a rough cost trade-off analysis must be performed before design values can be chosen that will lead to a good design. It is just as wrong to use too low a design CBR as it is to use too high a value, though

**Figure 10. Cost trade-off trends with a multiplier.**

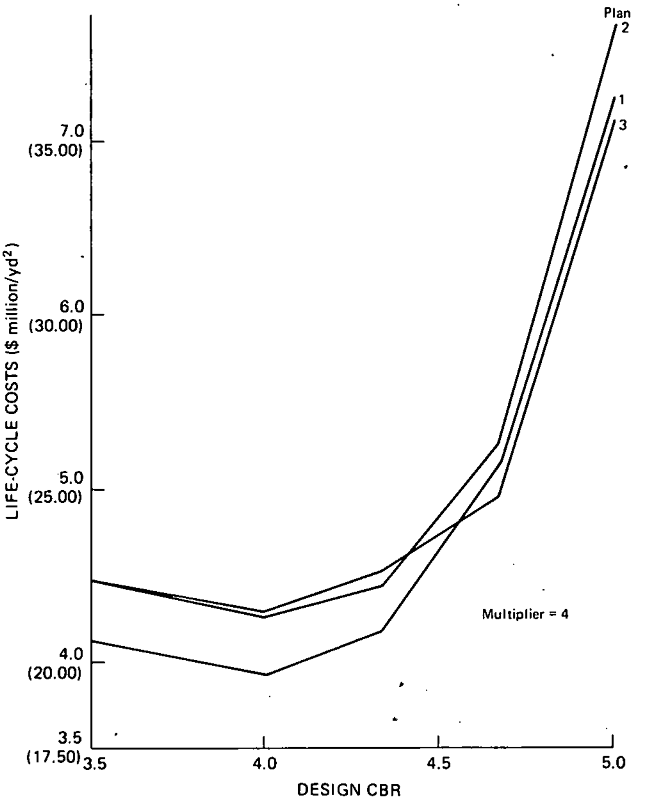
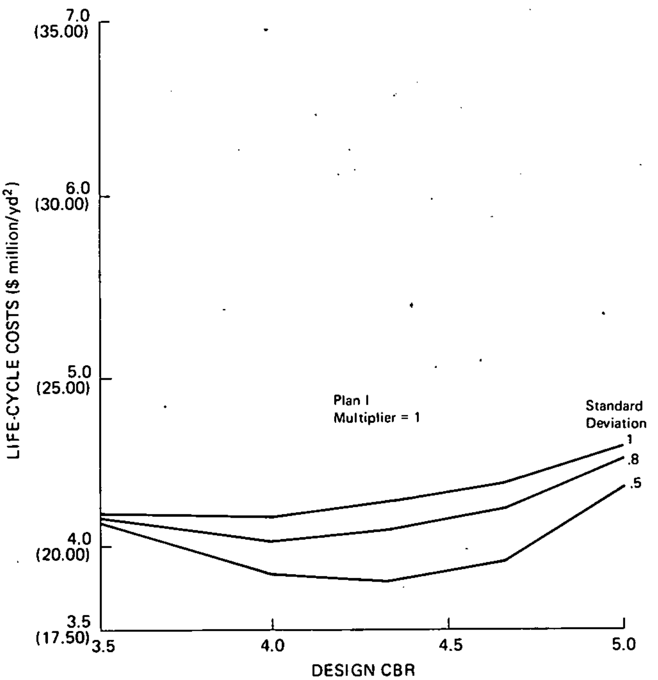



**Figure 11. Sensitivity to assumed standard deviation.**



perhaps less embarrassing in terms of the subsequent failure rate. The only way to play it safe in pavement design is to make the most comprehensive analysis possible of the appropriate cost trade-offs.

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## discussion

Eldon J. Yoder, Purdue University

The paper by McManus and Barenberg, in which they present the matter of variability of subgrade strength values, discusses concepts also presented by this writer (5).

This writer agrees with the authors' philosophy: "look at the consequences of design decisions." In the paper on soil strength values mentioned above a generalized solution to the problem of looking at the trade-offs is presented. The paper is based on soil test data from a wide variety of physiographic units both in the United States and in other countries.

It is the writer's belief that the variability values used by McManus and Barenberg may be somewhat low. For example, in the paper by Hampton, Yoder, and Burr (6), the data show that the coefficient of variability of CBR data may be as high as 40 percent for highly variable soils. This serves to illustrate the critical nature of selecting soil strength values that account for initial, road user, and maintenance costs (both routine and major).

Another point of interest is shown in Figure 9 of the McManus and Barenberg paper in which they present multipliers to the cost data. In the writer's paper on soil strength values, this technique was used and was called the "cost ratio." The need for applying a multiplier to unit costs is important in remote areas where cost of mobilization of maintenance crews can be extremely high. This need is also apparent in highly urbanized areas where added road user costs resulting from shutdown of the facility are high.

In summary, the writer agrees with the authors that soil variability is a complex problem that should be evaluated on an individual basis. However, and as illustrated in the writer's paper, guidelines can be set that are useful to the design engineer. The trade-offs, incidentally, as illustrated in the reference paper are a function of traffic, climate, and a wide variety of factors.

### References


5. Yoder, E. J. Selection of Soil Strength Values for the Design of Flexible Pavements. Highway Research Record 276, 1969, pp. 1-13.
6. Hampton, D., Yoder, E. J., and Burr, I. W. Variability of Engineering Properties of Brookston and Crosby Soils. HRB Proc., Vol. 41, 1962, pp. 621-649.

## closure

The authors wish to thank Yoder for his discussion of the paper. Yoder's earlier work in this area (5) was an inspiration to the authors to undertake such an analysis, but reference to his work was inadvertently omitted.

The authors agree that the variability values used in the paper are lower than would be expected from field data. At the AASHO Road Test, for example, the coefficient of variation for CBR values just before paving was between 40 and 45 percent compared with the 10 percent used in the paper. Less controlled construction sites could be expected to have even higher values.

The lower value for the variability used in the paper was justified on two criteria. First a CBR test represents only a very small area, an area much too small to influence the behavior and performance of pavement systems, especially airfield pavements. Thus, an attempt was made to select a variability that reflected areas large enough to have a significant influence on pavement behavior. Although the extent of such an area is not known, it was assumed that the variability representative of such areas would logically be much smaller than variability of individual test points. Second, the authors were describing a procedure and were not necessarily trying to define prototype conditions. Furthermore, the authors were trying to operate with data that would produce a relatively low variability, for it was believed that, if materials with high variability were used in the example, the results would be such that many engineers would simply not accept the numbers produced and would subconsciously reject the approach presented. Thus, because some judgments were required to select the appropriate input values, an attempt was made to deliberately keep these values in a range to produce relatively moderate variability in the final result. The authors agree with Yoder that these results, low though they may be, indicate the critical nature of selecting values for engineering properties, not only for the subgrade, but for all paving materials.

The authors also agree with Yoder's final comment; that is, the trade-offs are a function of many factors such as traffic and environment. In many areas, because of traffic problems, the indirect cost of closing a facility for maintenance will far outweigh the direct maintenance costs. These factors should all be included in the trade-off analysis when appropriate values for paving materials are selected.

# part 2

# concrete pavement construction

# construction of the transportation research center test track

Woodrow J. Anderson
Ohio Department of Transportation

At the Transportation Research Center, located on 8,100 acres approximately 50 miles northwest of Columbus, Ohio, a test track was designed and constructed. The purpose of this paper is to describe the relevant and unique design and construction details of the base materials and portland cement concrete placed on the 7.5-mile-long loop of the test track. The paper also relates construction details and problems that eventually led to changes in the contract design.

•The Transportation Research Center was created by the Ohio General Assembly to permit extensive full-scale testing of both transportation hardware and novel ideas in the field of transportation. Details of the design and construction of the TRC test track follow.

## DESIGN

### Geometrics

The main loop of the TRC test track is 7.5 miles long, and the width ranges from 36 feet along the tangents to more than 47 feet in the curved areas. The tangents and curves are 1.89 and 1.86 miles long respectively (Fig. 1). The tangents are sloped to the inside at $\frac{3}{16}$ in./ft for the full width. The transition between the tangents and curves is accomplished with 2,300 feet of spiral. To accommodate design speeds approaching 140 mph required that extreme superelevation and a safety lane be provided in the curves. The degree of curvature for the end loops is 2 deg 23 min with superelevation in the middle of the curve varying from 0.127 at the low side of the pavement to 0.800 at the outside of the safety lane (Fig. 2). Because of the design, pavement surface in the curves appears to be concave or parabolic.

The track was designed with a longitudinal gradient of 2.28 feet per 1,000 feet, a differential in elevation of only 23 feet between the north and south extremes of the straightaways. The embankment in the loops provides nearly 18 feet of elevation variance between the outside edge and the corresponding inside edge.

### Pavement

Pavement design criteria for the large loop and adjacent skid pad area far exceeded requirements of conventional PCC pavement design for public highways. The design single-axle load was 32 kips and 48 kips for tandem. The original design provided for a 10-in. plain PCC pavement with a 4-in. cement-treated base (CTB) in addition to a 4-in. aggregate subbase (Fig. 3). Pipe underdrains were provided at the inside edge

Figure 1. Schematic of large loop of test track.

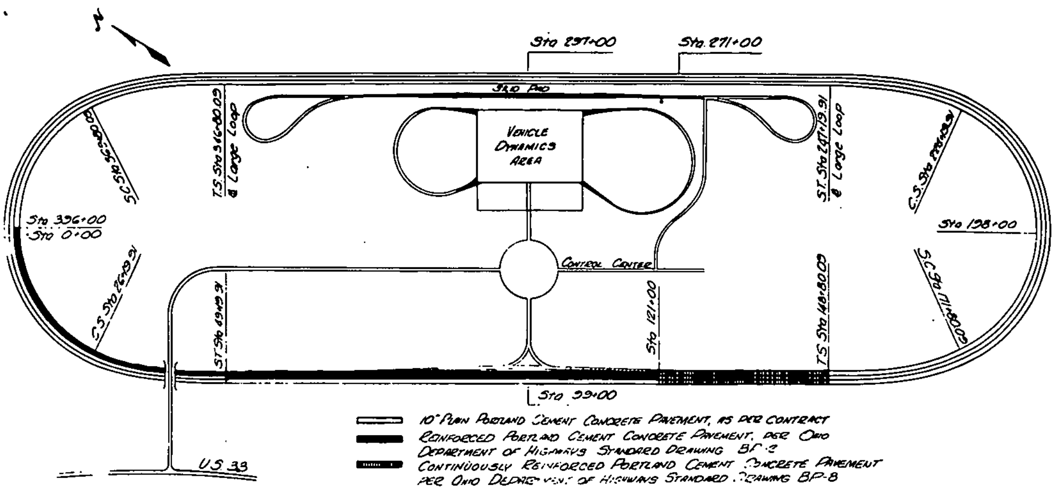
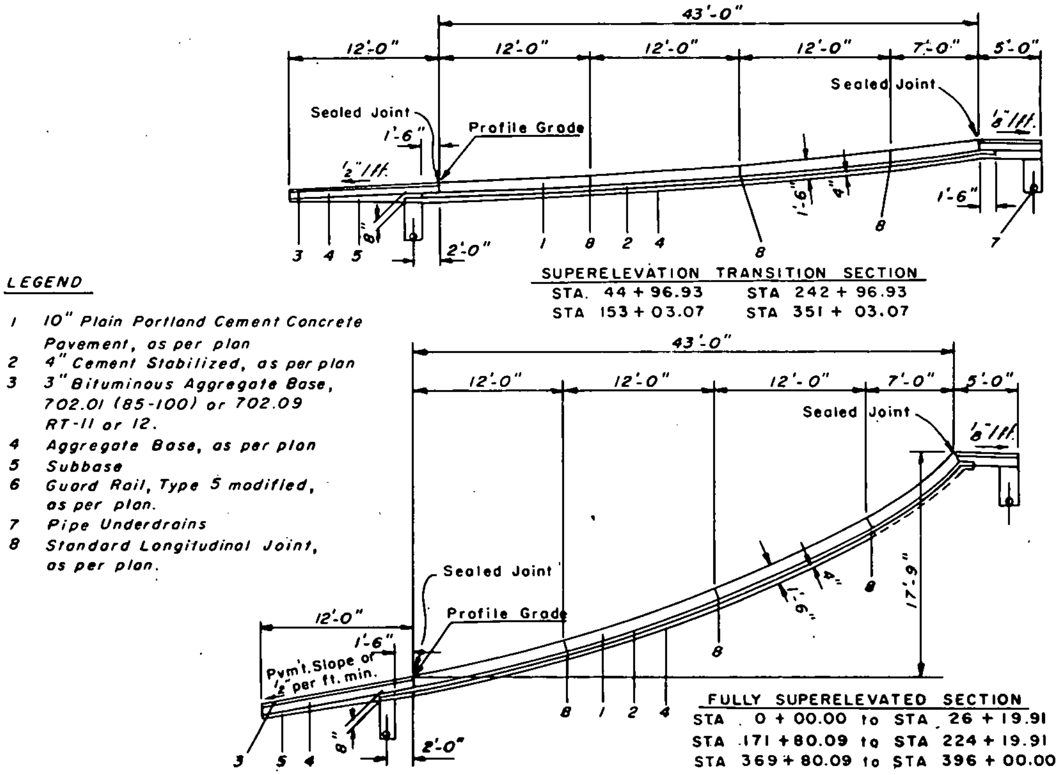



Figure 2. Superelevation of large loop.



of the pavement and on the outside edge in areas where excavation dictated and at the direction of the engineer.

The concrete pavement was designed with an 18-ft joint spacing. The design called for jointing of the slabs with 1¼-in. plastic-coated dowels. The ¼-in. contraction joint was sawed 2 in. in depth and sealed with a neoprene compression seal. Although subsequently changed, the original design of the longitudinal joint called for a premolded nonmetallic keyway to be placed for multilane construction or a formed keyway between separately placed lanes. This design required 27- by ⅝-in. deformed tie bars to be placed at middepth along 48-in. centers. The ¼-in. longitudinal joint also used a similar but smaller compression seal. As an aid in keeping water from entering between the pavement edge and the 3-in. bituminous aggregate shoulder, a liquid joint sealer was specified. For this purpose, the design specified a joint not less than 2 in. deep with a minimum width of ¼ in. in which a conventional hot- or cold-applied joint seal would be placed following pavement and shoulder construction.

As stated previously, the original design specified that the large loop be constructed with unreinforced, doweled PCC pavement. Subsequently, however, TRC officials recommended that a segment of the large loop be constructed with varying degrees of reinforcement. For several years Ohio State University has been researching and developing a prototype automobile with an electronic guidance system. In the operational phase of development of this automated vehicle, it became apparent that the degree and location of reinforcement could affect its performance. With this in mind, we decided to use the test track facilities to conduct further research and tests of the vehicle under varying pavement design techniques.

Ultimately it was agreed to incorporate three variations using reinforcement (Fig. 1). On one test section in the north curve for 4,920 feet, conventional 54-lb mesh would be placed on the CTB on the 12-ft inside lane. The second test area included the adjacent west tangent in which the same reinforcement would be placed within the PCC pavement on the inside 24 feet of roadway for a length of 7,180 feet.

With this arrangement the 24-ft pavement was constructed with transverse contraction joints placed at 36-ft intervals. The adjoining and tied 12-ft lane was built according to the original design. The third test section included the remaining segment of the west tangent. The design for this 2,780-ft segment called for the placement of 10-in. continuously reinforced concrete (CRC) pavement for the full width of 36 feet. The CRC pavement incorporated deformed wire fabric amounting to 0.612 percent of the pavement cross section.

## CONSTRUCTION

### Subgrade and Subbase

The subgrade was constructed in accordance with currently available construction techniques. The soil subgrade was prepared in accordance with the Ohio Department of Transportation's (ODOT) Construction and Material Specifications. Soils with a minimum laboratory dry weight of 100 lb/ft$^3$ were required. The subgrade was compacted to a depth of 12 in. with a density stipulation that compaction be not less than 100 percent of maximum dry density as determined by AASHO T-99.

The subgrade on the tangents was trimmed by using combinations of a CMI Autograde and a motor grader. To obtain the desired cross section required in the grading contract, we used a D8 dozer and grader. The grader, supported by the dozer, made successive passes down the slope and trimmed the surface to grade (Fig. 4). The contractor tentatively plans to use a Gradall to cut the final subgrade. On the curves the surface of the final subgrade and base will be finished to a series of chords rather than the more typical curved template. The contract states that all specified minimum thicknesses must be met, and any additional base or concrete required for this procedure will be furnished at no additional cost.

A 4-in. aggregate base was placed and compacted on the soil subgrade. This material met the following grading requirements:

| Sieve Size | Percentage Passing |
|---|---|
| 2 inch | 100 |
| 1 inch | 70-90 |
| ¾ inch | 50-85 |
| No. 4 | 25-60 |
| No. 40 | 7-30 |
| No. 200 | 0-10 |

Crushed gravel containing a minimum of 90 percent fractured pieces was the aggregate.

At the beginning of the compaction operation a test section was placed to establish the maximum density of the material to be used. With the granular material at or near the predetermined optimum moisture, the degree of compactive effort was established. Thereafter, the remaining material to be placed was compacted to not less than 98 percent of the test density. The maximum dry weight for the material used was 129 $lb/ft^3$.

On the west tangent the contractor placed the subbase for a width of 28 feet and then followed this with a 12-ft add-on section by using a conventional dozer-attached spreading box. The box spreader was used solely on the east tangent. The subbase was compacted with a vibrating roller and then trimmed with a CMI and grader. The finished surface of the completed subbase was required to fall within a ⅜-in. tolerance after the grade was checked with a 10-ft straightedge applied parallel to the centerline.

## Cement-Treated Base

The immediate underlying support for the PCC pavement was a 4-in. blanket of cement-treated base. The CTB specification required that the aggregate consist of durable particles of limestone, slag, gravel, or sand and be free from injurious amounts of shale, clay lumps, and organic impurities. Pit-run or crushed-run material, commercially sized materials, or a combination was permitted. The material was required to contain a minimum of 25 percent between the No. 10 and No. 200 sieves, and the plasticity index could not exceed 12. The aggregate proposed for use was well graded and had a high percentage (90 percent and above) of fractured pieces and a low percentage passing the No. 200 sieve.

Table 1 gives the grading requirements along with a typical grading reported by the ODOT testing laboratory.

Based on the materials to be used, our laboratory determined the quantity of cement that would be required to obtain a minimum strength of 400 psi in 7 days. The laboratory specimens conformed to the size designated in AASHO T-134. Although it was possible to obtain the desired strength with less than 5 percent cement by weight, it is an established procedure not to reduce the cement content below this figure because of the large number of freeze-thaw and wet-dry cycles encountered in Ohio.

The laboratory determined the density of the combined cement aggregate mixture to be 136.6 $lb/ft^3$ with an optimum moisture of 8.6 percent. The specification for the CTB required that, after the mixture had been placed, it be compacted to 95 percent or more of the maximum laboratory density as determined in accordance with AASHO T-134, Method B. The combination of cement, aggregate, and water could not be left undisturbed for more than 30 minutes. In addition, at the start of compaction, the percentage of moisture in the aggregates based on oven-dry weights was not to be below optimum nor more than the amount that would cause the CTB to become unstable during compaction and finishing. Compaction was to be completed within 2 hours after water was added to the mixture.

In addition to using the specially designed and fabricated Gunnert-Zimmerman slip-form equipment to place the concrete pavement, the contractor wanted to use this same equipment for placing and compacting the CTB in one operation. This procedure would ultimately lead from a minimum placement width of 39 feet to a maximum of 50 feet in the curves.

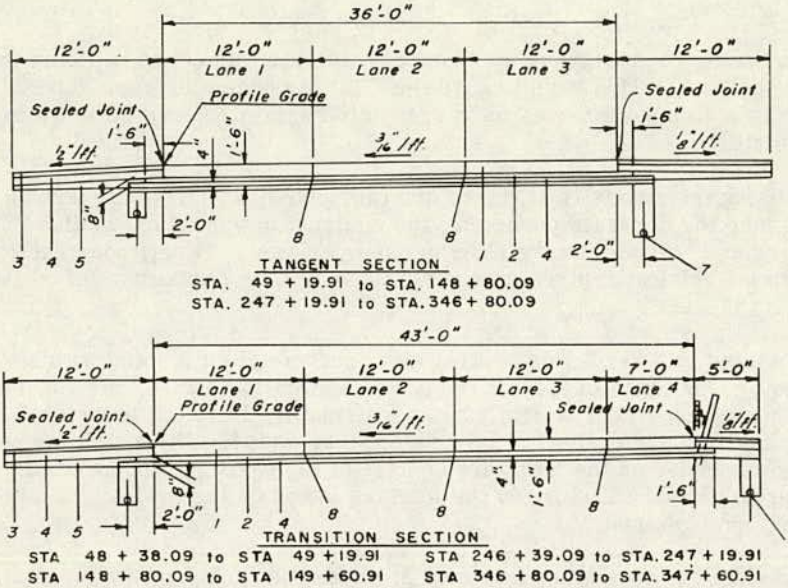
The CTB was mixed in a 9-yd$^3$ Rex central mix concrete plant located within ½ mile of the west tangent. One batch of plant mix yielded approximately 14.5 tons of total mixed materials. The aggregate dry weight was approximately 24,700 lb to which 1,240 lb of cement and 3,500 lb of 420 gallons of mix water were added. The actual amount of water required depended on the moisture content of the aggregate being used. As one can see the percentage of moisture in the mixture based on the dry weight was quite high. This will be explained.

As noted previously the contractor had wanted to use the Gunnert-Zimmerman Slip-form equipment to place the CTB. On the first day of placement, many possibilities and few reservations about the capabilities of the slip-form procedure in general were revealed. It was the intent to place the CTB at the lowest possible moisture content and yet obtain the desired density. Although the plant was capable of producing a continuously uniform mixture at or slightly above optimum, the paver could not compact a properly finished surface. The density could be obtained, but the surface was beset with irregular deep striations and an unacceptable surface. A Gunnert-Zimmerman representative who had operated this equipment for similar base placement elsewhere noted that successful performance could be obtained with higher moisture levels.

After several hundred feet and an unsatisfactory end result the moisture content of the mixture was increased. The desired workability was obtained by increasing the percentage of moisture to approximately 14 percent. The adjusted mix was closely observed to determine the effect of higher water content on density. As expected, the field tests showed a notable reduction in density as moisture increased. Although density was reduced, the minimum requirements of 140.9 and 129.8 lb/ft$^3$ for wet weight and dry weight were obtained. Because of the increased water-cement ratio, there was concern about obtaining the ultimate design strength. The resulting CTB appeared plastic and in effect had the consistency of low-slump concrete. Because of the nature of the mixed materials, cylinders were cast to gather information on compressive strength. Conventional 6-in. concrete cylinder molds were used in lieu of the standard 4- by 4.5-in. specimen. Cylinders were cast and then broken at 1-, 4-, 5-, and 8-day intervals (Table 2). The initial breaks after 1 day of cure appeared low; but, given the low cement factor (approximately 2 sacks/yd$^3$), early handling of specimens, and large L/D ratio, high early strengths were not expected. The cylinder strengths following moist curing over a weekend nearly doubled, which was encouraging. The averages for four cylinders at the 4- and 5-day intervals were 172 psi and 191 psi. Two cylinders broken after 8 days of cure averaged 218 psi.

The plastic CTB was transported to the test track site in triaxle dump trucks in batches of about 7.5 yd$^3$. The CTB was discharged onto a belt spreader attached to the slip-form equipment (Fig. 5). Five sensors, operated by two taut wire lines, provided the horizontal and vertical controls for the paver. The spreader was designed to place the base for the full width. The base in the tangent is 39 feet wide and widens to approximately 50 feet in the loops. The mixture was deposited uniformly in front of the paver, consolidated with spud vibrators spaced at 18-in. centers, confined, and then extruded. No oscillating screeds were used in the finishing process. The extruded CTB was dense and homogeneous and had the appearance of a neatly placed and well-formed PCC pavement slab

Figure 3. Pavement design.




LEGEND

1 10" Plain Portland Cement Concrete Pavement, as per plan.
2 4" Cement Stabilized Base, as per plan.
3 3" Bituminous Aggregate Base, 702.01 (85-100) or 702.09 RT-11 or 12.
4 Aggregate Base, as per plan.
5 Subbase
6 Guard Rail, Type 5 modified, as per plan.
7 Pipe Underdrains
8 Standard Longitudinal Joint, as per plan.

Figure 4. Preparing rough subgrade on curve.

Table 1. Grading requirements of the CTB material.

| Sieve Size | Specified Gradation, Percentage Passing | Typical Gradation, Percentage Passing |
|---|---|---|
| 1-inch | 100 | 100 |
| 3/4-inch | — | 100 |
| 1/2-inch | — | 97 |
| 3/8-inch | — | 88 |
| No. 4 | 55-100 | 67 |
| No. 10 | 37-100 | 49 |
| No. 40 | — | 20 |
| No. 200 | 5-35 | 10 |

Table 2. Compressive strength, in psi, of cylinder specimens.

| | 5 Percent Cement | | 7 Percent Cement | |
|---|---|---|---|---|
| Age | Individual | Average | Individual | Average |
| 1 | 85 | | | |
| | 99 | 92 | — | — |
| 4 | 170 | | 375 | |
| | 173 | | 378 | 377 |
| | 170 | | | |
| | 173 | 172 | | |
| 5 | 195 | | | |
| | 202 | | — | — |
| | 184 | | | |
| | 184 | 191 | | |
| 8 | 216 | | 576 | |
| | 219 | 218 | 580 | 578 |

Figure 5. Placement of CTB.

(Fig. 6). The brown color and the 4-in. thickness were the only notable differences. Placement of the CTB reached a maximum production of 3,400 feet per day with an average daily rate of 2,300 feet. The contractor's personnel consisted of a superintendent, foreman, operator, oiler, and five laborers.

The mix design, density requirements, strength considerations, and problems in placing were resolved. However, one additional problem arose. Following placement of the first day's run, an unusual amount of transverse cracking was detected at the surface of the CTB. The CTB had to be maintained in a moist condition before the RT9 or RT10 curing material was placed. The specifications require that the tar be applied at a rate of 0.3 gal/yd$^2$ with a cover sand to be placed at a rate of 0.005 yd$^3$/yd$^2$. The cracking became apparent before the tar was applied. The cracks were irregularly spaced hairlines, 10 to 15 feet apart. Some areas depicted clumps of closely spaced cracks, 2 to 5 feet apart. The cracks did not open up and could not be detected through the curing material once it had been placed. It should be pointed out that because of the dense, satin type of finish of the plastic CTB any cracking was readily apparent. It was our position that, although the cracking might not affect the performance of the base, every effort should be made to reduce it to a minimum.

To maintain a better moist cure before the tar was applied, we decided to increase the cement content; this procedure is supported by data from the Portland Cement Association's Soil Cement Laboratory Handbook. A change from 5 to 7 percent was made. The additional cement reduced cracking immensely, and in most areas cracking could be detected only at intervals of several hundred feet. To determine the strength gain contributed by the additional cement, we cast 6- by 12-in. cylinders from the plastic CTB on the day the design was changed. Two cylinders were broken at 4 days and two cylinders at 8 days (Table 2). Despite the limited number of samples, there was good indication that the strength gain was considerably greater than the 5 percent design. After the field evaluation and examination of the test results, we decided to proceed with the higher cement factor.

To test the strengths of the CTB, we extracted two cores from the base on each of the first 3 days after the mix design was modified. The cores were approximately 3.5 in. in diameter, were cut to the full depth of the base, and were broken at 28 days. The 28-day strengths ranged from 620 to 910 psi with an average of 730 psi.

As noted previously, the specifications required that the CTB be cured and protected with the application of RT9 or RT10. Because difficulties in placing this material and the cover sand on the steep slopes of the curves were anticipated, as was the task of setting dowel baskets on the steep inclines, the contractor suggested applying a concrete curing membrane in these areas. Several years ago, on an Ohio highway project a test area of CTB was cured with white-pigmented concrete curing membrane applied at 200 ft$^2$/gal, and the results showed a 15 percent increase in strength at an age of 11 days; at 100 ft$^2$/gal, there was an increase of 49 percent. The comparison was made with adjacent areas cured with bituminous material.

Before any judgment was made on his suggestion, the contractor was asked to conduct a test installation on the skid pad area. A 365- by 39-ft test section of CTB was constructed on which the white-pigmented curing membrane was sprayed at a rate of 150 ft$^2$/gal. Subsequently, six cores were cut from the membrane-treated section and a like number from the adjacent area treated with tar. The two sections were placed on the same day, and cores were cut 7 days after placement. Three cores from each group were then broken in compression on the 7th day, and the remaining cores were placed in moist cure before breaking on the 28th day. The resulting data are given in Table 3. The compressive strengths at 7 days indicated that the membrane-cured base had a 39 percent greater strength; however, the remaining two sets of cores after an additional 21 days of moist curing revealed a noticeable change in the strength gain as compared to an 18 percent gain with the membrane-treated cores. After longer moist curing the tar-treated cores showed a comparable but greater strength average.

## Portland Cement Concrete Pavement

The concrete mix design for the PCC pavement included many items of interest. The cement factor was reduced from Ohio's standard for PCC pavement of 611 to 564 lb/yd$^3$. It was determined that the cement content could be lowered and the desired strengths would be obtained if an aggregate with a very good performance record were used. With this in mind a crushed dolomite stone from the Ohio Guelph or Greenfield ledges was specified. This coarse aggregate had a nominal top size of 1 in. and met the grading requirements of AASHO M-43, size No. 57 (Table 4). The fine aggregate met the grading given in Table 4 and, in addition, was a natural sand containing not less than 50 percent silica dioxide as determined by ASTM Designation C 146. Materials were proportioned in accordance with the American Concrete Institute's standard designation ACI 613. The concrete mixture design was based on a slump of 2 in. ± 1 in. with a water-cement ratio not exceeding 0.49. The air-entraining agent was sufficient to maintain an air content of 7 ± 1 percent.

The contractor chose a private test laboratory to design the mixes based on approved materials from the sources he had selected. During the laboratory testing and evaluation program, supplemental trial mixes were prepared that incorporated a water-reducing agent. The purpose of this testing was to determine whether the use of this chemical admixture would enhance the workability of the concrete. Of particular concern to the contractor was the ultimate task of having to place and finish low-slump concrete on the very steep circular curves. The laboratory evaluation indicated that 1.25-in. slump concrete with a water-reducing agent gave the same degree of workability as the standard concrete mixture with a 1.75-in. slump. Based on these findings the contractor elected to use a water-reducing chemical admixture meeting the requirements of ASTM C 494 for all concrete placed in the curves of the large loop.

The trial mix with a uniform slump of 1.75 in. and an air content of 7.75 percent produced 7- and 28-day compressive strengths for the conventional concrete mix of 3,870 and 5,670 psi respectively. Correspondingly, with a 1.25-in. slump, 7.75 percent air content, and a 3.7 percent reduction in mix water the admixtured concrete gave compressive strengths of 3,710 and 5,650 psi at the same respective curing periods. The trial mix design was the basis not only for establishing a proper yielding mixture but also for obtaining a good workable mix. Slight variations were made in the field to correct for minor changes in specific gravity. The typical mix design for the concrete placed in the tangents and skid pad areas is as follows:

| Material | Quantity/Cubic Yard |
|---|---|
| Type I cement | 546 lb |
| Coarse aggregate | 1,660 lb |
| Fine aggregate | 1,280 lb |
| Water | 31 gal |
| Air-entraining agent | 15 oz |
| Water-reducing admixture | 36 oz |

Immediately following placement of the CTB, the contractor made the necessary modifications in the Gunnert-Zimmerman slip-form equipment and proceeded to place the 10-in. PCC pavement. The contractor decided to first complete the base on the west tangent and then to pave that 10,000-ft section.

The west tangent included two of the variations in reinforcement. After the CTB was completed on the west tangent, reinforcement at the north end was begun. The revised plans called for placing conventional 6- by 12-ft (54 lb/100 ft$^2$) wire fabric. The reinforcement for this section of pavement was positioned on supporting continuous chairs so that it would be approximately 4 in. below the surface (Fig. 7).

Figure 6. CTB after extrusion behind slip-form paver.

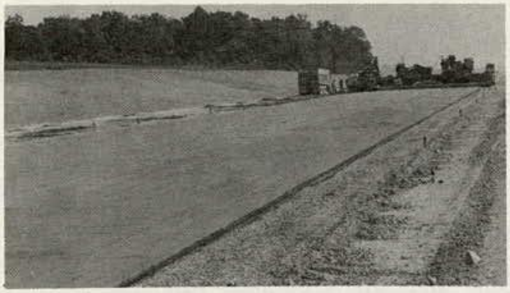


Table 4. Concrete aggregate gradations.

| Sieve Size | Percentage Passing: Fine Aggregate | Percentage Passing: Coarse Aggregate |
|---|---|---|
| 1½-inch | | 100 |
| 1-inch | | 95-100 |
| ¾-inch | | — |
| ½-inch | | 25-60 |
| ⅜-inch | 100 | — |
| No. 4 | 95-100 | 0-10 |
| No. 8 | 70-95 | 0-5 |
| No. 16 | 45-80 | |
| No. 30 | 25-60 | |
| No. 50 | 10-30 | |
| No. 100 | 1-10 | |
| No. 200 | 0-4 | |

Figure 8. Placement of concrete on west tangent.

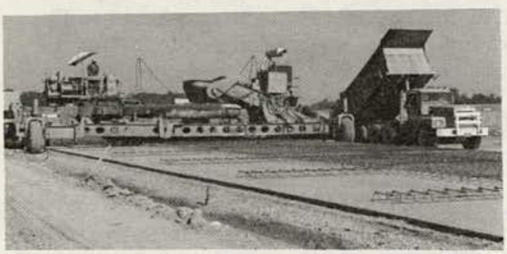


Figure 10. Placement of PCC pavement on CTB.

Table 5. Concrete test data.

| Tests | Range | Average |
|---|---|---|
| Slump, in. | 1.25-2.50 | 1.72 |
| Air, percent | 6.3-7.8 | 7.1 |
| Compressive strength, psi | | |
| Cylinders, 7 days | 3,200-5,000 | 4,100 |
| Cylinders, 28 days | 5,300-7,700 | 6,700 |
| Beams, 5 days | 660-930 | 780 |

Table 3. Compressive strength of CTB cured with RT9 and concrete curing membrane.

| Cure | Core No. | Age (days) | Compressive Strength (psi) | Average |
|---|---|---|---|---|
| RT9 | 1 | 7 | 460 | |
| | 2 | 7 | 470 | |
| | 3 | 7 | 540 | 490 |
| | 4 | 28 | 860 | |
| | 5 | 28 | 780 | |
| | 6 | 28 | 1,040 | 890 |
| Membrane | 7 | 7 | 620 | |
| | 8 | 7 | 770 | |
| | 9 | 7 | 660 | 680 |
| | 10 | 28 | 800 | |
| | 11 | 28 | 850 | |
| | 12 | 28 | 740 | 800 |

Figure 7. Supports for reinforcement, dowels, and tie bars.

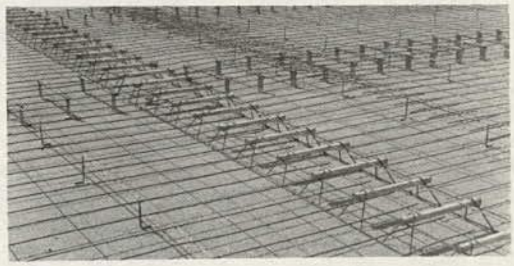


Figure 9. Nailing of supports for tie bars at 1-ft intervals.

Figure 11. Final finish: longitudinal texturing.

From the north for 7,180 feet, the inside two lanes (24 feet) required this type of reinforcement. The outside lane (12 feet) was placed according to the original plan (Fig. 8). In this length of pavement all lanes of pavement were tied together. The original plan specified not only that the lanes would be tied, but also that all adjoining lanes would require a keyway between them. It was the intent that preformed material be used to accomplish this for multilane construction. The consultant was asked to reevaluate this design because of anticipated construction difficulties in multilane placement. An alternate proposal was made. Instead of the original design, keyways could be eliminated if the tie bars were placed at 12-in. centers. A keyway with hook bolts at 30-in. spacing was used in longitudinal joints where adjacent lanes might be placed in separate operations. The contractor positioned the tie bars on chairs so that they would be at middepth in the slab. Initially each tie bar was supported by two chairs fastened to the CTB with 2-in. nails. The supports were nailed to the CTB with a compressed-air-actuated gun (Fig. 9). Problems of movement during placement of concrete caused the contractor to position the tie bars by using preassembled wire baskets.

The contraction joints in this reinforced section were spaced at 36 feet. With this arrangement the adjoining plain pavement with 18-ft joint spacing would have corresponding joints every 36 feet. The contraction joints were connected by plastic-coated dowels supported by wire baskets. The dowels were $1\frac{1}{4}$ in. in diameter and 18 in. long and were coated with a 4-mil adhesive undercoating and 17-mil surface coating of high-density extruded polyethylene plastic. This coating is expected to provide sufficient protection to prevent corrosion of the dowel bars. The dowel baskets were positioned to hold the dowels parallel to the surface and then fastened by driving $\frac{1}{2}$- by 18-in. pins through the CTB. Again it was necessary to use a compressed-air-driven tool to accomplish this task.

After sufficient lead time was obtained with the mesh, tie bar, and dowel basket installation, the contractor started placing concrete at the north end of the west tangent. The concrete was mixed in the same central plant that mixed the CTB. The 9-$yd^3$ batches were mixed for 60 seconds and then deposited into triaxle trucks with open top carriers and rear-mounted discharge chutes. After the concrete was transported to the paving site, it was discharged onto a belt spreader, which was an integral part of the slip-form equipment. One segment of the belt spreader was mobile and could move laterally across the pavement and distribute the concrete uniformly in front of the slip-form paver. The diesel-powered paver distributed the concrete with rotating augers and then consolidated the mass with vibrators spaced at 18-in. intervals for the full 36 feet of pavement width (Fig. 10). Following the high degree of consolidation the concrete was formed to the desired dimensions and extruded without the aid of any additional oscillating screeds. The resulting product required little additional hand finishing. The only powered equipment trailing the paver was a self-propelled rubber-tired bridge that was necessary for imparting the required longitudinal texture (Fig. 11). The desired texture depth of $\frac{1}{16}$ in. was obtained by dragging burlap and plastic-bristled brooms attached in combination behind the bridge. After the texturing operation, the concrete was cured with a white-pigmented membrane applied at a rate of 175 $ft^2$/gal. The personnel at the paving site on a typical day consisted of a superintendent, foreman, operator, oiler, three concrete finishers, and five laborers.

Construction of the combined plain and conventionally reinforced pavement on the west tangent required 7 days. The first 3 production days were marred with operational problems. The most recurring situation seemed to be the movement of wire fabric and longitudinal tie bars immediately ahead of the paver. As the paver moved forward with a head of concrete, the fabric tended to creep and buckle. This forced the reinforcing out of proper vertical position and across the contraction joints. This situation required that paving be stopped and corrective action be taken. This problem was overcome by driving iron pins to restrain this movement. Two rows of three pins each were driven for each mat of fabric. These pins were positioned at the end of the fabric nearest the paver; and, at laps, only the forward mat was restrained. By doing it in this fashion any slight movement of the reinforcement would not result in buckling. Once these problems were overcome, production averaged more than 1,900 feet per day excluding 1 day of shutdown due to rain.

Often at the completion of a day's run, the contractor employed a unique procedure for obtaining a construction joint. Instead of placing the conventional type of wood or metal bulkhead over the dowel basket assembly and fastening it to grade, he elected to pass slightly over the proposed construction joint with the paver and then return later in the day, saw over the midpoint of dowels, and carefully remove the concrete over that portion of the dowel assembly that would become part of the forward slab to be placed when paving resumed. As an aid in the removal, heavy polyethylene sheeting was placed over the dowels and base where concrete would subsequently be removed. It was the contractor's desire to develop this technique on the tangents to gain experience and then to employ the same procedure on the concave surface in the curves.

Following completion of the first 7,180 feet of the plain and reinforced pavement, the contractor proceeded to place the last 2,780 feet of 10-in. CRC pavement for the full 36-ft width (Fig. 1). A 14- by 10-in. wide flange beam was embedded into a 10-in. sleeper slab, and this acted as the joint between the previously placed pavement and the CRC pavement. One-in. styrofoam material was placed against the web of the beam and polyethylene at the surface of the sleeper slab to permit movement of the CRC pavement. As in the area of jointed reinforced pavement, the contractor again positioned the deformed wire fabric on supporting chairs. The requirements for the longitudinal joints remained the same, and there were no variations in construction. Placement of concrete required 2 days and proceeded very smoothly.

Immediately following completion of paving on the west tangent, the paver was moved to the east tangent and the CTB was placed. Paving commenced again in 3 weeks. The east tangent contained no pavement design variables and was constructed as plain 10-in. PCC pavement with contraction joints at 18 feet. All the original design concepts were included. With the exception of several scattered days of rainy weather, construction of the east tangent proceeded in straightforward fashion. The best day produced 2,050 feet, and the average full day amounted to approximately 1,500 feet.

Field Test Data

Table 5 gives a summary of the concrete test data taken from field and laboratory records for the pavement placed during 1972. Forty cores were taken after completion of the two tangents to determine compliance for design thickness. The cores ranged from 9.5 to 10.7 in. with an average thickness of 10.0 in. After the thickness measurements were made, the cores were capped and broken. The average age at testing was 4.1 months, and the average compressive strength was 6,600 psi. As can be seen from Table 5 the compressive strength of the cylinders at 28 days is slightly greater than that of the pavement cores broken 3 months later.

A similar relationship was determined after the concrete was sampled and cylinders and cores that were specimens from the last 2 days of paving were tested. Four pairs of cylinders and eight cores were obtained to determine the variation in strength, if any, between cylinders and cores of the same age and sampled at the same location. All the cores were extracted on the 5th day of cure and shipped to the laboratory. Four were broken on the 7th day, and the rest were moist cured for the remaining 21 days. The cylinders were brought to the laboratory 2 days prior to testing. Again one-half were broken at 7 days and the remaining at 28 days. The results revealed average cylinder strengths of 4,490 and 7,030 psi at 7 and 28 days. Correspondingly, the cores were 3,000 and 5,020 psi. These data show that the cylinders exhibited 50 percent greater strengths at 7 days and 40 percent greater at 28 days. Although past research has indicated such strength variations between cores and cylinders, there was feeling that, with the high degree of consolidation or densification induced by this slip-form equipment, this strength gap would be significantly reduced.

Because of weather and field conditions there were no roughness index data to include at the time of this report. The overall riding quality is good and roughometer data should eventually bear this out.

Because of very poor weather during the fall paving season, hope of paving the end spirals and curves was dropped. The contractor, however, was able to place the CTB for the skid pad and initiate a few days of paving in that area.

The most challenging task lies ahead when the contractor must face an assignment that seems insurmountable. Nearly 1,000 man-hours will be required to widen the form line of the slip-form paver to approximately 50 feet in order to place the 4-in. CTB and 10-in. PCC pavement on the curves with a vertical grade differential of nearly 18 feet.

# innovations in design and construction of concrete pavement for illinois tollway extension

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Based on recommendations of a 1970 engineering report, a 68.9-mile, fully controlled-access, dual-lane, 70-mph facility was proposed that would meet or exceed Interstate standards. The goals of the project, an extension of the Illinois East-West Tollway, were that the facility be of an improved design that could be constructed at a low cost and yet make use of readily available construction materials. To meet these goals, the design made allowances for (a) soil conditions along the corridor with special attention to high groundwater level and frost susceptibility characteristics, (b) ease and speed of construction, (c) safety, (d) riding comfort, and (e) expected performance and ease of maintenance. Four pavement types were examined: 10-in. reinforced pavement on permeable subbase, 8-in. continuously reinforced concrete pavement, flexible pavement, and the adopted design consisting of a 10-in. unreinforced concrete pavement on a 4-in. plain concrete base. When contracts were let, however, the contractors proposed an alternate of 14 in. of unreinforced pavement. This proved to be satisfactory for roadway traffic and provided easy maintenance and speedy construction. To date, only performance and need for maintenance remain to be tested. Soil moisture and temperature gauges have been installed at six critical soil locations to monitor long-term soil moisture-density conditions and to study frost action. As segments of the extension were completed, profilometer records were obtained to be used with future maintenance records and ridability determinations.

•As a result of enabling legislation passed in 1953, which created the Illinois State Toll Highway Commission, a 187-mile-long toll highway facility was opened to traffic in December 1958. The Commission (later changed to an Authority) was charged with incorporating the benefits of advanced engineering skill, design, experience, and safety factors into a system of toll highways to eliminate existing traffic hazards so as to prevent automotive injuries and fatalities. Additional routes were studied and recommended for incorporation into the initial phase; however, traffic reviews indicated that sufficient revenues would not be generated to justify their construction as part of the original system.

In 1969, the Toll Authority authorized an engineering report and a traffic and earning report for a corridor from Aurora to Sterling-Rock Falls. These studies indicated the generation of sufficient revenues to sustain a bond issue.

The 68.9-mile extension of the East-West Tollway (Ill-56) begins at the western terminal point of the existing east-west route near Aurora, extends in a westerly direction, runs south of the communities of Sycamore, DeKalb, Rochelle, and Dixon, and terminates about 3 miles east of the cities of Sterling and Rock Falls (Fig. 1).

The roadway proposed in the engineering report is a fully controlled-access, four-lane, 70-mph highway that meets or exceeds Interstate standards. Local road overpasses have horizontal and vertical clearances, which meet federal standards for highways and drainage structures. Both pipe and box culverts and bridges have been designed to accommodate the runoff of a 50-year design storm. The pavement for the two-lane roadway will be 25 feet wide and designed for heavy vehicle traffic.

## DESIGN CONSIDERATIONS

As consultants to the Illinois State Toll Highway Authority, we have had the opportunity to observe with a critical eye the performance of a facility that was constructed to design criteria materially different from those adopted for the extension. The performance of the original facility during the first 12 years of service under extremely heavy traffic has been outstanding.

Improvement of the roadway section, reduction of costs, and the use of readily available construction materials compatible with soils conditions prevailing in the area were our goals.

### Development of Design Study Program

To achieve these three goals required that any effort to hold costs down take place in three categories of construction expenditures: earthwork, pavement, and structures.

Cost information was carefully analyzed, and adjustments were made for the variables in the anticipated construction schedule, special processing of aggregates, anticipated increases in material and labor costs, and lack of readily available local materials.

The serviceability index method of analysis was not used although records, when available, were reviewed and considered. The final pavement section was arrived at by using data from the Asphalt Institute, the Portland Cement Association, and the Continuously Reinforced Pavement Group along with reports from the AASHO Road Test and the Highway Research Board on pavement performance.

### Traffic Considerations

The East-West Tollway extension is within a corridor now served by outmoded highways, US-30 and Ill-64, -72, and -38. The cities of Sycamore, DeKalb, Rochelle, Dixon, Rock Falls, and Sterling have demonstrated above-average growth, and this increased population must rely on an antiquated highway system that cannot be substantially improved for a number of years.

In 1969, Wilbur Smith and Associates made a traffic study and prepared a report containing projections of traffic volumes, vehicle classifications, and revenue for the proposed facility (6). Vehicle classification counts were made for a 24-hour period, and origin and destination surveys were taken at five stations that made up the major traffic screenline.

Passenger cars were, by far, the largest vehicle category recorded at each of the survey locations. Overall, passenger cars and two-axle, four-tire trucks accounted for 83.9 percent of the total traffic passing through the survey stations; large commercial vehicles accounted for the remaining 16.1 percent. A breakdown of the five truck or commercial categories indicated that five-axle vehicles accounted for 53 percent of the total truck traffic through a survey station. Two-axle, six-tire vehicles made up an additional 20.4 percent, followed by four-axle vehicles with 14.7 percent and three-axle vehicles with 11.9 percent.

Soils

The proposed alignment for the East-West Tollway extension passes through an area consisting principally of Pleistocene glacial material with minor amounts of preglacial material. These soils have a wide range of physical properties depending on the nature of their deposition and the length of time they have been acted on by the weathering processes.

The materials found in the upper soils strata are mainly A-4, A-6, and A-7 soil types derived from the Sterling, Argyle, Esmond, Lee Center, and Tiskilwa glacial tills (Table 1). These materials are silty and highly frost susceptible. Only in the eastern and western portions of the project are underlying deposits of sand and gravel found. Throughout the central portion of the corridor, water tables are high and drainage is relatively poor. Spangler and Pien (4), investigating the frost susceptibility of glacial soils in Illinois and Iowa, consistently observed a capillary rise of 40 in. or more, a 45 percent strength reduction during thaw, and at least 2 in. of heave when frozen.

These soils conditions cause wet, unstable cuts both at grade and in the backslopes, excessive settlement of embankments, and below optimum compaction in embankments.

## TYPICAL SECTION STUDIES

Inasmuch as pavement, subbase, and shoulders are closely allied, studies of typical sections should include all three items. It was decided that each typical pavement section to be studied would be designed for soil conditions generally prevailing within the route corridor and that each design would be as structurally equal and as capable of handling the load repetitions expected for the facility as possible.

Given structural equality and equal life expectancy with respect to load repetitions, factors of importance are

1. Expected performance and maintenance considerations,
2. Availability of materials,
3. Initial cost,
4. Speed and ease of construction,
5. Safety, and
6. Riding comfort.

It was our contention that all the types studied, if constructed in strict accordance with specifications, would provide acceptable standards of safety and comfort; thus the choice was narrowed to the first four factors. Because of the interrelationship of these factors, it was not possible to make clear separations between them in our studies, so, for each typical pavement section studied, all were considered.

Eight separate pavement designs were considered in the study. However, because of various combinations, these fell into four types as follows:

1. Ten-inch reinforced concrete placed on a 4-in. granular subbase underlain with 6 in. of select material. The shoulders were 3-in. bituminous concrete. (This is the typical section used on the original facility except for a reduction in thickness of the select material layer.) This design was also studied with a 4-in. stabilized subbase.
2. Eight-inch continuously reinforced concrete on 4-in. stabilized bituminous subbase and also on 4-in. granular subbase underlain with 6 in. of select material. Various shoulder widths were studied.
3. Flexible pavement 22 in. thick consisting of $1\frac{1}{2}$-in. surface course, 2-in. binder course, and $5\frac{1}{2}$ in. of deep strength asphaltic concrete on 13 in. of granular subbase, stone, and select material. The shoulders were 7-in. bituminous aggregate mix.

Figure 1. Location map.

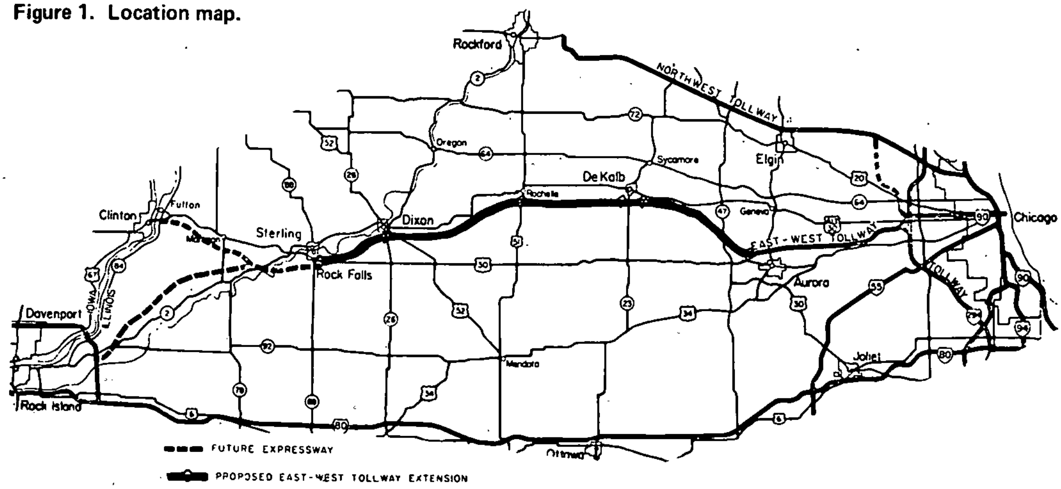
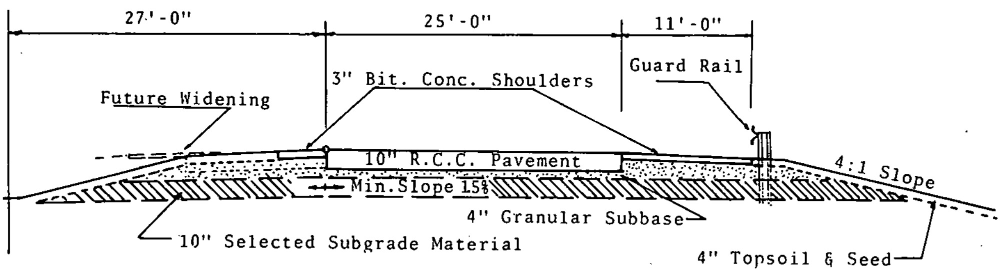



Table 1. Generalized geological and physical soils properties.

| Time Stratigraphy | Rock Stratigraphy | Description |
|---|---|---|
| Quartenary period | | |
| Holocene epoch | Topsoil | Black to brown clay silt |
| Pleistocene epoch | | |
| Wisconsinan stage | Tiskilwa till | Pinkish to brown clay silt (A-4 to A-6) |
| | Lee center till and esmond till | Yellowish brown to gray silty clay (A-7-6 to A-6) |
| | Argyle till | Pinkish brown to reddish gray silty clay (A-6) |
| Illinoian stage | Sterling till | Yellowish brown to gray silty clay (A-7-6 to A-6), underlain with outwash sands and gravel |
| Ordovician period | | |
| Galena | Undifferentiated | Brown to buff sandy dolomite, cherty and fractured |
| Ancell | St. Peter | White fine sand (A-2 or A-3), unconsolidated and dry |
| Cambrian period | Franconia | Greenish brown silty sand (A-4 to A-6), unconsolidated and dry |

Figure 2. Typical roadway section.



4. Ten-inch plain concrete roadway slab on a 4-in. portland cement concrete subbase. Various shoulder combinations were studied.

Ten-Inch Reinforced Concrete

As background for the changes we recommended, we first examined a typical section of the original facility. The 10-in. reinforced concrete slab, 25 feet wide (37 feet for three-lane construction), was placed on a permeable, frost-free, granular subbase 4 in. thick. Under the subbase was a 10-in.-thick layer of select material (Fig. 2). Both the select material and the granular subbase extended completely through the section, showing through in the median ditch and in the embankment slope.

The subbase was then built up and compacted to form the base material under 3-in.-thick bituminous concrete shoulders. The 10-in. reinforced concrete pavement on a permeable subbase as used on the existing Tollway has served well; most problems can be traced to local deficiencies. Based on service, a duplication of this design for the extension could be considered an excellent investment. However, a comparatively large amount of granular subbase was required, and, though aggregate sources were available, cost analysis indicated that the special processing required to render the subbase material permeable by removal of all but 3 percent of the materials passing a No. 200 sieve made the cost of an already expensive material almost prohibitive. The estimated cost of this section was $16.05 per square yard.

Eight-Inch Continuously Reinforced Concrete

Eight inches of continuously reinforced concrete on a 4-in. stabilized bituminous subbase placed on 4-in. granular subbase and 6 in. of select material were estimated to cost $11.04 per square yard.

Much of the continuously reinforced pavement examined was comparatively new and in relatively good condition. Cracking at close intervals is a characteristic of this pavement type, but the cracks remain tight because of the continuous reinforcing between anchorages. Some of this type of pavement was cracked at intervals of less than 2 feet. With this condition prevailing, it was difficult to understand how future large maintenance costs were to be avoided. If replacement of entire sections of pavement becomes necessary, the continuity of the reinforcing would be broken for the entire stretch of pavement between a set of anchorages, and the continuity advantage would be lost unless special costly methods of concrete removal were employed.

To be economically competitive, continuously reinforced pavement must, of necessity, comprise a thinner slab than other types. Inasmuch as continuous reinforcing must be meticulously placed, it is evident that progress in constructing this type of pavement is slower than in other sections. When interest costs on outstanding bonds amount to $25,000 per day, time is important.

Flexible Pavement

Flexible pavement has not been widely accepted in Illinois. Even though the estimated cost of the flexible pavement section was the least of all types studied, attempting to supply the large quantities of aggregate within the time permitted by the tight construction schedule may have caused a production problem. The 22-in.-thick study section was estimated to cost $9.16 per square yard.

Bituminous aggregate mix (BAM) is used extensively by the Illinois State Division of Highways as subbase under continuously reinforced pavement. However, its cost in Illinois is more than twice the cost of similar material in neighboring states. BAM used in shoulder construction has proved to be less than satisfactory. Important in our decision to reject the use of bituminous materials as subbase in main-line construction was our attempt to accelerate construction by eliminating the need for one more major subcontractor with related bituminous paving equipment.

Final Adopted Design

The fourth section investigated in light of the six performance criteria was 10-in. unreinforced concrete on a 4-in. PCC subbase. This section offered possible solutions to inherent problems with the soil conditions prevailing along the corridor; and, because concrete has properties of rigidity and beam strength that would distribute wheel loads over larger areas of the subgrade, it tends to keep deflections small and unit pressures on the subgrade low. This property of concrete is important during periods of reduced subgrade support, which accompanies subgrade thawing.

Traffic data from Wilbur Smith and Associates and soils information used with the Portland Cement Association design recommendations (3) suggested a 10-in. unreinforced slab. Comparison with similar pavements on the AASHO Road Test indicated excellent serviceability for slabs of 9½-in. unreinforced concrete on both granular subbase and subgrade with no base material. The original 10-in. reinforced slab constructed on 4 in. of free-draining base course has performed well for over 12 years, thus substantiating the adequacy of a 10-in. slab. The next problem was the selection of the subbase and base course.

Both bituminous- and cement-stabilized subbases were investigated. A cement-stabilized subbase was not compatible with the existing soil conditions, and the use of bituminous-stabilized material appeared prohibitively costly. The use of granular material also appeared costly because extensive processing of local materials would be required to supply the large quantities of free-draining subbase. A 4-in. PCC rigid base was selected. Study of paving literature, field examination of old concrete, and the results of the AASHO Road Test indicated that such a subbase could be used successfully with the 10-in. unreinforced pavement section (Fig. 3).

The section was completed with plain concrete shoulders that had a nominal thickness of 8½ in. and that were tied to the pavement slab. They were designed to be poured directly on the subgrade. Outside and inside shoulders are 11 and 5 feet wide respectively, and each has corrugations at 100-ft intervals to provide a rumble strip for safety purposes.

The resultant 10-in. unreinforced concrete pavement on a 4-in. PCC base was estimated to cost $9.25 per square yard. Actual award unit prices varied from $8.80 to $10.50 per square yard.

With the pavement section complete, parameters for joint spacing, control of warping and creep, and load transfer at joints had to be established within the constraints of cost, maintenance, and riding comfort.

The design specifications required that allowable flexural stresses be maintained within 50 percent of the ultimate strength of the concrete, whether tensile or compressive. This was possible even with a reduction in cross section of the slab resulting from sawed joints equaling one-fourth of the slab depth. Transverse warping was minimized by use of a longitudinal joint, usually sawed and equipped with load transfer bars. Such a joint was used in the recommended pavement.

Visual inspection of the Illinois Tollway pavement indicated the presence, almost universally, of two approximately equidistant cracks between each 50-ft sawed joint. They were usually relatively tight, but, as they open, sealing is required that, in the case of irregular cracking, is difficult because of the need for routing or grooving. Design requirements, visual inspection data from the Tollway and the AASHO Road Test section and experience in California with short joint spacing of plain concrete slabs resulted in adoption of a random joint spacing averaging 15 feet. Skewing the joints and varying the spacing prevent simultaneous wheel loading and eliminate the harmonic effect (Fig. 4).

Dowel cages at each joint for load transfer would all but destroy any economy inherent in the design. Without expansion joints and with short slab length, joints can open only slightly, and load transfer will be maintained by aggregate interlock. Computations indicated a thermal shrinkage of about 0.05 in. in a slab length of 15 feet due to a 60-deg drop from placement temperature. This feature of minimum shrinkage at each joint also minimizes the opportunity for infiltration of foreign material into the joint.

Pavement creep was provided for by using 4-in.-wide slots across and entirely through the pavement near bridge structures and by filling them with a compressible polyurethane material. To compensate for loss of aggregate interlock near these slots, load transfer bars were used in the adjacent 13 transverse joints.

## CONSTRUCTION

With the pavement section set as a 10-in. unreinforced concrete slab on a 4-in. PCC subbase, construction contracts were let to three prime contractors. Every effort was made in the field to control soils problems and to ensure timely performance of the contract.

Inasmuch as the ultimate performance of a pavement section depends on control of soils conditions in the subgrade, many problems were eliminated by using the following methods:

1. When water was encountered, the side ditch grades were lowered (where this was impractical, subdrainage was provided);
2. Drainage under the inner edge of the outside shoulder was provided where water is most likely to collect, e.g., in sag vertical curves;
3. Frost-susceptible soils at grade were removed or replaced; and
4. Moisture and density were carefully controlled in both embankments and cut sections.

With tolls as the only source of revenue, timely performance of the contract was a prime concern of the Toll Authority. Therefore, when the prime contractors proposed a change in construction of the pavement section in an effort to speed up the project, prompt action resulted.

### Field Construction

With the roadway grading under way, each of the three prime contractors requested that the 4-in. PCC subbase be incorporated into the pavement slab, making the section 14 in. thick, and that the entire unit be slip-formed.

This request prompted a review of the pavement section to determine how these changes would affect performance under traffic conditions. Generally, automobile traffic has little detrimental effect on a pavement section; however, the estimated 16 percent or more of heavy five-axle wheel loads and special heavy loads allowed under permits cause most of the destruction of pavements at thaw. Because the addition of 4 in. of unreinforced concrete would make the flexural strength of the slab almost twice that of the 10-in. section and with special care being taken with soils conditions to provide a stable subgrade, the acceptance of the proposal depended on the ability of the contractors to slip-form the 14-in. section. Agreement to use the 14-in. section resulted in a reduction in cost to the Toll Authority. This brought the cost for the roadway pavement section into a range of $8.40 to $10.10 per square yard.

Two contractors proposed to construct the 14-in. pavement section according to alternate 1 (Fig. 5). This scheme required that the 14-in., 25-ft-wide pavement unit be placed first, with the shoulder subgrade compacted and paved later. One contractor proposed to pave the entire 41-ft-wide roadway section, including shoulders, with a single pass according to alternate 2 (Fig. 6).

## Profile Control and Fine Grading

The final grade line of the pavement and the riding quality depend on the care exercised in constructing and finishing the subgrade, especially when no subbase material is used. How well the fine grading is accomplished and profile control is maintained determines the ridability of the pavement.

For alternate 1, the contractors used a standard 28-ft Autograde base trimmer controlled electronically by string line.

Both vertical and horizontal controls were maintained by electronic sensing units that adjusted the trimming action of the machine to the desired grade. The subgrade was left about 0.30 in. high, and excess material trimmed from the main line was used to construct the base for the shoulders. After completion of main-line paving, shoulders were compacted and trimmed to grade by an adjustable width unit that could be varied from 5 to 11 feet wide. For alternate 2, in which the pavement and shoulders will be constructed 41 feet wide, the trimming and shoulder shaping will be performed by using three trimming units so that the shoulders can be placed, rolled, and trimmed to form the raised section.

After proofrolling the subgrade, the 28-ft trimmer unit was able to prepare about 3,000 linear feet of base per day. Usually, the Autograder took two passes, with the subgrade being sealed after the last pass of the trimmer with a steel-wheeled roller.

## Main-Line Paving

Because paving alternate 1 and alternate 2 would differ little (requiring only equipment modifications), the 41-ft-wide technique is described.

The basic 41-ft paving train consists of two 41-ft-wide double-bucket side-loaded spreaders. This is followed by a specially adapted 41-ft-wide paving machine to vibrate, meter, and finish the pavement and shoulders. This unit will place 1.41 cubic yards of concrete per linear foot of roadway, as compared with the 25-ft-wide unit that places 1.06 cubic yards per linear foot. During the pavement placement, tie rods and plastic polyethylene tape are inserted at the centerline and shoulders.

Horizontal control for the paver is maintained by a magnetic sensor reading a metal banding strip placed on the subgrade. Vertical control is maintained by a ski on the subgrade for tangent sections and dual wires in the curved and superelevated sections. Because of the extra paving train width, haul roads are provided by a cut in the outside embankment slope and the median ditch, which will be left full and then finished later.

A 41-ft-wide curing and covering machine trails the slip-form paving unit. This unit includes a rumble strip maker for the shoulder area. By scraping and vibration, a 6-ft-wide safety rumble strip is formed at 100-ft centers. With the 25-ft-wide unit, the rumble strips were formed by hand inasmuch as several attempts to use a machine met with failure.

The final step is to use two dual 41-ft, skewed, diesel-powered sawing units to cut the dummy joints. The skewed joint angle is maintained electronically by sensing the pavement edge. The unit performs the 41-ft-wide 3.0-in.-deep cut in one pass by using tandem saws. Together, both units should encounter little difficulty in sawing the pave-

Figure 3. Original design for Tollway extension.

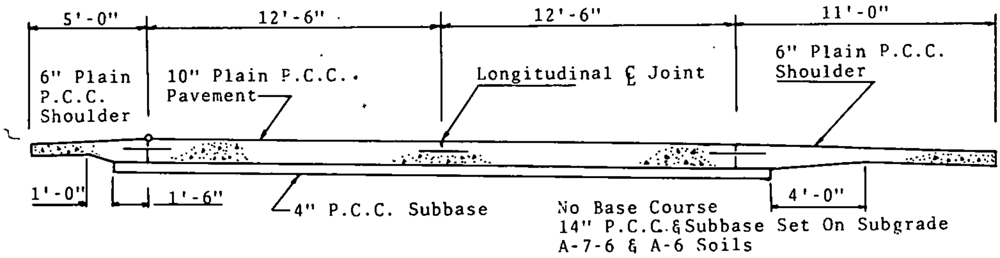



Figure 4. Typical random joint spacing.

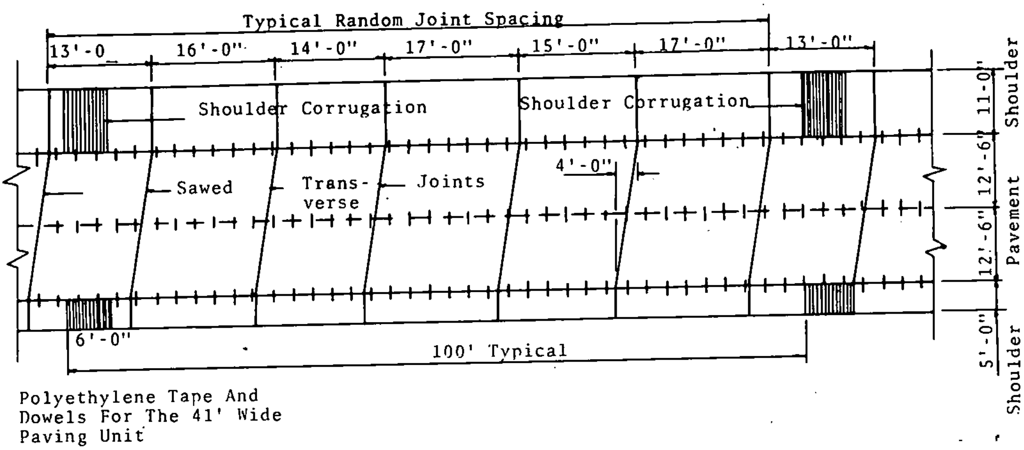



Figure 5. Paving alternate 1.

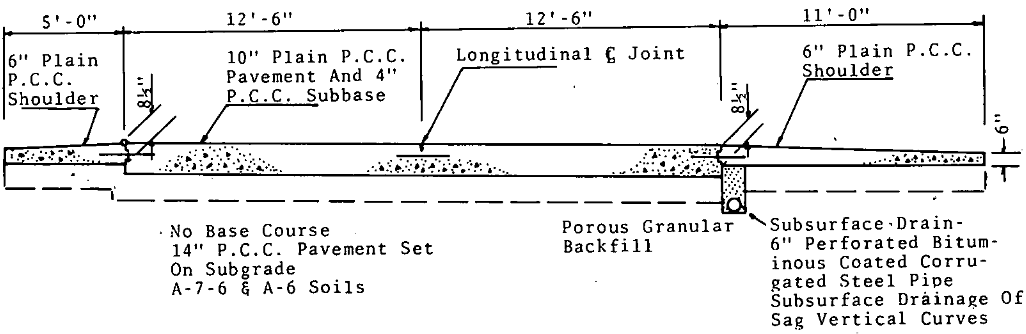
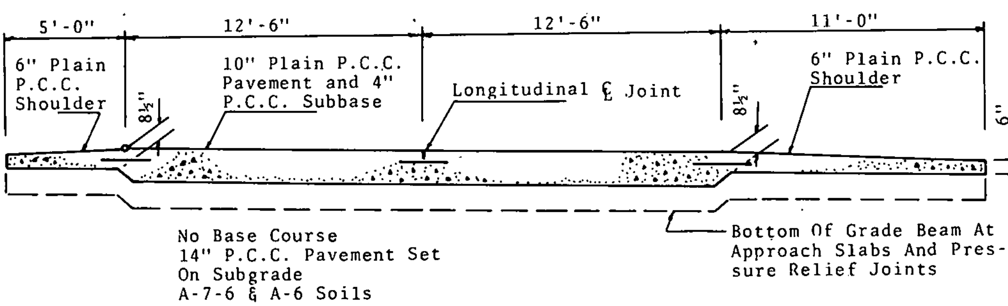



Figure 6. Paving alternate 2.



ment joints in sufficient time to prevent the formation of shrinkage cracks. In the 41-ft-wide section, the dummy joints are skew-cut the full pavement and shoulder width, whereas, in the narrower section, only the 25-ft roadway pavement dummy joints were skewed. The shoulder joints were formed perpendicular to the pavement edge.

### Keyway Formation

Under alternate 2, no keyway is required, but, for alternate 1, in which the main-line pavement section is 25 feet wide, a dual keyway was required to tie the shoulders and roadway together. Because the roadway section is 14 in. thick, edge slump is more probable than with thinner sections. To alleviate this problem by providing a partial form, an automatic keyway forming and installing unit was built by the contractors. The keyway was formed from a 3⅝-in.-wide, 20 gauge strip steel. The unit was added to the paver on a separate wing and contained shaping dies, a tie rod hole punch, a unit to punch holes every 5 in. to aid in holding the keyway in place, and an insertion roller. The steel strip was spliced by brazing or crimping. Each roll weighed 400 lb and contained about 2,200 feet of keyway.

Two major problems were encountered. Because the unit was on a separate wing, the keyway tended to ride up or down on the roadway slab. By rigidly attaching the unit to the paving machine, this problem was overcome. The second problem, which was eventually solved, was with the tie rods. These had to be installed by hand, and, if they failed to clear the insertion roller, the keyway strip and a section of the pavement were damaged.

## CONCLUSION

To meet the goals of an improved pavement design that could be constructed at a low cost and with the readily available construction materials, special considerations were given to

1. Soil conditions, especially the high water table and frost susceptibility,
2. Ease and speed of construction,
3. Safety,
4. Riding comfort, and
5. Expected performance and maintenance considerations.

Of the four pavement types examined, a 10-in.-thick unreinforced concrete slab on a 4-in.-thick concrete subbase met these goals most nearly. However, when contracts were let, the contractors proposed a 14-in. unreinforced pavement. This proposal proved to be satisfactory for traffic and provided ease and speed of construction.

To date, only the performance and maintenance considerations remain to be tested, and to thoroughly test these criteria requires time and traffic.

In an effort to provide long-term evaluation of the 14-in. section, soils moisture temperature gauges have been installed at six critical locations to monitor long-term moisture-density conditions and to study frost action. As the various segments of the East-West Tollway extension were completed, profilometer records were obtained, and an initial serviceability index was determined for comparison with maintenance records and future ridability determinations.

## ACKNOWLEDGMENT

This report was prepared in cooperation with the Illinois State Toll Highway Authority. Special acknowledgments are made to C. A. Benowicz, Chief Administrative Engineer, and George Knoblock, Engineer, who provided assistance and information to develop the basic paper.

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# slip-form construction of highway appurtenances

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This paper describes slip-form construction techniques for widening and resurfacing, curb and gutter building, single-lane concrete paving with integral curb, and concrete safety barrier. Formerly, most of this type of construction was done by using forms and hand tools. Labor requirements were high, and production rates were low. In recent years development of specialized slip-form equipment by contractors and equipment manufacturers has revolutionized this type of concrete construction. The authors describe some of the equipment available and the wide variety of equipment that provides for interchangeable mules or forms to shape concrete to almost any cross section. The paper presents some recommendations on suitable concrete mixes and finishing techniques for this type of work.

•When James Johnson of the Iowa State Highway Commission extruded his first concrete slab in the laboratories in Ames in 1947, it is doubtful that any of the engineers who were present could have imagined the ultimate development of slip-form pavers discussed in this paper. The Iowa engineers were looking forward to the development of a machine that could place a 10-ft-wide, 6-in.-thick slab. In 1949 such a machine was used to build a mile of a rural road in two 10-ft lanes. Other papers in this Special Report discuss some of the current projects using this sophisticated equipment. Today giant electronically controlled pavers are building concrete slabs up to 17 in. thick and 50 ft wide.

This paper deals with more prosaic paving. It describes the use of slip-form or extrusion equipment to build concrete safety barriers; to widen concrete shoulders, curb, and gutter; and to pave single lanes 10 to 16 ft wide with integral curb for ramps or city streets. Such construction has traditionally been done by hand methods. In many cases rather primitive wood forms and tools were used; production rates were low and labor costs high. Generally this type of work was subcontracted to specialty contractors, and unit costs were high. In some urban projects the total costs of miscellaneous paving work exceeded main-line paving costs.

Today the know-how and ingenuity of contractors and equipment manufacturers have provided slip-form equipment to build almost all concrete appurtenances without fixed forms and with a minimum of labor. This equipment, which has relatively high production rates, can be used with transit-mixed or centrally mixed concrete. The final quality of the work—of the concrete mix, alignment, finish, and texture—is usually superior to hand work, and the costs are lower.

Most equipment is now made so that different templates, molds, or mules can be interchanged quickly and easily on a single standard machine, to the user's specifications. Any curb style, any configuration of gutter, and various heights and shapes of walls can be built; longitudinal steel can be inserted through the front of the machine. Some provide built-in texturing brushes or brooms. Equipment has been developed to form a continuous key in footing and to leave marks to locate positioning of tie bars. Some can add devices that form and position keyways and shoot tie bars into the edges of slabs and gutters.

At least one curb and gutter machine has been made to shape the curb, gutter, and sidewalk simultaneously. Curb and gutter machines can trim the grade and slip form in a single operation, or grade trimming can be done separately.

Some machines have an "outrigger" hopper-conveyor that allows the concrete supply truck to transfer concrete to the slip-form machine without leaving the adjacent pavement slab or haul road. This unit is removable so that concrete can be dumped either into it or directly into the main hopper on the machine.

## CONCRETE WIDENING AND SHOULDERS

In the late 1940s and 1950s many 18-, 20-, and 22-ft pavements were widened to 24 ft, which had become the new standard for a two-lane primary highway. In many cases the widening work also included overlay of the entire 24-ft width. For this reason surface tolerance and edge alignment were not considered so important as on new exposed pavement. These concrete widenings were usually built with a crude improvised slip form, consisting of little more than a hopper box equipped with a vibrator, which was usually pulled by a tractor or motor grader.

In the case of widening by a full lane, e.g., older four-lane Interstate highways upgraded to six or eight lanes, the self-propelled slip-form pavers came into use to build new exposed concrete. One track of the paver operates on the edge of the existing slab as single 24-ft lane slip-form machines build the additional lanes. In most cases this work is carried out on the inside lane of the divided highway with no interruption to traffic. One or two additional lanes can be added to existing concrete pavements in this manner with no difficulty. On those projects where tie bars are required between the old lanes and the widening, these can be quickly placed in the edge of the existing slabs through the use of self drilling or stud types of threaded anchors (5).

Concrete shoulders have been used on some urban freeways in several states for some time. Missouri uses red pigment to provide a color differentiation with the main-line pavements. Illinois was the first state to build concrete shoulders on a rural highway in 1965. In an experimental project Illinois used corrugations built in the plastic concrete to provide extra safety (1). The corrugation discourages drivers from using the shoulder as an extra driving lane and provides a rumble or warning to alert drowsy or inattentive drivers who stray from the main traffic lanes (2).

The first project and two subsequent experimental shoulder projects conducted in Illinois in 1966 and 1967 were all built by using a rather simple slip-form device designed by the contractor. The device was towed by a tractor and held against the old slab by a motor grader. Because of the excellent performance of these experimental concrete shoulders, the Federal Highway Administration established the National Experimental and Evaluation Project in 1970, and many states began to plan and build concrete shoulders (Table 1) (3).

Paving contractors began to apply to shoulder paving the same type of high production equipment and techniques that they were using in main-line paving (4). Special smaller slip-form models were employed. Some contractors blocked out full-size slip-form pavers to build a 10-ft-wide shoulder. These more sophisticated self-propelled pavers established production records of a mile of shoulder per day. With most of the slip-form equipment used for shouldering, line and grade were controlled by an electronic

Table 1. Details of concrete shoulder projects.

| Present Condition | State | Year | Route | Length (miles) | Width (ft) | Thickness (in.) | Jointing (ft) | Remarks |
|---|---|---|---|---|---|---|---|---|
| Built | Ill. | 1965 | Ill-116 | 5 | 8, 4 | 6 P | 50, 100 | Described in Illinois R&D reports 24 and 27 |
| | | 1966 | I-74 | 0.83 | 10, 4 | 6 P | 10 to 100 | Described in Illinois R&D reports 24 and 27 |
| | | 1967 | I-80 | 1.9 | 10, 4 | 8 to 6 P | 20 | Described in Illinois R&D reports 24 and 27 |
| | | 1971 | Ill-72 | 7.5 | 10, 4 | 6 P | 20 | |
| | | 1972 | E-W Tollway | 69 | 11, 5 | 8½ to 6 | Random | Partially built; to be completed in 1973 |
| | Iowa | 1971 | I-80 | 1.25 | 10, 6 | 6 P | 20 | In connection with reconstruction project |
| | Md. | 1969 | | 3.6 | 3 to 8 | 7 R | 40 | |
| | Mich. | 1971 | I-69 | 1.5 | 9 | 9 to 6 P | 17, 9 in. | |
| | Neb. | 1970-71 | Neb-36 | 10 | 8 | 5½ P | 15 | |
| | N.C. | 1972 | US-52 | 4.5 | | 7 | 30 | 2-in. red colored concrete topping |
| | N.D. | 1972 | I-29 | 14.7 | 10, 3 | 8 | CRCP | 3-ft width slip formed with roadway pavement |
| | Pa. | 1971 | I-81 | 5.9 | 10, 4 | 6 P | 15½ | |
| | Tex. | 1971 | I-30 | 6.9 | 10 | 8 | CRCP | Experimental textures |
| Awarded | Ala. | | I-59 | 5 | 10, 4 | 8 | | Evaluate designs and delineation methods |
| | Ark. | | I-430 | | 10, 4 | 8 | CRCP | |
| | Ky. | | US-31W | 3.7 | 10, 4 | 5, 6, 7 | 20 P, 50 R | |
| | N.M. | | I-40 | 4.3 | 10, 4 | 8 | 13-19-18-12 | Stage construction |
| | Pa. | | US-220 | 23,000[a] | | 6 P | 15½ | |
| | | | | 21,000[a] | | 8 to 6 P | 15½ | |
| | | | | 2,500[a] | | 8 P | 15½ | |
| | W.Va. | | I-77 | 30,200[a] | | 8 | 20 | |
| | | | I-64 | 14,300[a] | | 8 | 20 | |
| Planned | Ariz. | | I-10 | | 10, 5 | | Skewed and random | In planning stage |
| | Calif. | | US-101 | | | | | In planning stage |
| | Ga. | | I-75 | 8 | 10 | 11 to 6 | | No subbase |
| | Idaho | | I-90 | | | | | In planning stage |
| | Minn. | | | | 10, 3 | | | In planning stage |
| | Nev. | | I-15 | 2.2 | 10, 4 | | | Being designed |
| | Ohio | | I-675 | 4.2 | 10 | | | Being designed |
| | S.D. | | I-29 | | | | | Experimental, in planning stage |
| | Utah | | I-15 | 1 | | 9 | Random | |

[a]In square yards.

Table 2. Constructed and planned concrete safety shape barriers (as of Dec. 31, 1972).

| State | Miles | State | Miles |
|---|---|---|---|
| Alabama | 1.0 | Nebraska | 1.7 |
| Alaska | — | Nevada | 12.1 |
| Arizona | 10.2 | New Hampshire | — |
| Arkansas | 15.3 | New Jersey | 218.0 |
| California | 182.0 | New Mexico | 8.3 |
| Colorado | 27.0 | New York (Thruway) | 1.0 |
| Connecticut | 2.0 | New York City | 34.0 |
| Delaware | 2.0 | North Carolina | 26.1 |
| D.C. | 6.0 | North Dakota | — |
| Florida | 6.0 | Ohio | 34.3 |
| Georgia | 11.7 | Oklahoma | 11.0 |
| Hawaii | 4.0 | Oregon | 62.5 |
| Idaho | 4.1 | Pennsylvania | 6.9 |
| Illinois | 3.0 | Rhode Island | — |
| Indiana | 6.4 | South Carolina | 4.6 |
| Iowa | 0.8 | South Dakota | — |
| Kansas | 1.0 | Tennessee | 10.6 |
| Kentucky | 15.7 | Texas | 45.0 |
| Louisiana | 17.0 | Utah | 20.0 |
| Maine | — | Vermont | — |
| Maryland | 47.0 | Virginia | 14.5 |
| Massachusetts | 1.0 | Washington | 49.5 |
| Michigan | 15.5 | West Virginia | 9.9 |
| Minnesota | 12.8 | Wisconsin | 32.0 |
| Mississippi | 3.0 | Wyoming | — |
| Missouri | 30.1 | Total | 1,032.4 |
| Montana | 5.8 | | |

guidance system. Such a system usually operates against the edge of the main-line slab to keep the shoulder tight against the pavement. A string line may be established outside the shoulder to provide elevation control for the outside edge of the shoulder, or a skid can run on the pretrimmed subgrade. The inside edge of the paver for the shoulder takes its elevation from the already-built pavement.

In 1972, on a project on I-29, the North Dakota Highway Department allowed a contractor to build the 3-ft inside shoulder integrally with the 24-ft two-lane slab. A 27-ft paver was used, and, because the same design and thickness were specified, the shoulder paving presented no special problem. This method completely eliminated one separate paving operation. This year on the extension to the Illinois East-West Tollway a contractor will use a 41-ft-wide slip-form paver to place the 25-ft main-line slab, the 11-ft outside shoulder, and the 5-ft inside shoulder all in a single pass. The details of this project are covered by Stone and Grimes (13).

Corrugated rumble strips in the shoulders of the Illinois Tollway have also been formed by newly developed equipment. A power-driven roller was mounted on a self-propelled machine that straddled the shoulder. The rotating corrugated roller made several passes to completely form the 6-ft-wide rumble strip according to job specifications.

Whereas most early shoulder projects used transit-mixed concrete, the slip-form equipment used recently permits a wider variety of hauling equipment. Several paving contractors have used their regular central mix plants and dump trucks with or without agitators. This permits higher rates of production and reduced paving costs.

## CURBS AND CURB AND GUTTER

For several years equipment that can build concrete curbs and curb and gutter sections without the use of side forms has been available. Early models were extrusion machines that required a special concrete mix having smaller coarse aggregate and low slump (6, 7). Generally these extrusion machines are smaller and less expensive than slip-form pavers, but their production rates are lower. Today there are at least seven equipment manufacturers who produce slip-form paving equipment capable of building a wide variety of curb and curb and gutter widths and shapes (8, 9).

All of these machines are electronically controlled and take both line and grade from preset string lines. They are all self-propelled. Many have provision for final trimming of subgrade to grade. Some of them have three crawler tracks and some two. Some machines straddle the curb with a track on each side, whereas others operate with the slip form outside of the tracks to permit construction of the curb close to fixed obstacles such as utility and light poles, fire hydrants, and traffic signs. Nearly all of the curb and gutter machines are designed to permit construction around curves of the type normally encountered at street intersections and at expressway ramps and cloverleafs. Whereas all of these pavers are equipped with some type of hopper to receive and hold the concrete, they are too small to receive an entire truckload of concrete directly from a dump truck. Hauling units must be equipped with a chute similar to those on transit-mix or agitator trucks.

A variety of slip-form pavers are being used to place curb and gutter sections. One machine also is being used to place a rolled curb and a hollywood type of sidewalk immediately behind the curb.

## STREET PAVING WITH INTEGRAL CURBS

In many municipalities where concrete streets are used, contractors have built integral curbs to eliminate having to build a separate curb and gutter. Many times city streets are built only one lane or half a street wide at a time to permit through traffic access to abutting property. Several equipment manufacturers have met this need by producing narrower slip-form pavers. Generally these are capable of paving lanes from 6 to 20 ft wide. Some provide for power widening. Most are also capable of building an integral curb on at least one side.

The manufacturers have attempted to produce slip-form pavers for city street work that are smaller, lighter, and less expensive than equipment for main-line highway paving. Usually city jobs are smaller and more cut up by intersections, drainage structures, and general street layouts. This is particularly true in residential areas with many curved streets and cul-de-sacs. For these reasons flexibility, portability, and maneuverability are probably more important than high rates of production. Because slabs are thinner (frequently only 6 in.) and because most street pavements are unreinforced, pavers can be considerably lighter and have smaller power units. In most models the concrete is deposited in a hopper from a chute, so transit-mixed concrete is used on many street paving projects. Today some manufacturers are providing conveyor spreaders with belts to receive concrete from a conventional dump truck hauling centrally mixed concrete.

## CONCRETE SAFETY BARRIERS

From construction of a curb and gutter with 16 or 18 in. back height it was only a small step to slip forming a 32-in.-high safety shape concrete barrier. Until 1971, all barriers were built in fixed forms or as precast sections. In 1971 two manufacturers of curb and gutter slip-form machines modified them to produce concrete barriers on projects in Maryland and Michigan (10). In less than a year at least five different machines were available, and barriers with a variety of designs and details had been built (Table 2).

In the Milwaukee area alone in 1972 two contractors used two different slip-form machines to build 29 miles of double barrier on each side of center-mounted lighting, traffic signs, and bridge piers in the median of an existing multilane Interstate freeway. Much of this work was done at night to minimize traffic delays, although all construction equipment including concrete trucks were confined to the inner lane adjacent to the median. Heavy truck traffic in adjacent lanes did not disturb the free-standing concrete barrier behind the slip form. On the Wisconsin project, where fixed obstacles were too close to the barrier to allow passage of the slip-form equipment, precast or cast-in-place barrier was substituted.

Slip-form machines have proved their versatility in placing concrete barriers. On some projects the concrete foundation for the barrier has been built by the same slip form, equipped with a different template, a day or two before barrier construction. Some contractors have successfully slipped the barrier integrally with the foundation in one pass. At least one machine is available that will slip a barrier on a superelevated curve section where the base of the barrier is at different elevations on the two sides. Another machine extrudes a barrier containing sheathed prestressing cables that are subsequently tensioned. The surface of barriers behind the slip form is excellent. Except for application of a light brush texture, no additional hand finishing should be necessary. Excessive floating or troweling may create an uneven surface that has less durability in a severe freezing and thawing environment than the surface immediately behind the slip form. Joints in slip-form barriers are formed most easily by sawing. With most aggregates a one-man portable saw can be used to cut surface grooves on both sides and over the top of the barrier at the specified intervals. Joints have been formed with hand tools, but this requires considerable manipulation of the free-standing barrier.

Curing of the barrier can be accomplished with polyethylene sheets, wet burlap, or white-pigmented curing membranes. Probably the latter method is most satisfactory and most economical. Care should be taken to ensure uniform application of the specified amount of membrane.

## CONCRETE MIXES AND CURING

The concrete mix used in slip forming highway appurtenances is not greatly different from that used in the same type of project built with forms. However, more uniform concrete with lower slumps will produce better results. Generally in street and shoulder paving, one size of coarse aggregate with a maximum of 1 to 1½ in. is recommended (11). For curb and gutter and barrier a coarse aggregate with a maximum size of ¾ or 1 in. will probably give better results. The ratio of fine to coarse aggregate will, of course, vary depending on aggregate type and gradation. Best results with slip forming are obtained when the fine fraction minus 100 mesh is high to produce concrete that will stand well when unsupported.

The slump of such concrete should generally be less than 1½ in. Most contractors report good results with slumps from ¾ to 1 in. Slip-form equipment works best with air-entrained concrete, inasmuch as it provides better consistency and workability. Air entrainment is also necessary to prevent segregation during hauling from a central mixing plant. Because all of the highway appurtenances discussed—shoulders, widening, curb and gutter, streets, and barriers—are subjected to heavy concentrations of de-icing materials in states where snow and ice are present, the concrete must have excellent durability. Air contents should be in the range of 5 to 8 percent, cement contents should be a minimum of 564 lb/yd$^3$, and water-cement ratios should be a maximum of 0.49 (12).

Recommended air contents for various maximum aggregate sizes are as follows:

| Maximum Aggregate Size, in. | Percentage of Entrained Air |
|---|---|
| 1½ | 5 ± 1 |
| ¾, 1 | 6 ± 1 |
| ⅜, ½ | 7½ ± 1 |

Adequate curing and a sufficient period of air drying are necessary before de-icing materials are applied. Laboratory studies have shown that curing periods of 7 days are necessary to ensure adequate strength and durable scale-resistant concrete. In periods of below-normal temperatures 14 days' curing must be allowed for types I and II cements. The use of type III cement or acceleration will permit the use of a shorter (7 day) curing period when temperatures fall to near 40 degrees. With slip-formed construction, this curing can best be obtained through the use of white-pigmented curing membrane. Field experience substantiates laboratory studies that a period of air drying after curing greatly increases the resistance of air-entrained concrete to de-icers. Pavements may be opened to traffic during the air-drying period provided that no de-icing chemicals are used during this period. Thirty days beyond the regular curing period is recommended for air drying before de-icers are used.

In areas of unusually severe exposure, e.g., urban areas where large quantities of de-icing materials are normally used, special surface treatments may be considered for added insurance against scaling. This is particularly important on projects completed late in the fall. Linseed oil treatments may be beneficial on shoulders, curb and gutter, and street pavements. But these treatments are not recommended on barriers because they darken the concrete and reduce its visibility at night. For this reason a clear or light-colored sealer of the type used on architectural concrete may be preferred if a sealer is considered necessary.

## CONCLUSIONS

Because of the need for uniformity of concrete—consistency and workability—contractors have learned that quality control at the plant is important. Extra care is taken at central mix plants or in transit-mix operations to ensure uniform slump, air content, and thorough mixing. Concrete placed by slip-form or extrusion methods has generally been of a higher quality than that used in formed work.

Most of the highway appurtenances discussed are not subjected to high-speed traffic as is the main-line pavement. For this reason specifications should provide practical tolerances on line and grade. Too much attention to minute details or unrealistic tolerances are not justified and will preclude the use of modern high production equipment or will add unnecessarily to the construction cost.

Slip-form equipment has proved its versatility and suitability for building all sorts of highway facilities. Wherever a street, highway, or barrier project is fairly long without frequent interruptions, the slip-form method of construction will generally be the most economical. Slip-form construction eliminates the need for form stripping and subsequent cleanup, thus minimizing delays or interruptions to traffic and permitting earlier opening to traffic.

Slip-form paving and construction have revolutionized concrete construction of transportation facilities. Contractors, engineers, and equipment manufacturers continue to demonstrate their ability to find better, more economical ways to build with concrete. Designs and specifications must be flexible enough to permit use of these new methods to obtain the greatest benefit from our limited transportation funds.

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# posttensioned concrete pavement

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This report covers the initial main-line installation of a posttensioned concrete pavement in Pennsylvania. A pavement thickness of only 6 inches was possible through the use of posttensioned strands in the slab. The installation was a single slab 500 feet in length and 24 feet wide. It rests on 3-in. bituminous concrete base and a 9-in. subbase, with a slip plane provided between the base and slab. There are 10 longitudinal strands in the slab as well as transverse reinforcement. The construction methods used and details of placement and materials are discussed. Concrete was placed over the strands by conventional slip-form paving equipment. Hydraulic jacking was used to apply tension in two increments during the curing period. Posttensioning promises to provide a crack-free pavement structure, thereby eliminating a source of deterioration and need for maintenance. It is too soon to make conclusions on the performance of this installation, although cracks have not appeared. The future of posttensioned pavement will depend mainly on cost of construction and benefits in relation to other pavement types.

•A new application of prestressed concrete pavement construction is currently being evaluated. The principle is similar to that used in prestressed beams for bridges and building structures. Steel strands are embedded in the concrete during paving and are posttensioned as the concrete cures.

There are several advantages to the use of posttensioned pavement: a reduction in pavement thickness from the normal 9 or 10 inches to only 6 inches and a reduction in reinforcing steel from about 15 to less than 5 $lb/yd^2$. It is anticipated that pavement cracking will be eliminated, thus reducing the maintenance and deterioration from this cause. Paving can be done with conventional equipment including slip-form pavers.

The first prestressed concrete pavement slab in Pennsylvania was designed and built by Jones and Laughlin Steel Corporation in 1957 (1). A 5-in.-thick, 12-ft-wide, 530-ft-long pavement section was built and tested under controlled loading conditions. The longitudinally prestressed, 400-ft center slab was flanked by two end slabs to simulate continuous construction. An extensive test series that included both static testing and moving load tests was carried out, but because of the time and cost involved no traffic load testing was included.

In 1962, the then Pennsylvania Department of Highways sponsored a research program at the University of Pittsburgh to evaluate the slab under extreme moving load conditions (2). Repeated loads from a weighted test vehicle were made to determine the magnitude and distribution of stresses and deflections in the slab. The slab performed well, even under subbase failure conditions, and remained free of transverse cracking although longitudinal cracking over the tendons occurred. No follow-up highway test installations were made at that time, since the construction method was not considered economically justified.

Recently, however, there has been renewed interest in this pavement design concept. New materials, new equipment, and advanced technology promise to make the system more practical. In July 1971, a 14-ft-wide, 300-ft-long ramp was posttensioned as a part of the interchange of US-113 and Del-14 near Milford, Delaware. Under the sponsorship of the Federal Highway Administration, 3,200 feet of 24-ft-wide posttensioned pavement was constructed as an access road to Dulles International Airport (3). Although final evaluation of these installations is not yet possible, they have shown the construction method to be valid and worthy of further study.

## POSTTENSIONED PAVEMENT PROJECT

The first use of posttensioned pavement for main-line highway construction was installed on US-222 (LR 157-25), the Kutztown bypass, in September 1972. The bypass is a 4.98-mile section of limited-access, four-lane highway in Berks County. The construction was performed by a contractor under the supervision of PennDOT.

Because of the contractor's previous experience on the Dulles job, he was interested in making a similar installation in Pennsylvania and offered to place a short section for the same unit price as conventional pavement. There was an adjusted price made for a portion of treated subbase. Cost details will be discussed later.

One 6-in.-deep, 24-ft-wide, 500-ft-long posttensioned slab was placed in the westbound lane from station 890+00 to 895+00 just east of the interchange with Penn-737. The slab is on tangent alignment with a 2.05 percent grade. The bypass was completed in May 1973.

## MATERIALS

The subgrade material was predominantly A-4 soil. A 9-in. subbase course was placed prior to construction of a 3-in. bituminous concrete base course. The same design was used for both the conventional and posttensioned concrete; design data are given below. Mixing was done on the job at the contractor's batch plant.

| Factor | Value |
|---|---|
| Cement factor, bags/yd | 6.25 |
| Slump, in. | 1.5 |
| Air content, percent | 6.5 |
| Cement-sand-stone ratio (adjusted) | 1:1.938:3.365 |
| Water, gal/bag | 5.08 |

The prestressing strands were a 7-wire strand of high tensile steel in a polypropylene conduit, prepacked with corrosion-inhibiting grease. The properties of the strand are given in Table 1.

## PROCEDURE

The installation began with the placement of a subslab at each end of the experimental section (Fig. 1). Perforated underdrain pipe was placed under each subslab and was connected to the longitudinal underdrain 2 feet from the edge of pavement. A 9-in. aggregate subbase had previously been placed for both the conventional and posttensioned

Table 1. Strand data.

| Item | Quantity |
|---|---|
| Strand diameter, in. | 0.6 |
| Steel area, $in.^2$ | 0.215 |
| Length per pound, ft | 1.36 |
| Modulus, psi | $28 \times 10^6$ |
| Ultimate strength, kips | 58.6 |
| Temporary force-maximum (80 percent of ultimate), kips | 46.9 |
| Stressing load (70 percent of ultimate), kips | 41.0 |
| Design load (60 percent of ultimate), kips | 35.2 |

Figure 1. Subslab and joint detail.

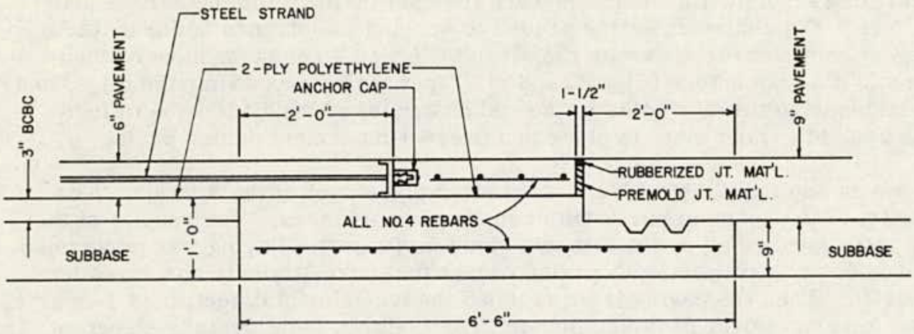
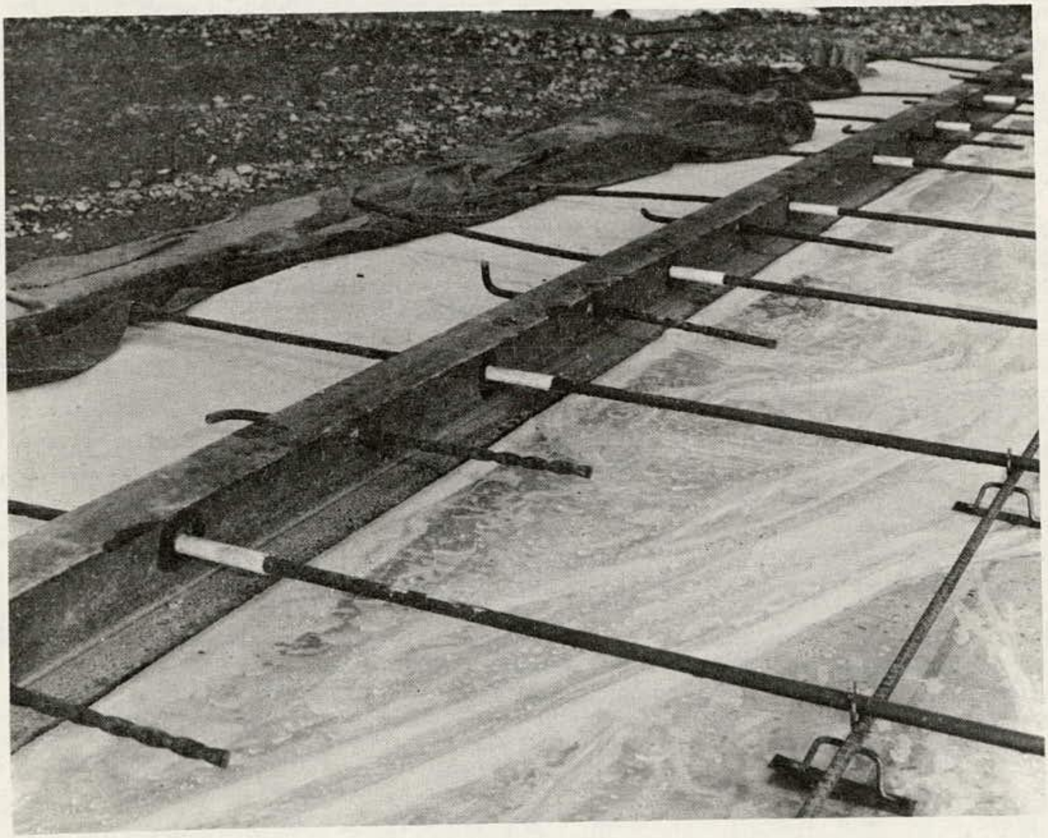



Figure 2. End beam.



pavements. A 3-in. layer of bituminous concrete base course was used under the posttensioned pavement for additional support and to make up the difference in thickness between the 6-in. pavement and the adjoining conventional 9-in. pavement. Sand was swept over the surface of the bituminous concrete base course to fill the surface voids and prevent any interlocking action between the slab and base course. Two layers of polyethylene sheeting were then placed on the base.

A 6-in. steel channel section was placed on each subslab to distribute the load from the strands across the slab (Fig. 2). The channels were temporarily held in place by welding them to reinforcing rods that had been embedded in the subslab (Fig. 3). The rods were cut off after paving.

Prefabricated transverse steel assemblies were then set on the bituminous base and spaced at 2½ feet. The chairs and clips needed to support the strands had been tack welded to them at each intersection with the strand (Fig. 4). Ten strands were then laid in place on 2-ft 5-in. centers (Figs. 5 and 6). They were snapped into the clips, and their ends were inserted through holes in the end channels. A small tension of less than 1 kip was applied to hold them in place and prevent movement during paving.

Paving was done on September 14, 1972, 3 days after placement of the strands. The contractor used a CMI slip-form paver with two Maxon spreaders. The results of the air and slump tests performed on the site are given in Table 2. Paving was performed in the same manner as in conventional paving except that normally only one spreader would be required. When the paving train reached the experimental section, a 1-hour delay occurred for changeover of the equipment from 9-in. to 6-in. depth. A section of conventional 46.5-ft pavement adjoining the subslab at each end was left out to provide a changeover area.

Paving of the posttensioned slab began at 12:30 p.m. and was completed in 2 hours. Concrete was placed in two layers and was given a standard transverse broom finish. A white-pigmented curing membrane was applied, and white polyethylene sheeting was used to cover the slab. The weather during paving was partly cloudy and breezy with temperatures in the 80s (Table 3). Temperatures during the curing period ranged from 48 to 83 F (Table 4). A 2-in.-deep by ⅜-in.-wide longitudinal center joint was sawed the next morning. The polyethylene sheeting was removed at the same time.

Tensioning of the strands was specified in two stages. The first load of 10 kips was to be applied to each strand when the concrete compressive strength reached 1,000 psi. The purpose of this loading was to control shrinkage and eliminate early cracking. The final load of 46.9 kips was to be applied at a strength of 2,500 psi. This load is 80 percent of ultimate for the tendons; the design load for the slab is 60 percent of ultimate or 35.2 kips. This allows for losses due to friction, strand relaxation, and concrete shrinkage.

Tests on the morning of September 15 showed the strengths to be well above the 1,000 psi required for the initial loading (Table 5). Because only one strand could be stressed at a time, an alternating pattern was used, from the center outward toward the edges (Fig. 7). Tensioning was done with a hydraulic jack on all strands first at one end and then at the other to reduce the effect of frictional loss in the strand. Elongation data are given in Table 6. Final tensioning was accomplished on September 17.

The end of each strand is anchored by a collar with a two-piece wedge set that grips the strand. The collar bears against a 3-in.-diameter washer that has a pipe coupling welded to it. After jacking was completed and the excess strand cut off, a cap was screwed into the pipe coupling, enclosing the collar. Grease was injected into the cap to prevent corrosion of the wedges and strand ends.

Figure 3. Temporary strand anchor.

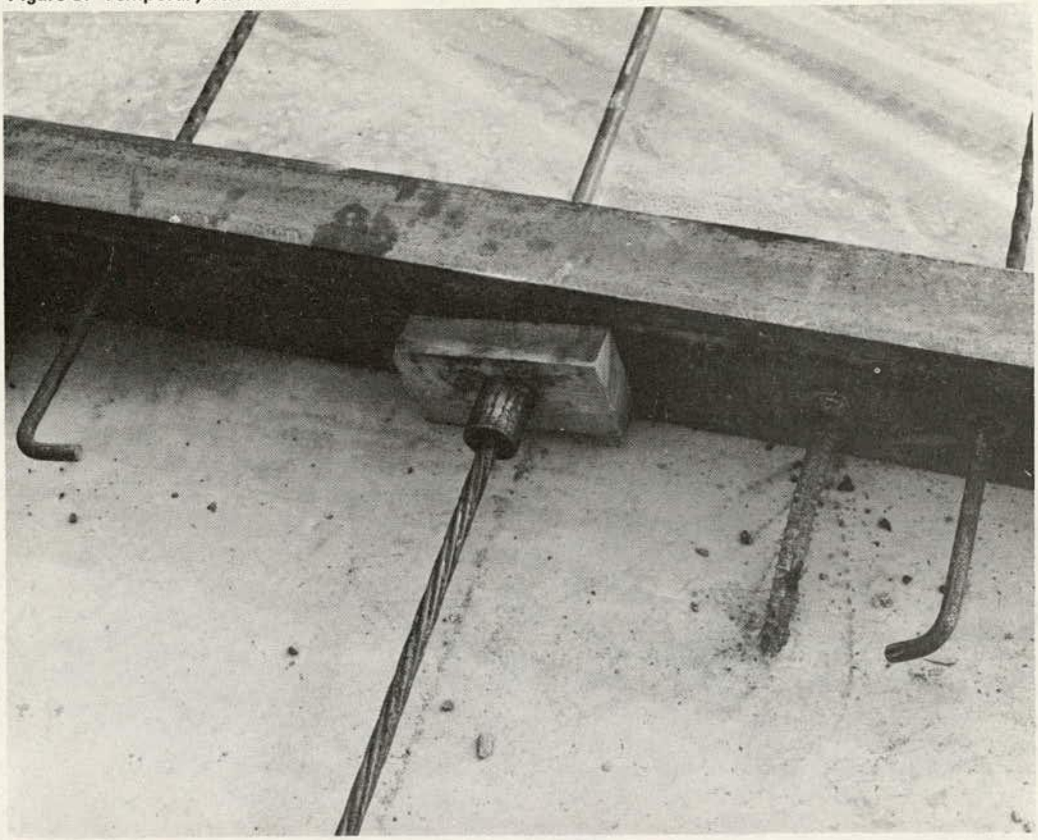


Figure 4. Chair and clip detail.

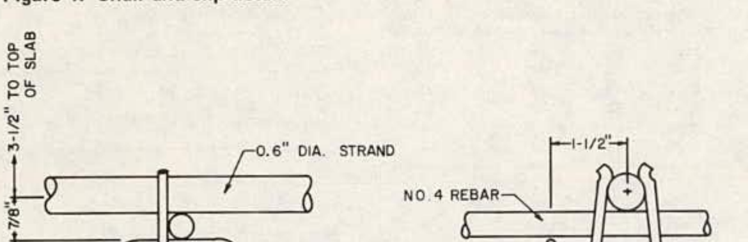



Figure 5. Pavement cross section.

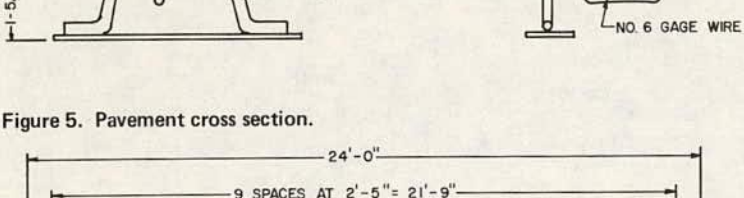


Figure 6. Steel layout.

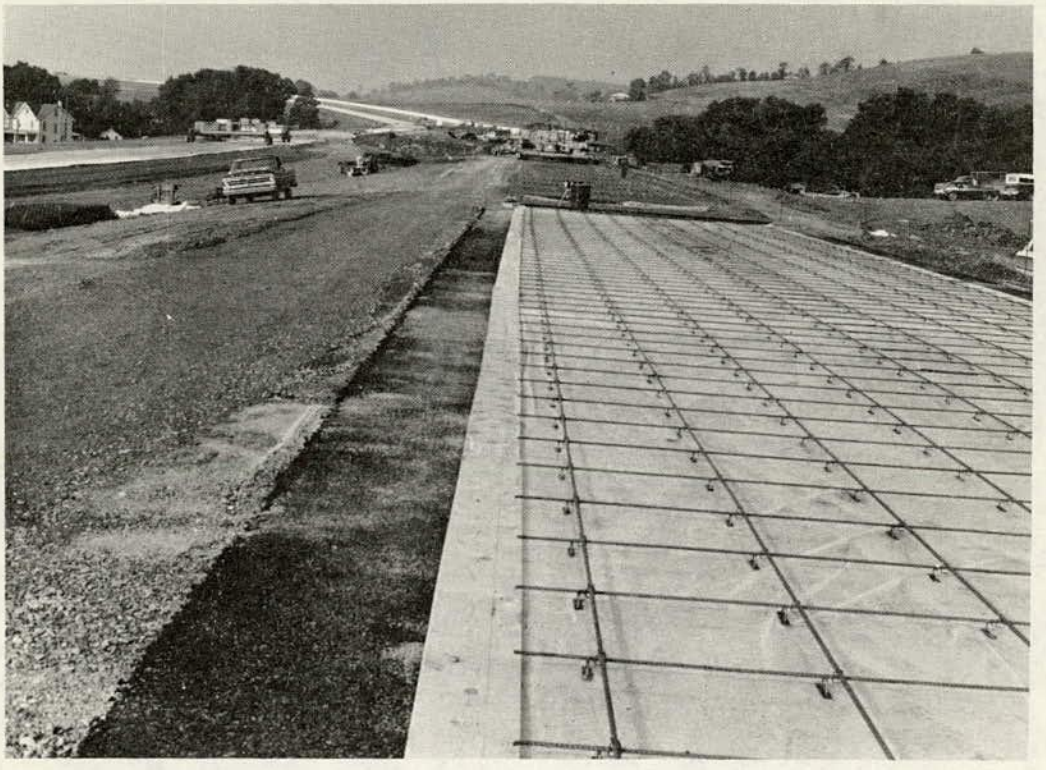


**Table 2. Results of air and slump tests.**

| Time | Slump (in.) | Air Content (percent) |
|---|---|---|
| 12:35 | $1\frac{1}{2}$ | |
| 12:40 | | 5.3 |
| 12:45 | $1\frac{3}{4}$ | |
| 12:50 | | 5.7 |
| 1:05 | $2\frac{3}{4}$ | |
| 1:10 | | 6.4 |
| 1:20 | $2\frac{1}{2}$ | |
| 1:25 | | 6.9 |
| 1:35 | 3 | |
| 1:40 | | 7.0 |
| 1:45 | $2\frac{3}{4}$ | |
| 1:50 | | 6.6 |
| 2:00 | $2\frac{3}{4}$ | |
| 2:10 | | 7.5 |
| 2:15 | $1\frac{1}{2}$ | |
| 2:20 | | 5.7 |

Note: Design slump was 1.5 in.; design air content was 6.5 percent.

**Table 3. Temperatures during paving.**

| Time | Air | Concrete | Base |
|---|---|---|---|
| 12:30 | 88 | 78 | 94 |
| 12:50 | 86 | 80 | 92 |
| 1:30 | 84 | 78 | 92 |
| 1:50 | 82 | 80 | 88 |
| 3:00 | 80 | 74 | – |
| 4:06 | 80 | 84 | – |

**Table 4. Concrete curing temperatures.**

| | Concrete | | Air | |
|---|---|---|---|---|
| Date | High | Low | High | Low |
| 9-15 | 89 | 70 | 74 | 50 |
| 9-16 | 87 | 63 | 78 | 48 |
| 9-17 | 93 | 55 | 83 | 52 |

The short slab adjoining the end of the posttensioned slab was placed last. It encases the collar assemblies and is jointed to the end channel by 10 hook dowels. Polyethylene sheeting between the end slabs and the subslab allows them to move as part of the posttensioned slab. Premolded joint filler and rubberized joint sealer at the ends completed the installation.

## OBSERVATIONS

1. The time between paving and initial tensioning should be carefully considered on future projects. Strength gain and hydration temperature must be related to ambient temperature changes. Rapid evaporation of moisture from the concrete could lead to shrinkage cracking if tensioning is delayed. Although this did not occur on the Kutztown project, it is an important factor. Under ideal curing conditions it may be possible to tension in only one stage.
2. The sand used to fill the voids in the surface of the bituminous concrete base course was taken from the concrete mix stockpile and contained particles as large as ¼ in. Some of these larger particles were left on the surface to create an interlocking action between the slab and the base. The severity of this interlocking is probably negligible, but a finer grade sand should be used in future installations with this type of base, or the base gradation should provide a smooth surface.
3. The initial stress of less than 1 kip, which was applied to hold the strands in place, had been lost by the time paving was done. This was probably due to a change in temperature and appeared to cause no problems in paving.
4. The clips on the transverse steel assemblies, which held the strands, were too tight. Approximately 40 percent of the clips cut through the polypropylene conduit, allowing the protective grease to seep out.
5. When the first layer of concrete was spread, it was necessary for the spreader operator to take care that the concrete did not pile up and get pushed by the machine. The transverse steel assemblies were pushed along with the concrete and were displaced as much as 6 inches. The problem was solved when the operator became aware of it and took appropriate precaution.
6. A test was made before final jacking to determine the friction loss in the strands. Strand No. 5 was released and a load cell was placed on the end at station 895+00. The other end was jacked, and readings from both the load cell and the jacking gauge were taken. Readings at several loads are given in Table 7. These readings represent the loss for a 500-ft length.
7. The theoretical strand elongation at the final load of 46.9 kips for 500 feet is 46.7 in., calculated as follows:

$$\text{Theoretical elongation} = \frac{PL}{AE} = \frac{46.9 \text{ kips} \times 500 \text{ ft} \times 12 \text{ in./ft}}{0.215 \text{ in.}^2 \times 28 \times 10^3 \text{ ksi}} = 46.7 \text{ in.}$$

Actual average strand elongation was 12.9 percent less than theoretical.

8. No cracking of the posttensioned slab had been observed 6 weeks after paving. A hairline crack had formed between each end slab and end beam. These are unavoidable and should cause no problems.

PennDOT will continue to periodically observe the project's performance and serviceability.

## COST DATA

The future of posttensioned pavement will be determined by its ability to compete economically with conventional paving methods. Savings in materials and maintenance are somewhat canceled by increased labor costs. The contractor's costs for the Dulles Airport experiment were \$7.46 per yd$^2$ for this type of construction (4). This same

Table 5. Concrete compressive strengths.

| Cylinders | Age (hours) | Average Compressive Strength (psi) |
|---|---|---|
| 1, 2 | 18 | 1,611 |
| 3, 4 | 22.5 | 1,841 |
| 27, 28 | 21 | 1,434 |
| 5, 6 | 28 | 1,593 |
| 29, 30 | 26 | 1,815 |
| 7, 8 | 46 | 2,611 |
| 31, 32 | 44 | 2,009 |
| 33, 34 | 48 | 2,210 |
| 9, 10 | 50 | 2,684 |
| 35, 36 | 67 | 2,570 |
| 11, 12 | 65.5 | 2,629 |

Note: Cylinders 1 to 24 were molded at station 890+00; cylinders 25 to 48 at station 895+00.

Figure 7. Strand indentification and loading sequence.

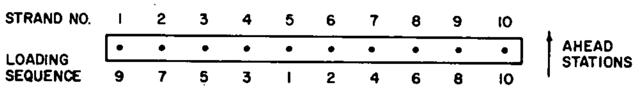



Table 6. Strand elongations in inches.

| Strand No. | Tension Applied (kips) | | |
|---|---|---|---|
| | 0 to 10 | 10 to 46.9 | 0 to 46.9 |
| 1 | $6\frac{1}{8}$ | $32\frac{5}{8}$ | $38\frac{3}{4}$ |
| 2 | $6\frac{5}{8}$ | $33\frac{3}{8}$ | 40 |
| 3 | $5\frac{15}{16}$ | $33\frac{1}{2}$ | $39\frac{7}{16}$ |
| 4 | $7\frac{1}{8}$ | $32\frac{3}{8}$ | $39\frac{1}{2}$ |
| 5 | $6\frac{7}{8}$ | $38\frac{1}{4}$ | $45\frac{1}{8}$ |
| 6 | $7\frac{1}{16}$ | $33\frac{1}{2}$ | $40\frac{9}{16}$ |
| 7 | $7\frac{1}{16}$ | $33\frac{1}{2}$ | $40\frac{9}{16}$ |
| 8 | $7\frac{3}{16}$ | $34\frac{3}{16}$ | $41\frac{3}{8}$ |
| 9 | $6\frac{3}{8}$ | $34\frac{11}{16}$ | $41\frac{1}{16}$ |
| 10 | $6\frac{11}{16}$ | $33\frac{15}{16}$ | $40\frac{5}{8}$ |
| Average | 6.71 | 33.99 | 40.70 |

Table 7. Friction loss in strands.

| Load on Jack (kips) | Load on Cell (kips) | Friction Loss (percent) |
|---|---|---|
| 28.0 | 22.1 | 21 |
| 41.3 | 34.0 | 17.5 |
| 46.9 | 41.2 | 12 |

contractor expects a cost of \$6.02 per yd$^2$ for longer projects. These costs do not include joints. The contract price for conventional 9-in. pavement on the Kutztown bypass was \$6.00 per yd$^2$.

Costs are expected to decrease if the method is put into general use. Treated subbase has been used on previous projects rather than separate base and subbase courses. It has been suggested that elimination of the transverse steel may be possible, and it may be possible to use four strands per lane instead of five. The strands could be laid directly on the base and picked up by holding devices on the paver to position them at the proper height. Specialized gang jacking equipment could also be used.

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4. Concrete Construction, Vol. 17, No. 10, Oct. 1972, p. 478.

# new construction equipment and techniques for airports

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This paper discusses some of the new techniques and equipment used to construct the mammoth Dallas/Fort Worth Airport. As a background to this discussion, the author gives some of the advantages and disadvantages in using new equipment. To give an indication of the magnitude of the Dallas/Fort Worth Airport project, the author relates some of the quantities of materials used on the project. He goes on to mention some of the problems in meeting the contract specifications and tells how they were resolved.

•Contractors have three basic tools: men, machinery, and methods. The successful contractor is the one who makes the best mix of these three m's. Some may wonder why some contractors have modern equipment and others are still using older machines and methods. There are of course advantages and disadvantages affecting the decision of any contractor on whether he should purchase new equipment. It is not always a question of whether he has the money to spend, but a question of where he can get the greatest return on his investment.

## ON USING NEW EQUIPMENT

There are some basic considerations that influence the use of new equipment in the construction industry. In the first place, it must be more economical in either cost, time, or performance. We should not automate something that a man can do more cheaply if the same results are obtained.

### Advantages

At the present time there is a lack of manpower skills. This, along with the rising cost of wages (from 5 to 7 percent a year), favors the automation of certain jobs. In addition to rising wages, there has been a continuing decrease in labor productivity, roughly at the rate of 10 percent a year. Invariably, in any crew, the slowest man sets the pace; therefore, when there is a scarcity of skilled people, the overall job incentive is low.

Specifications are becoming stricter, and there are closer control and tolerance requirements. This becomes almost cyclic: As better testing facilities are developed and causes are determined for past failures, it is only natural that closer controls and tolerances result. Also, the manufacturers' competitive position has a terrific influence on whether new equipment is introduced. At the present rate of technology, new equipment appears almost every 2 years, whereas in the past it was usually 5 to 10 years before new pieces of equipment were introduced to the market.

Also, as projects become larger and time limits become shorter, greater production is required. Overhead costs are rising each year and time becomes an even greater factor. Time lost because of weather becomes a major factor in meeting completion schedules; consequently, high production is required when weather conditions are favorable. It is in a contractor's interest to try to organize his men, machinery, and methods such that the overall time on a given project is reduced.

Disadvantages

There are also disadvantages that discourage the use of new equipment. There is sometimes a lack of economic incentive; i.e., short-term use of specialized equipment makes it almost impossible to justify the purchase price unless the need for the equipment is considerable. High investment costs may overrule the labor savings. Also, the obsolescence occurring in today's equipment dilutes the value of the investment. As mentioned before, technology is changing every 2 to 3 years. This technology, along with the steep competition equipment manufacturers are confronted with, forces the introduction of automated equipment and systems into the market before they have been properly tested. The results are sometimes poor. This has caused field people to question the use of highly automated equipment, inasmuch as it results in downtime due to repairs. Repair requires highly specialized skills. It used to be that you could wire something with a piece of bailing wire and get along, but now you almost need an electrician, a mechanic, and a manual on automation.

Severe environmental conditions, e.g., dust, moisture, heat, and vibration, sometimes discourage the use of automated equipment. Field conditions can never be duplicated in the laboratory, and what will work in a closed, airtight building sometimes will not work at all out in the environment.

Another deterrent to using new equipment is owner specifications that limit the use of modern equipment by requiring detailed types of equipment to perform the work. It is very important that engineers keep abreast of modern advancements in equipment designs and capabilities.

Everyone must be willing to gear up for the new equipment pace. Ordinary industry advancement in equipment, even though not always new or revolutionary, has to be adjusted to present-day problems. The construction industry must learn to cope with automation, which is the trend of the times. Costs are going to continue to rise, and we must find ways to maximize the use of cost-saving devices and to properly evaluate the economics involved. It is contractors' responsibility to develop their own systems and make recommendations to the manufacturers. Most new equipment ideas start at the field level. The needs for lower cost, mass productivity, time savings, and so on are again the catalyst for invention. We must train our people to newer skills to use and maintain this type of equipment. Without proper education and training of personnel, the age of machines can soon revert back to stone age methods and the cycle can start all over again.

## INNOVATIVE METHODS AND MACHINERY USED ON DALLAS/FORT WORTH AIRPORT

Included in the planning of airport construction are the projection of aircraft of the future, trends in research and development, legislation affecting airports, environmental considerations, and land use planning. In addition, financing larger facilities and engineering management of all this planning become major factors in the operation. This tremendous planning and the design phase become reality only after the airport is actually constructed.

The Dallas/Fort Worth Airport is the largest that has ever been built, with a contract amount of $57 million. It required initial planning of men, machinery, and methods capable of doing a volume of work each month of around $3.2 to $4 million. Because of this, the methods and equipment would not be practical on a smaller project; by the same token, the size of the job made it possible to spend a lot of time, effort, and money devising innovations that would produce the type of volume required.

The quantities of materials used give an indication of the magnitude of this project. Stabilizing the subgrade in the apron areas required 50,000 tons of lime. The cement-stabilized subbase required 770,000 $yd^3$ of base material and approximately 400,000 barrels of cement. The concrete runways, taxiways, and aprons, varying from 15 to 17 in. thick, required approximately 1,500,000 $yd^3$ of concrete, approximately 3,000,000 tons of aggregate, 2,250,000 barrels of cement, 450,000 gal of additives, and 18,400 tons of reinforcing steel. The electrical and lighting systems required 250 miles of buried cable and 7,000 lights. The surface of the concrete runways required 5,775,000 $ft^2$ of nonskid grooving.

The first consideration was how to economically fulfill the specification requirement that the aggregates be stockpiled and used in such a way that they would not be segregated at the time they entered the mixer. The fact that we were purchasing concrete aggregate from four producers created an additional problem in this regard. It would be impossible to keep each producer's stockpile separate and then introduce each into the overall concrete batching operation. The answer was to use mass stockpiling methods and rescreen the materials before they were stored in the aggregate bins that feed the concrete mixers. To ensure adequate amounts of materials, we stockpiled approximately 500,000 $yd^3$ of aggregate before paving operations were begun.

As soon as the required excavation was accomplished in the apron areas, it was necessary to stabilize the areas with as much as 18 inches of lime. We started the project by depositing dry lime on the surface and then mixing it with the soil-stabilizing equipment. Because of environmental dust, however, we changed to a slurry mix that required mixing the lime with water prior to placement.

The specifications required that cement-stabilized base be placed not only on the apron areas but also on the taxiways and runways. The base material was hauled in from an outside source and mixed through a stabilizing plant with 5 percent cement. Again, we had to stockpile a considerable amount of material before placement.

Most of the automated equipment on today's market requires that a string line be set along the side of the operation so that the sensing devices attached to the equipment have something to follow. This was used for trimming the base and for paving the concrete runways as well.

With the schedule required on slip-forming the concrete runways and taxiways, it would have been impossible to tie and place the reinforcing steel on the site; therefore, we erected a building in a central location where we fabricated the 25- by 50-ft reinforcing mats and then hauled them to the paving spread for placement.

Before the concrete was placed on the runways, the cement-stabilized base had to be sawed to provide trenches for the electrical cables that would eventually supply power to the runway lights. It was also necessary to place the light cans in the runway before the concrete was placed. Dowel baskets had to be inserted at 50-ft spacings prior to placement of the concrete.

Because of the heavy mass of concrete that was to pass over the dowel baskets, we found it necessary to drill steel pins into the stabilized base and tie the dowel baskets to these pins to keep the baskets from moving.

Once most of the preliminary construction was out of the way, we were ready to start the paving operation. We anticipated that we would need at least 10,000 to 12,000 yards a day to meet our schedule. To accommodate this, we employed two 12-$yd^3$ Rex mixers and one 8-$yd^3$ plant. To use these large plants required that we employ hauling equipment that would be large enough to carry the concrete away from the mixers to the paving spread. We used Cat 769 hauling units and placed a special ejector bed on them that would eject the concrete without having to raise the truck bed. We placed two 10-yd batches in each truck, which allowed us to haul 20 yards of material at a time. Although paving concrete with slip-form pavers was nothing new, the maximum width that had been tried up to that time was 48 feet. All of this had occurred on highway work but had never been used on airfield paving. We chose to accept the challenge of using a 50-ft slip-form machine to pave the Dallas/Fort Worth runways and taxiways after the Gunnert-Zimmerman manufacturing company guaranteed us that such a machine could be built.

Inasmuch as we started paving in the middle of the summer, temperature was a critical factor. To keep the concrete mix as cool as possible required that a liquid nitrogen system be installed that would cool the water to 32 F.

This paper has presented a thumbnail sketch of the construction methods and equipment that were involved in building the Dallas/Fort Worth Airport. In conclusion, I would like to leave these thoughts with you. The construction industry, including contractors, engineers, and equipment manufacturers, is an industry that must grow with automation, not fight it. We must all have the courage to make new tracks and not follow the ruts of the past. The future will bring greater demands for high-level ingenuity, creative thinking, and advanced education.

# a review of field applications of fibrous concrete

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The use of fibrous concrete as an overlay or resurfacing material has been tested in many field research projects. These projects have been designed to test thickness requirements, fiber requirements, mix design, jointing requirements, placement conditions, and construction procedures. Several tests have been conducted of fibrous concrete resurfacing on airport taxiways, urban expressways, arterials, residential streets, and bridges, and the results of some of these tests are discussed.

• Fibrous concrete is a composite material consisting of a concrete matrix containing a random dispersion of small fibers. The fibers, closely spaced at random angles, reinforce the concrete matrix in all directions. The fibers not only compensate for the relatively low tensile strength and brittle character of concrete but also improve other properties of the composite material. The broken beam section in Figure 1 shows the distribution of 1-in. fibers in a mix containing 120 $lb/yd^3$.

Laboratory research has shown that among the important changes in the properties are

1. A substantial increase in flexural and tensile strength,
2. An increase in fatigue endurance limit,
3. Higher resistance to abrasion and spalling, and
4. Higher impact resistance.

The degree of improvement depends on a number of factors. Some of the more important factors will be discussed later.

## TYPE OF FIBERS

Fibers have been produced from steel, plastic, and glass in various shapes and sizes. The fibers used in the following field applications are flat steel fibers produced either by shearing steel sheets or by cutting or chopping wire into lengths ranging from 1 to 2½ inches. The flat steel fibers are approximately 10 by 22 mils in cross section (Fig. 2), whereas the chopped wire is in the range of 16 to 25 mils in diameter.

## MIX DESIGN CONSIDERATIONS

The additional strength imparted to the concrete by the fibers depends on the bond between the fibers and the concrete mortar. With adequate bond, microcracks that develop in the mortar will be arrested upon reaching a fiber, which takes over the tensile stresses from the mortar.

Because the addition of fibers greatly increases the surface area that must be coated with cement paste, a higher percentage of mortar is required to produce a workable mix than is required for normal concrete. The wire segments also contribute to a high internal friction, which also affects the workability of the mixture. Thus fibrous concrete mixes are designed with a higher percentage of fine aggregate and either a higher cement content or the addition of pozzolanic material such as fly ash to improve the lubricating properties of the mortar. Water reducers are frequently used to reduce the water demand of these mixes. Air entrainment is also used to assist in providing increased workability.

The term aspect ratio has been adopted as a convenient numerical parameter describing a fiber. Aspect ratio is the length of a fiber divided by its diameter. For flat fibers the equivalent diameter is used, which corresponds to a diameter of a circle of equivalent cross section.

The state-of-the-art paper on fibrous concrete by ACI Committee 544 recommends aspect ratios from 30 to 150. The projects covered in this report used fibers with aspect ratios in the range of 60 to 100.

It is important to obtain a uniform distribution of fibers during mixing. Gray (1) reported that "it has been the general experience that segregation or balling of fibers during mixing is related to three major factors: the aspect ratio of the fibers, the volume percentage of the fibers, and the mixing procedure." From the standpoint of fiber handling, plant batching, and mixing, it would be desirable to use fibers with the lower aspect ratio; however, from the strength standpoint, it is desirable to use fibers with the higher aspect ratio.

Fiber contents of the mixes used on the experimental projects covered in this report vary from 60 to 265 $lb/yd^3$. The mix design and mixing procedures are covered under the individual projects.

## FIBROUS CONCRETE RESEARCH

Laboratory and field research projects during the past few years indicated that fibrous concrete has great potential advantages for certain applications in the paving field. Since 1967 several pavement slabs have been placed in the field. These early field projects served to develop the techniques for mixing and placing a fibrous concrete mix. Performance of these slabs was promising; however, the installations were usually limited in size because of the availability of fibers and the experimental nature of the early work.

Given the physical properties of fibrous concrete, it is apparent that the material shows great promise as an overlay or resurfacing material.

## GOALS OF FIELD RESEARCH PROJECTS

To determine the future potential for this material requires that the performance characteristics of fibrous concrete in pavement overlays receiving normal mixed highway and airport traffic be determined. To do this, field research projects have been placed by cooperative efforts of federal agencies, airport authorities, state highway departments, city engineering departments, fiber producers, cement manufacturers, Michigan and Iowa Concrete Paving Associations, Portland Cement Association, and American Concrete Paving Association. These projects were designed to determine the following:

1. Thickness requirements for fibrous concrete resurfacing for mixed traffic on streets, highways, and airports;

Figure 1. Broken beam showing distribution of 1-in. fibers in mix.

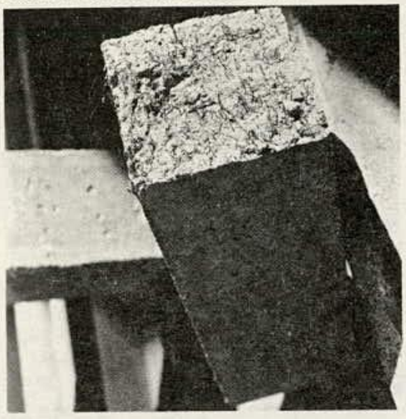


Figure 2. Flat steel fibers produced by shearing steel sheets.

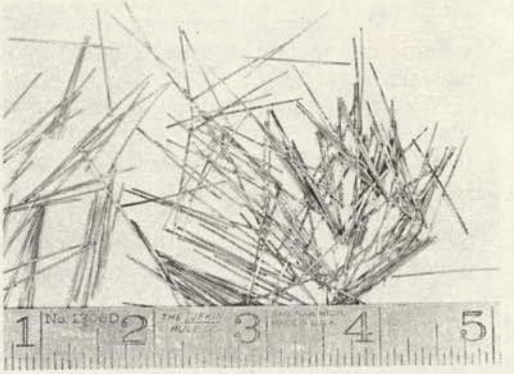



Table 1. Mix design and strength properties of fibrous concrete.

| Property | Six-Inch Slab | Four-Inch Overlay |
|---|---|---|
| Cement factor (type 1), bags/yd³ | 9 | 9 |
| Water-cement ratio | 0.50 | 0.46 |
| Fine-coarse aggregate ratio | 3 | 3 |
| Maximum size coarse aggregate, in. | ³⁄₈ | ³⁄₈ |
| Fiber content, percent (265 lb/yd³) | 2 | 2 |
| Fiber type | Steel | Steel |
| Fiber cross section, in. | 0.016 ⌀ | 0.010 × 0.022 |
| Fiber length, in. | 1.0 | 1.0 |
| Test age, days | 73 | 28 |
| Flexural strength, psi | 940 | 1,140 |
| Modulus of elasticity (flexure), psi × $10^6$ | 5.30 | 5.28 |
| Compressive strength, psi | 5,760 | 6,960 |
| Tensile strength, psi | 760 | 870 |
| Air content, percent | 5.5 | 5.9 |
| Slump, in. | 5 | 3¼ |

Figure 3. Spalling of cracks in base slab before 4-in. fibrous overlay.

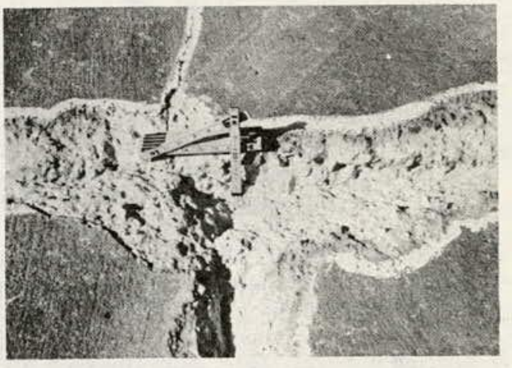


Figure 4. Random (a) longitudinal and (b) transverse cracks in taxiway pavement prior to 6-in. fibrous overlay.

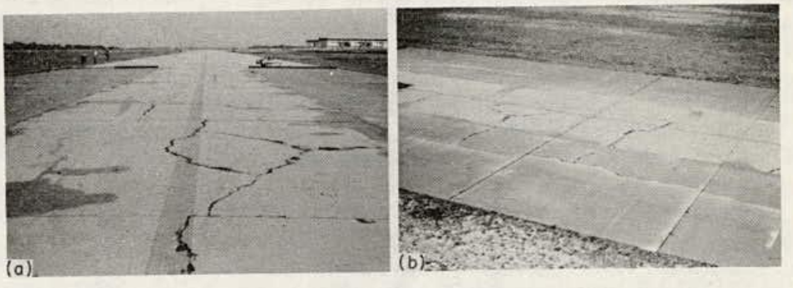



Table 2. Mix design for Tampa Airport fibrous concrete overlays.

| Item | Amount | Volume (ft³) |
|---|---|---|
| Cement (at 5½ sacks/yd³), lb | 517 | 2.63 |
| Fly ash, lb | 225 | 1.44 |
| Fibers (1.5 percent by volume), lb | 200 | 0.41 |
| Coarse aggregate (¾ in. max), lb | 1,200 | 7.66 |
| Fine aggregate | 1,525 | 9.38 |
| Water | 275 | 4.40 |
| Water-cement ratio of fly ash | 0.37 | —[a] |
| Air, oz | 3.3 | |
| Water reducer-retarder, oz | 38.5 | |
| Air (4 percent) | | 1.08 |
| Total | | 27.00 |

[a]By weight.

Figure 5. Three-quarter-in. aggregate used in fibrous concrete mix for Tampa overlay.

2. Fiber content required for various pavement applications, the influence of traffic volume and weight on fiber content requirements, and the effect of fiber length and type on performance;

3. Mix design required to develop adequate bond between fibers and mortar and the maximum size of coarse aggregate best suited for fiber-reinforced concrete mixes (field research projects have generally used ⅜-in. coarse aggregate as maximum, except on the Tampa Airport overlay in which ¾ in. was maximum);

4. Jointing requirements for fibrous concrete placed as a bonded, partially bonded, or unbonded overlay and the jointing requirements for a full-depth fibrous concrete pavement;

5. Placement of a fibrous concrete overlay either directly on a jointed concrete pavement or with a separating layer; and

6. Ability of present equipment to mix and place fibrous concrete and the type of fiber handling methods and equipment required on large-scale operations.

## FIELD RESEARCH PROJECTS

One of the best examples of the potential of fibrous concrete as a resurfacing material is provided by the heavy load test project conducted by the Corps of Engineers (2) and sponsored by the Construction Engineering Research Laboratory at Champaign, Illinois.

Included in this research project was a 6-in. fibrous concrete slab on a 4-in. sand filter course having a modulus of subgrade reaction of 52 lb/in.$^3$. The test pavements were constructed in June 1971 and included other thicknesses of plain concrete pavement. The test items were trafficked with C5A and dual tandem gear.

One of the test items in this research project was a 10-in. plain pavement that was constructed to test the effects of multiple-wheel heavy gear loading (C5A) on various types of construction joints. After the 10-in. pavement was trafficked to failure, a 4-in. fibrous overlay was placed. Major structural cracks were present, and the cracks had spalled severely (Fig. 3). The 4-in. overlay was of a partially bonded type with the base pavement merely cleaned and moistened prior to overlaying. The overlay was cast monolithically over the entire 50- by 50-ft base pavement. Table 1 gives mix design information for both the slab on grade and the 4-in. resurfacing.

The 10-in. base pavement failed at 950 repetitions of C5A loadings. After the 4-in. overlay was placed, 900 additional load repetitions were applied before the first crack became visible. After 6,900 load repetitions, the test was discontinued, and one working crack was noted. The hairline cracks that developed as loads were applied were located over cracks in the broken base pavement. This certainly is evidence of outstanding performance of a thin overlay on a structurally overloaded base pavement.

The 6-in. fibrous concrete pavement on the weak foundation ($K = 52$ lb/in.$^3$) withstood 350 load applications of C5A gear before the first visible crack occurred. A number of hairline cracks were evident at 8,700 repetitions, which would not interfere with normal aircraft operations on an in-use pavement. Only one working crack was evident at this time and was located near the center of the slab in a transverse direction.

The measurements of deflections, strains, and subgrade pressures along with observations of the performance under loadings of these test items will provide some of the information necessary to develop a design procedure for fiber-reinforced concrete pavements and overlays.

The Corps of Engineers is continuing to develop design information for fibrous concrete pavements by additional test sections including a 7-in. fibrous concrete slab on a membrane-encapsulated soil and a 4-in. pavement on 17 inches of cement-stabilized clay-gravel. In June 1973 a 1,000-ft section of unjointed fibrous concrete pavement was placed at the Waterways Experiment Station in Vicksburg. The pavement was 4 inches thick and 24 feet wide. The purpose of this installation was to study the crack pattern that would develop in long sections of unjointed fibrous concrete.

## TAMPA INTERNATIONAL AIRPORT PROJECT

Two fibrous concrete overlay sections were placed on a taxiway at Tampa International Airport with the cooperative efforts of the local airport authority, FAA, Corps of Engineers, U.S. Steel Corporation, and a local contractor. The taxiway is used by mixed aircraft traffic including the Boeing 747. The 25-ft-wide taxiway lanes had midpanel random longitudinal cracks and some random transverse cracks as shown in Figure 4. It appeared that a subgrade problem existed in this area, and a slight amount of faulting was evident on some of the cracks.

The two sections were placed directly on the old pavement after cleaning and prewetting. One section consisted of three 25-ft lanes, 175 feet long and 6 inches thick, with butt types of longitudinal joints. The longitudinal construction joints matched the base pavement construction joints; however, no transverse joints were cut in the 175-ft-long overlay.

The other section was 4 inches thick and 50 feet square placed in two 25-ft lanes with the longitudinal construction joints in the base pavement centered under the overlay pavement lanes.

### Mix Design

The mix design on this project departed significantly from that on other experimental fibrous concrete pavement projects. The maximum size coarse aggregate was $^3/_4$ inch (Fig. 5) instead of the $^3/_8$ inch used on other experimental projects. The mix design called for $5^1/_2$ sacks of cement per cubic yard and 225 pounds of fly ash. The mix design proportions for a 1-yd$^2$ batch are given in Table 2.

The fibers used on this project were 1-in.-long flat fibers of 10- by 22-mil cross section. The flexural strength of the concrete was 765 psi at 7 days, 830 psi at 28 days, and 1,010 psi at 90 days.

### Mixing, Placing, and Finishing

The fibers were furnished in 40-lb boxes and were manually fed onto a conveyor belt that deposited the fibers on top of the aggregates on the charging belt to the mixer. The mixer was a 9-yd$^3$ Rex tilting mixer equipped with a horizontal premix drum. The concrete was mixed in 4-yd$^3$ batches to match the capacity of the temporary fly ash handling system. Fiber distribution in the mix was excellent with no balling evident in the mixed concrete. The mixing plant is shown in Figure 6.

The concrete was transported to the paving site in side-dump haul units and spread with a box spreader. A CMI slip-form paver was used to strike off, consolidate, and finish the pavement. A tube float followed the slip-form paver, and the texture was applied with a wire comb. A white-pigmented cure was applied to the pavement. The middle lane of the 6-in. section was textured with a stiff-bristled broom. The procedure is shown in Figure 7.

The significance of this project from the standpoint of pavements is that it demonstrated modern high-capacity paving equipment capable of mixing, hauling, placing, and finishing fibrous concrete. The project also illustrated the need for development of some method of bulk handling of fibers. Mix design changes were also significant.

## DETROIT EXPERIMENTAL PROJECT

In October 1972, a 1,100-ft demonstration project was placed on a major urban expressway in Detroit, Michigan. Eight Mile Road is an eight-lane divided highway carrying 100,000 vehicles per day, 18 percent of which is heavy commercial traffic. This project was sponsored by members of the Michigan Concrete Paving Association and the American Concrete Paving Association in cooperation with the Michigan Department of State Highways and the Federal Highway Administration.

This was the largest area of fibrous concrete pavement that had been placed to that date, and it was the first time high production paving equipment was used to pave with fibrous concrete on a highway project.

The 48-ft-wide overlay was placed on an existing concrete pavement consisting of two 10-ft lanes constructed in 1932, one 12-ft lane built in 1954, and 16 feet of new concrete base widening added in 1972. The condition of the base pavement is shown in Figure 8. Surface preparation consisted of blowing out cracks, sweeping with a power broom, and then wetting the surface before the fibrous overlay was placed.

### Mix Design

A water reducer-retarder was used during the first paving day and a straight water reducer the second. These proportions were altered slightly for job conditions and for the changes in fiber content. Half the concrete placed each day contained 120 lb of steel fibers/yd$^3$ and the other half contained 200 lb/yd$^3$. The mix design was as follows:

| Item | Amount |
|---|---|
| Cement, lb | 850 |
| Coarse aggregate, slag (½-in. max), lb | 906 |
| Sand, lb | 1570 |
| Water (34 gal), lb | 283 |
| Fibers ($0.016 \times 0.010 \times 1$ in.), lb/yd$^3$ | 120 and 200 |

### Equipment

The fibrous concrete was produced in a central mix plant with a 9-yd mixer. Batch size was limited to 6 yards. Agitating trucks transported the concrete to the paving site, about a mile away, and spread it directly on the existing concrete ahead of the paver. The slip-form paver (Fig. 9) was locked to a string line for electronic control of line and grade. A texturing-curing machine with a wire comb textured the surface transversely and applied white-pigmented curing compound (Fig. 10). A tube float machine was used on the first paving day but not the second.

Concrete saws cut a longitudinal joint in the center of each 24-ft-wide placement. On one section of pavement a longitudinal joint was not cut in order to determine whether cracking would develop. The transverse joints were cut by using abrasive blades the first day and diamond blades the second. In each case the joints were sawed the day after paving. Some of the transverse joints were ¾ inch wide spaced at 100 feet, and others were ½ inch wide spaced at 50 feet. No attempt was made to match the joints in the existing concrete pavement. Preformed compression seals (Fig. 11) were installed in the transverse joints, and hot-poured sealant was used in the longitudinal joints.

### Fiber Handling

As on other fibrous concrete paving projects to date, the steel fibers were delivered in small boxes in 40-lb lots. Workers opened the boxes of fibers and dumped the amount

Figure 6. Central mix plant with horizontal premix drum (note fiber charging belt at extreme right).

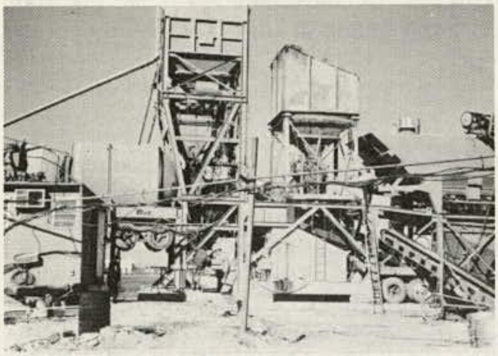


Figure 7. Paving train on 6-in. fibrous overlay.

Figure 8. Condition of base pavement prior to fibrous overlay.

Figure 9. Slip-form paver used on Michigan fibrous overlay.

Figure 10. Equipment used to texture and apply curing compound.

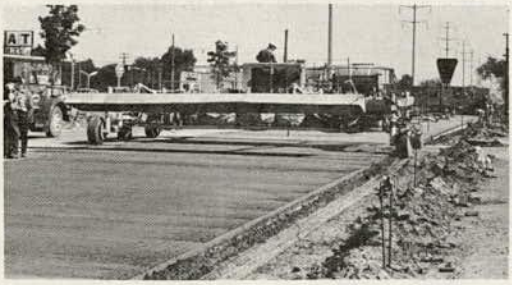
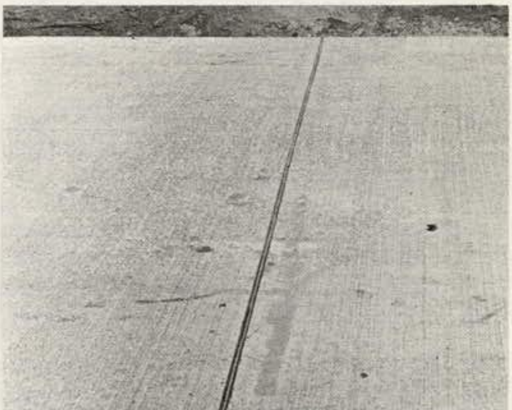


Figure 11. Preformed compression seal in transverse joint.



required for one batch into the bucket of an end-loader. The loader placed the fibers on a table next to a belt feeder where workers, using hand rakes and forks, fed them through an 8-ft-long grizzly onto a high-speed belt. This belt discharged onto the main plant belt, which fed the fibers and aggregates into the mixer drum. The batch cycle generally ranged from 4 to 6 minutes.

### Construction Schedule

On October 7, 1972, two of the four eastbound lanes were closed to traffic, and a slip-form paver resurfaced the existing concrete with 3 inches of fibrous concrete. Two days later, the new overlay was opened to traffic, and the remaining eastbound lanes were resurfaced. In two more days, all four lanes were back in service.

This overlay having different fiber contents will be subjected to a heavy volume of mixed highway loadings to determine the effect of fiber content on performance. This project is the first one of sufficient length to study the jointing requirements of a fibrous concrete overlay placed directly on a concrete base pavement.

## CEDAR RAPIDS EXPERIMENTAL PROJECT

Four fibrous concrete overlays were constructed in fall of 1972 in Cedar Rapids, Iowa. These projects included the overlaying of an airport taxiway, an arterial street, a residential street, and a bridge. The projects were cosponsored by the Iowa Concrete Paving Association and Battelle Development Corporation in cooperation with the City of Cedar Rapids.

### Equipment

The concrete was supplied by a local ready-mix firm. The firm experimented in developing the sequence for charging the truck mixers so as to minimize the formation of balls in the concrete and finally settled on charging 70 percent of the water and all aggregates and then adding the steel fibers. During charging of the fibers, it became necessary to run the truck mixer at mixing speed to prevent balling of the fibers. For each batch the fibers were spread evenly on a table on the second level of the plant and were pushed by hand through a vibrating screen into a chute that charged the mixers. It took from 5 to 15 minutes to charge the fibers depending on fiber size and content. After the fibers were mixed with the aggregates, the cement and the remainder of the water were added. The batch size was limited to 5 yards for the 7-yd mixers.

### Concrete Mixes

Aggregates for the fibrous concrete were $3/8$-in. pea gravel and river sand. A typical mix design for the projects is that for the portion of the airport taxiway in which $2\frac{1}{2}$-in.-long fibers were used. It included 752 lb (8 bags) of cement, 750 lb of coarse aggregate, 1,848 lb of fine aggregate, 346 lb of water, 150 lb of steel fiber, and 6 percent air. A water reducer-retarder was used in the airport concrete and a straight water reducer on the other projects. Concrete consistency was maintained at a slump of about 4 inches.

### Placing Equipment

The contractor used a variable-width bridge deck paver (Fig. 12) to place and finish the concrete. Workers spread the concrete ahead of the paver with rakes and shovels after the base pavement had been swept and sprinkled with water. Hand finishers behind the paver used aluminum straightedges for smoothing the surface and applied the surface texture with a hand broom. White-pigmented curing compound was sprayed on the finished surface at the airport, but on the street projects the concrete was cured by emulsified linseed oil.

Figure 12. Bridge deck paver used to place concrete on Cedar Rapids project.

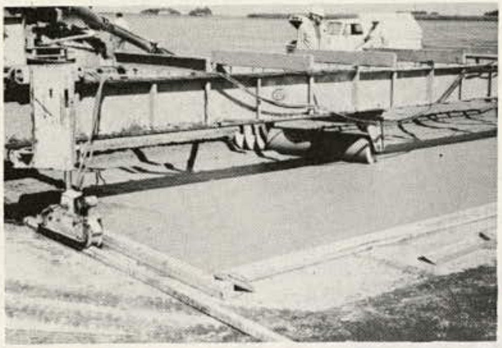


Figure 13. Crack pattern on Greene County project.

Table 3. Greene County fibrous concrete experimental resurfacing.

| Section No. | Section Length (ft) | Pavement Type | Pavement Thickness (in.) | Cement Content (lb) | Fiber Length (in.) | Fiber Content (lb) | Pavement Bonding | Transverse Joints | Longitudinal Joints | Comments |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 450 | A-4 | 5 | 569 | — | — | Partial | At 20 feet | Yes | Dowels at 3 ft c. to c. |
| 2 | 450 | A-4 | 4 | 569 | — | — | Partial | At 30 feet | Yes | 6 × 6 mesh |
| 3 | 200 | A-4 | 4 | 569 | — | — | Bonded | See plan | Yes | CRC anchor section |
| 4 | 600 | A-4 | 4 | 569 | — | — | Unbonded | See plan | Yes | CRC elastic joints |
| None | 100 | A-4 | 3 to 4 | 569 | — | — | Unbonded | See plan | Yes | Transition section |
| 5 | 600 | A-4 | 3 | 569 | — | — | Unbonded | See plan | Yes | CRC elastic joints |
| 6 | 200 | A-4 | 3 | 569 | — | — | Bonded | See plan | Yes | CRC anchor section |
| 7 | 400 | Fibrous | 3 | 600 | 1 | 60 | Partial | At 40 feet | Yes | Transverse joints cut full depth |
| 8 | 400 | Fibrous | 3 | 750 | $2\frac{1}{2}$ | 60 | Partial | At 40 feet | Yes | 250 ft of $2\frac{1}{2}$-in. fibers; remainder 1 in. |
| 9 | 400 | Fibrous | 3 | 600 | 1 | 100 | Partial | At 40 feet | Yes | Transverse joints cut full depth |
| 10 | 400 | Fibrous | 3 | 750 | 1 | 100 | Partial | At 40 feet | Yes | |
| 11 | 400 | Fibrous | 3 | 750 | $2\frac{1}{2}$ | 100 | Unbonded | At 40 feet | Yes | |
| 12 | 400 | Fibrous | 3 | 750 | 1 | 100 | Bonded | At 40 feet | No | |
| 13 | 400 | Fibrous | 3 | 600 | 1 | 60 | Partial | At 40 feet | Yes | |
| 14 | 400 | Fibrous | 3 | 500 + 234 FA | 1 | 100 | | | | |
| 15 | 400 | Fibrous | 3 | 500 + 234 FA | $2\frac{1}{2}$ | 100 | Partial | At 40 feet | Yes | |
| 16 | 400 | Fibrous | 3 | 600 | $2\frac{1}{2}$ | 60 | Partial | At 40 feet | Yes | |
| 17 | 400 | Fibrous | 3 | 750 | 1 | 60 | Partial | At 40 feet | Yes | |
| 18 | 400 | Fibrous | 3 | 600 | 1 | 160 | Partial | At 40 feet | Yes | |
| 19 | 400 | Fibrous | 3 | 750 | 1 | 160 | Partial | At 40 feet | Yes | |
| 20 | 400 | Fibrous | 3 | 750 | $2\frac{1}{2}$ | 160 | Unbonded | At 40 feet | Yes | |
| 21 | 400 | Fibrous | 3 | 750 | $2\frac{1}{2}$ | 100 | Bonded | At 40 feet | Yes | |
| 22 | 263.1 | Fibrous | 3 | 500 + 234 FA | 1 | 160 | — | See plan | Yes | On grade |
| 23 | 159.7 | Fibrous | 3 | 750 | 1 | 160 | Bonded | See plan | | Bridge |
| 24 | 499.2 | Fibrous | 3 | 600 | 1 | 100 | Partial | See plan | Yes | Curb section |
| 25 | 478 | Fibrous | 3 | 750 Chem Comp | $2\frac{1}{2}$ | 100 | Unbonded | At 40 feet | No | |
| 26 | 400 | Fibrous | 2 | 750 | $2\frac{1}{2}$ | 160 | Partial | At 40 feet | Yes | |
| 27 | 400 | Fibrous | 2 | 600 | 1 | 100 | Partial | At 40 feet | Yes | |
| 28 | 400 | Fibrous | 2 | 750 | 1 | 100 | Partial | At 40 feet | Yes | Transverse joints cut full depth |
| 29 | 400 | Fibrous | 2 | 750 | 1 | 100 | Bonded | At 40 feet | Yes | |
| 30 | 400 | Fibrous | 2 | 750 | 1 | 160 | Partial | At 40 feet | Yes | |
| 31 | 400 | Fibrous | 2 | 600 | 1 | 100 | Partial | At 40 feet | No | |
| 32 | 400 | Fibrous | 2 | 750 | 1 | 100 | Partial | At 40 feet | No | |
| 33 | 400 | Fibrous | 2 | 600 | 1 | 160 | Partial | At 40 feet | Yes | |
| 34 | 400 | Fibrous | 2 | 750 | 1 | 160 | Partial | At 40 feet | Yes | |
| 35 | 400 | Fibrous | 2 | 750 | $2\frac{1}{2}$ | 100 | Unbonded | At 40 feet | Yes | |
| 36 | 400 | Fibrous | 2 | 750 | $2\frac{1}{2}$ | 100 | Bonded | At 40 feet | Yes | |
| 37 | 400 | Fibrous | 3 | 600 | $2\frac{1}{2}$ | 60 | Partial | At 40 feet | No | |
| 38 | 400 | A-4 | 4 | 569 | — | — | Partial | At 30 feet | Yes | 6 × 6 mesh |
| 39 | 400 | A-4 | 5 | 569 | — | — | Partial | At 20 feet | Yes | Dowels at 3 ft c. to c. |
| 40 | 200 | Fibrous | 3 | 500 + 234 FA | 1 | 100 | Partial | Various | No | |
| 40A | 218.1 | Fibrous | 3 | 500 + 234 FA | 1 | 160 | — | At 40 feet | Yes | On grade |

Information on the individual projects in Cedar Rapids follows.

Airport—The airport overlay varies from 1 to 3 inches thick across a 75-ft-wide concrete taxiway. It was paved in 37½-ft passes. The base concrete has longitudinal joints at 12½-ft intervals and transverse joints spaced at 20 feet. The overlay pavement has no joints except the longitudinal construction joint in the center.

Two concrete mixes were used. One was a nine-bag mix with 200 pounds of 1-in.-long, 16-mil-diameter steel fibers per cubic yard. The other was an eight-bag mix with 150 pounds of 2½-in.-long, 25-mil fibers per cubic yard.

Arterial—A 200-ft-long overlay, 25 feet wide and 2½ inches thick, was placed on Fifth Avenue, a heavily traveled one-way street in Cedar Rapids. The base was an asphalt surface on a brick pavement. Streetcar tracks down the center had reflected through the surface. The fibrous overlay was not jointed.

A nine-bag mix was used on this project. One-inch-long, 16-mil steel fibers were added at the rate of 250 lb/yd$^3$ for half the section and 200 lb/yd$^3$ for the rest.

Bridge Deck—A 3-in.-thick, 12-ft-wide, 150-ft-long fibrous overlay was placed on a wooden plank bridge deck that had been cleaned of loose asphalt and covered with a double layer of polyethylene sheeting. There were no joints in the overlay. Two and a half-inch-long, 25-mil steel fibers were added at 150 lb/yd$^3$ to the eight-bag concrete mix. The traffic count was 30,000 vehicles per day.

Residential Street—On October 5 a 28-ft-wide, badly scaled old concrete pavement on Danbury Street was resurfaced from curb to curb with 2½ inches of fibrous concrete for a length of 150 feet. One-inch-long, 16-mil fibers were added at 175 lb/yd$^3$ to the eight-bag concrete mix used on this overlay.

These projects will serve to provide information on the relative effect of fiber length and content on performance. The projects will also provide information on the effect of different traffic volumes and weights on performance.

## GREENE COUNTY, IOWA, EXPERIMENTAL PROJECT

One of the most ambitious projects planned to date is an experimental fibrous overlay project in Greene County, Iowa. The Greene County engineer, along with the Iowa Research Council, the Iowa Concrete Paving Association, and the concrete paving industry, conducted a full-scale (3.03 miles) fibrous concrete overlay research project. The purpose is to study and evaluate the relative performance of various fibrous concrete overlay designs and to compare their performance to that of other concrete overlay designs. The research project involved the widening and resurfacing of an unjointed concrete pavement built in 1920-21. The pavement crack pattern is shown in Figure 13.

Variables in the fibrous concrete sections included overlay thickness (2 and 3 inches), cement content (600 and 750 lb/yd$^3$), fiber content (60, 100, and 160 lb/yd$^3$), fiber length (1 and 2½ inches) and type, and condition of bond with the old slab. Special sections included one that used shrinkage-compensating cement and one with a partial replacement of cement with fly ash. The mix designs developed by Kesler at the University of Illinois are based on 7-day flexural strengths of 600, 800, and 1,000 psi.

Sections of 3- and 4-in. continuously reinforced overlay with elastic joints were included in the study. The joints were formed at 8-ft intervals by placing parting strips on the old pavement, and a bond breaker was applied to the reinforcing steel at the joint locations. Control sections of 5-in. conventional plain and 4-in. mesh-reinforced concrete overlay were included.

Joint spacing on the fibrous concrete sections was at 40 feet. Six of the test sections had no longitudinal joint. Two sections had neoprene preformed seals in the transverse joints. All other joints had hot-poured sealant. Lean-mix concrete was used for widening the existing 18-ft pavement to 22 feet.

This project is the first one of sufficient length to compare the effect of variations in thickness, fiber content and length, concrete strengths, and placement conditions (bonded, unbonded, or partially bonded). Table 3 gives details of the test sections built under this Greene County research project.

## SUMMARY

The field research projects discussed in this report along with laboratory research projects will serve to provide the information necessary to develop the design parameters for fibrous concrete pavement such as thickness, jointing, fiber content, fiber geometry, mix design, and the construction procedures necessary for this new material.

## REFERENCES

1. Gray, B. H. A Discussion of Field Considerations. Paper presented at conference on fibrous concrete, May 1972, Corps of Engineers, Champaign, Ill.
2. Gray, B. H., and Rice, J. L. Fibrous Concrete for Pavement Applications. Construction Engineering Research Laboratory, Department of the Army, Champaign, Ill., Rept. M-13, April 1972.

# part 3

# construction innovations

# computerized quality control

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This paper discusses the application of a computerized quality control package to a construction project in Utah. The package, comprising seven computer programs, was based on the premise that quality of bituminous pavements can be controlled by regulating bitumen content, aggregate gradation, relative compaction, and surface variation. The seven computer programs are described. As a result of this pilot project, Utah has begun statewide implementation of this quality control system.

•The quality of materials and workmanship used in construction has long been of significance to the highway engineer. For this reason, much effort has been directed toward development of specifications that control the quality of construction without forcing the costs of this construction to exceed reasonable limits. No specification, however, can be more effective than the material test on which it is based, and the results of this test are of no value until they reach the project engineer and the contractor. If the results of a test that show nonconformance to specifications were not made known to the project engineer and the contractor in time, a whole section of roadway might have to be either accepted at a low level of quality or removed at great expense. Problems of this type, as well as others, inherent in present quality control methods can be minimized by using revised specifications and testing procedures in conjunction with a computerized quality control system. A pilot project using such a system has recently been completed in Utah. The results of that project as well as some of the characteristics of the quality control system itself are discussed. In addition, the new quality control system being instituted in Utah, which was a direct result of this pilot project, will be covered briefly.

The feasibility of establishing a computerized quality control system with its associated specifications and test methods in Utah was tested by setting up a pilot project for construction of a 5.3-mile section of Interstate 80N near Tremonton in the northern part of the state.

## THE SYSTEM AND ITS USE

The specifications and computer programs used at Tremonton were modified versions of a quality assurance computer program package obtained from the Federal Highway Administration. The package, which will be discussed in detail, contained several programs using punched cards as input-output media. The input cards were to be punched in the field and were to contain records of daily test results. They were then to be delivered to a computer terminal for processing and the results of the computer analysis were to be delivered back to the project engineer. Because the project was relatively remote from an available computer system and because of the problems associated with using punched cards for data records in a project of this type, an alternative solution was devised. The computer programs were converted to operate on a commercial time-sharing computer system, and a teleprinting computer terminal was placed in the project engineer's office near the construction site. This allowed the engineer or members of his staff to build daily data files, call the program wanted, and receive almost instantaneous results on the conformance to specifications on the project.

The project presented some new experiences for the engineering staff and the contractor. The project engineer who had graciously consented to take on this pilot program spent many extra hours in the first few weeks learning to communicate to the computer and learning how to get the computer to respond in meaningful terms. Because the computer programs were hurriedly converted and were not specifically designed to operate on a time-sharing system, the data input requirements were somewhat more cumbersome than normal. However, it was not long before the engineering staff had mastered the new system and was making meaningful comments about future refinements to the program package. The contractor too had his problems initially. He set up operations and began placing material only to receive a 60 percent reduction in payment for the first 2 days' operation. Upon learning of the reduction, he promptly shut down operations on the project; and, when they were resumed several days later, the specifications were being met. After the initial shock of the new system was overcome, the project ran rather smoothly except for a few minor problems, which were to be expected in an experimental project of this nature.

As was mentioned previously, the original quality control package was modified before the beginning of the pilot project to operate on a time-sharing system, and later it was again modified to more effectively serve the project engineering staff. These modifications were almost entirely in the input-output functions of the computer programs. The basic analytic and deterministic processes of the package were not altered. A brief discussion of the programs in the package and how they combine to provide a total quality control system for bituminous concrete follows.

## QUALITY CONTROL PACKAGE

The package is based on the premise that the quality of bituminous pavements can be controlled by regulating four characteristics: bitumen content, aggregate gradation, relative compaction, and surface variation. This is done by using the first three programs of the seven in the package.

The vacuum extraction program computes the results of each vacuum extraction test made on the bituminous pavement. The averages of these results are compared to target values for each sieve previously agreed on by the project engineer and contractor. The deviation of the results from these target values determines the adjustment that is made to the unit bid price. This deviation depends on the particular sieve and on the number of tests taken from the test lot. The program produces a lot summary of the tests taken on each production day. The program prints out the lot averages for percentage of bitumen, percentage passing each sieve, unit bid price adjustment, and amount of the reduction for the day. Input for the program is placed into an input file from the remote terminal located in the project office. Output is provided on the same device.

The nuclear density program computes the relative compaction of material by using as input the raw measured data from a portable nuclear gauge and its gauge constants. A control strip of pavement is constructed, and 10 nuclear readings are taken. The average density of the readings is used as a target value for the remaining material in the test lot until another control strip is constructed. Core and Marshall density tests can also be taken to ensure that the target density is the required percentage of laboratory density. The measured densities on the pavement are compared to the target values to obtain the relative compaction of the material and the subsequent penalty to be assessed if any. The program produces a summary of the control strip data including core and Marshall data. It also produces a table of density tests with their associated percentages of compaction and a lot summary containing the average density, target density, percentage of compaction, and pay factor. Input methods are similar to those of the vacuum extraction program.

The smoothness program computes the percentage of nonconforming tolerances in pavement surface measurements by using a profilograph. The percentage of nonconformance for each day is compared with an adjustment schedule for percentage of nonconformance to determine the unit bid price adjustment. The program produces a unit bid price adjustment schedule and a summary of each production day including lengths and sublots measured, lengths of sublots exceeding $^3/_{10}$ inch of surface variation in 25 feet, and the percentage of nonconformance and its associated pay factor.

The vacuum extraction, nuclear density, and smoothness programs also provide information that goes directly into storage files located in the computer system. These files can be used to provide summary data for a project, but their main function is to provide input data that are used by the other programs in the package.

The fourth program in the package is the total program. This program summarizes the data from the storage files of the first three programs and computes a single pay factor and a cumulative tonnage for each production day. The program produces a summary table for the period of time desired. The summary table contains the production day, the daily tonnage, pay factors for each program, and the total pay factor for each day.

The final three programs are statistical analysis programs that use storage file contents from the vacuum extraction program. The programs are analysis of variance, standard deviation, and a graphical program called histogram. The analysis of variance program analyzes the duplicate samples and gives the overall variance and the components of that variance. Other statistical parameters are given for both the production process and testing errors. The standard deviation program summarizes and computes the average, standard deviation, sum, and sum of square values for the differences between the test values and the target values. The histogram shows a graphical summary of the distribution of deviations from the target values for the vacuum extraction test.

## FURTHER COMMENTS

Thus, the programs that make up the quality control package act as a unit to provide the project engineer with information to help him regulate those characteristics that are essential to production of quality bituminous pavements and to provide this information soon enough to eliminate the possibility that large amounts of substandard work are completed.

As a result of the pilot project at Tremonton, Utah has begun statewide implementation of this quality control system. The specifications used in the new system have been modified based on the findings of the pilot project and have received endorsement from the Association of General Contractors. The most significant change perhaps is that Utah has purchased a new computer system, an IBM 370/155, and along with this system there are plans to place a remote-access terminal in each project office throughout the state. Not only will this provide the benefits associated with this quality control package, but also it will allow an engineer to use the capabilities of a large computer system in other areas such as design and earthwork calculations.

The computer has, to one degree or another, established itself in virtually every phase of highway engineering. It ranges from small one-time analysis programs to unbelievably large data storage systems. In Utah one of the last thresholds to be crossed by the computer was in construction. Now that the first step has been taken, it is inevitable that computers will become more and more valuable to those engineers and technicians involved in the construction area. With respect to quality control, as test methods are developed that permit more rapid detection of nonconformance in concrete pavements and bridge decks, the computer can provide the same advantages it does for bituminous pavements. The computer can take over all routine or redundant correspondence or paperwork associated with construction. It can provide information for various levels of management or for designers and those engaged in research. Thus, the only limitation to this engineering tool is the failure of the engineer to use it.

# use of shale in embankments

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Guidelines for the design and construction of soil embankments are sufficiently developed so that unsatisfactory performance of these fills is relatively rare. The same is true for rock fills. However, there are transition materials or "soft rocks" for which placement in large chunks may lead to highly unsatisfactory embankment performance. Shales are a prominent example, for the large pieces may degrade (slake) into soil in service. This soil may in turn sift down into the large voids, resulting in large settlements and even slope instability. The harder and more durable shales can probably be placed as rock fills if certain precautions are taken. The shales of very low durability must be thoroughly degraded at the end of compaction; i.e., they must be treated as soil fills. And a full spectrum of durabilities exists between these limits. The engineer obviously needs a classification system that will establish where, in the possible range of relative durabilities, a potential embankment shale lies. Such a classification for Indiana shales was developed by sampling materials and subjecting them to a battery of durability, stability, and other tests. The durability tests were the standard ones used for mineral aggregates, but were modified in severity to account for the soft rock being evaluated. It was concluded that the desired classification into four groupings, soil-like, intermediate-1, intermediate-2, and rock-like shales, could be accomplished with four simple tests: one-cycle slaking in water, slake-durability on an initially dry sample, slake-durability on a soaked sample, and a modified sodium sulfate soundness test. The paper describes the Indiana shales tested, the tests, and the response of the shales to the tests. It concludes with a flow chart showing how the tests are used to accomplish the shale classification.

•Highway embankments are commonly built with soil and less commonly with rock. However, in either case, design standards and construction specifications are backed with sufficient experience to be applied with confidence. But what do we know about the family of construction materials between soil and rock, i.e., the "soft rocks"?

Soft rocks include all types of mudrock, which is any sedimentary rock containing at least 50 percent silt and clay constituents. Mudrock is thus a general name for all varieties of siltrock, clayrock, mudstone, siltstone, mudshale, silt shale, clay shale, and argillite. Twenhofel (14), Underwood (15), Ingram (9), and Gamble (6) have differentiated among these rocks. Figure 1 shows an example classification. This paper concentrates on the shales.

When shales are used as embankment materials, the engineer tends to view them with suspicion and often recommends conservative design and construction procedures, e.g., fragmenting the material by extra rolling, placing another material between the shale and the atmosphere (encasement), flattening slopes, inserting special drains, and using berms. These procedures have reduced, but not eliminated, instabilities of shale embankments (8). However, the current practice is probably too conservative. Some usable shales are being wasted, and the strengths of relatively high-quality shales are not being used.

Shales can be grouped in the following four categories:

1. Highly susceptible to postconstructional degradation and, when degraded, inferior in performance to normal fine-grained soils (use of these materials in embankments should be restricted);
2. Similar to normal fine-grained soils and usable with common soil design and construction controls if thoroughly degraded in the construction process;
3. Imperfectly degraded in the construction process, only slightly degraded in service, stronger than soils, but not placeable as rock fill; and
4. Difficult to degrade and probably placeable as rock fill (these are intrinsically superior to soil in fills if certain construction problems can be overcome).

This paper reviews the current placement technology for shale embankments and suggests a simple and inexpensive testing program to classify the shales with respect to their use in embankments.

## PROBLEMS WITH SHALES AS EMBANKMENT MATERIALS

Potential problems within an embankment constructed with shales include: (a) settlement due to loading, drying, slaking, or thawing; (b) heave caused by wetting or freezing; (c) slope instability; and (d) surface and subsurface erosion. (Slaking is the process through which a material disintegrates or crumbles into small particulate units when exposed to moisture and especially when dried and immersed in water.)

The degree to which soft rocks will demonstrate poor performance depends largely on their service environment, both man-made and natural. For example, unless the material becomes significantly wetter than the placement condition, slaking may not occur. Once exposed to increased moisture, slaking may occur quickly, in many years, or not at all. The practical consequence of the slaking, if it occurs, depends primarily on the relative abundance of large voids in the compacted mass into which the slaked material can settle. The size and frequency of large voids are directly related, in turn, to the abundance of large chunks of shale in the embankment. If large chunks of slaking materials are placed in the embankment, major problems can be anticipated. If, on the other hand, the slaking material is reduced to small pieces in the construction process, the subsequent slaking in service may produce no unacceptable densifications or surface displacements.

Degradation of material in the embankment can be controlled by effective drainage or proper encasement of the embankments or both. Even nonslaking materials are weakened and made more compressible by increased moisture. Other shales contain enough expansive minerals to cause significant swelling upon wetting and shrinkage upon drying and potentially harmful effects to the embankment and/or the overlying pavement.

If one is able to assess the general susceptibility of a material to slaking, volume change, and the like in the projected service environment, more rational decisions can be reached in the design and construction processes, thereby increasing the probability that satisfactory service will be produced with economy.

Figure 1. Classification of shale and related rocks.

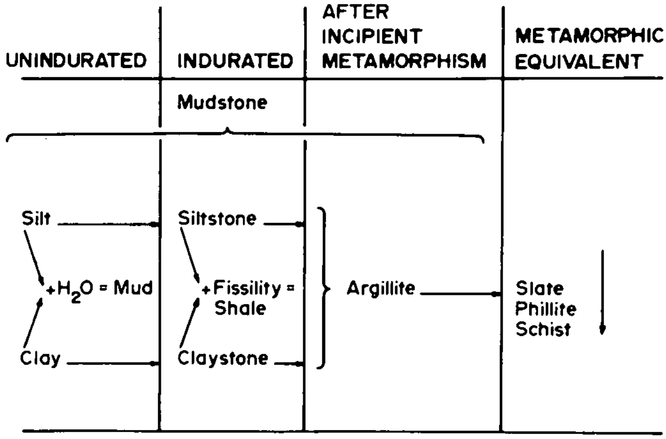
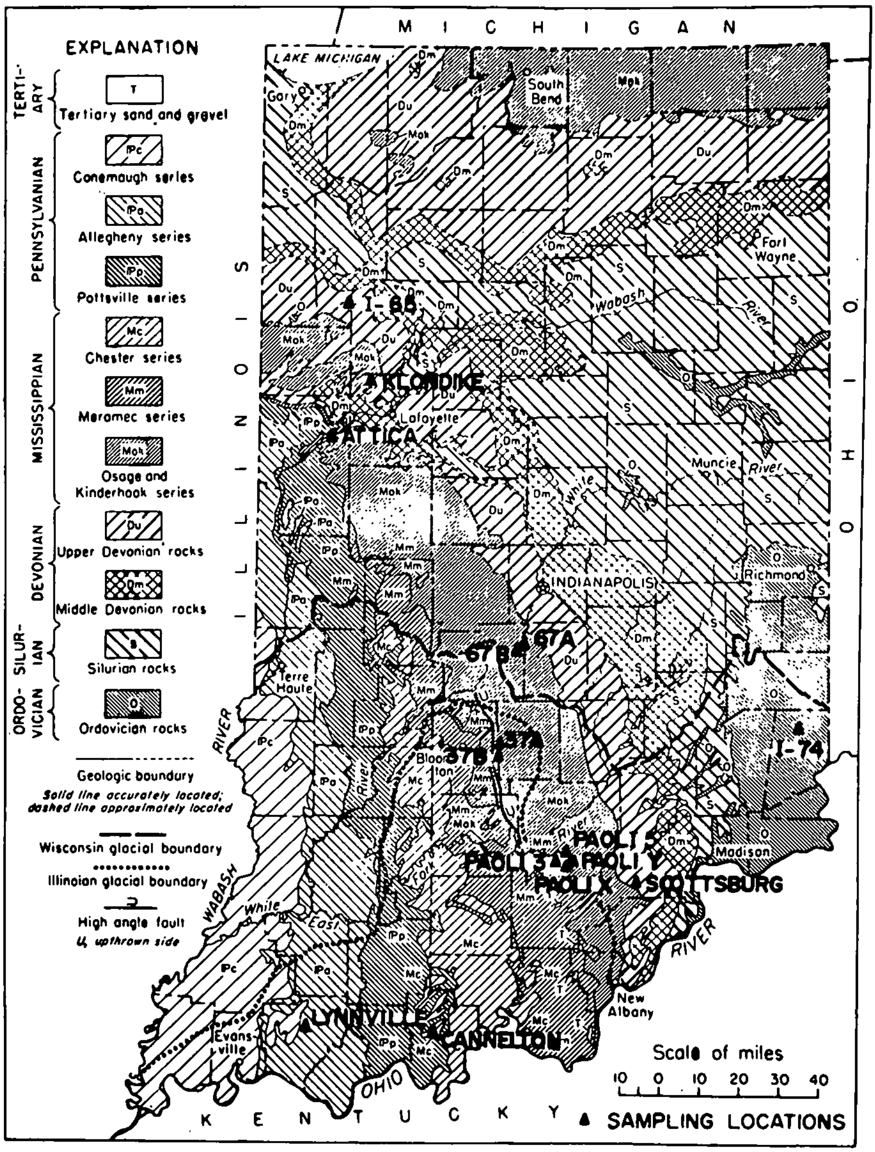



Figure 2. Bedrock geology of Indiana and shale sampling locations.



## CURRENT PLACEMENT TECHNOLOGY

Shales have been treated sometimes as soil and sometimes as rock in embankments. Sherard and others (12) emphasize the importance of proper investigation of these materials and of handling each as an individual problem. Test embankment sections are recommended, where possible.

The various agencies constructing embankments have separate specifications for soils and rocks. However, there may be no fixed specifications for shale or other soft rock embankments. The Indiana State Highway Commission uses shales in embankments with the following provisions:

1. Shales are subjected to thorough breakdown in the process of excavation, hauling, placement, and compaction; i.e., they are treated like soil fill. Occasionally, lift thicknesses are made even thinner than for soil.
2. A nonshale soil encasement of 2 or 3 feet is provided on all boundaries of the embankment.
3. The shale-soil mixture, when treated in the specified manner, is considered to be highly competent, and no other special design features are needed.

Such provisions are normally contained in a special construction specification statement and are often qualitative.

Some agencies, including the Soil Conservation Service in Indiana, use shale in the construction of small dams (13). Durable and nondurable (soil-like) shales are recognized, but there are no quantitative criteria to indicate into which group the shale in question should fall. The Indiana SCS has used durable shales with the following provisions:

1. The maximum size of rock fragments used is 18 inches, provided that such fragments are completely embedded in a matrix of compacted fill;
2. The maximum thickness of rock layers before compaction is 24 inches;
3. Broken shale and limestone mixtures may be used in rock fill;
4. Rock fill has a cover of weather-resistant material of 2 to 4 feet; and
5. A minimum compacted dry unit weight of 112.5 $lb/ft^3$ was used for two different shales [this number could vary for other shales (13)].

For soil-like shales, the following provisions are suggested:

1. A shale that completely slakes in water in a few (about 10) minutes can also be used in embankment, provided that it is thoroughly broken down to soil during excavation, hauling, placement, and compaction;
2. A minimum encasement of 4 feet of nonshale soil is needed; and
3. The unit weight of the fill should be at least 95 percent of the maximum determined by ASTM D698-66T (3).

With the current state of the art, a considerable amount of judgment may be required at the time of construction, and there is a definite potential for undue conservatism and, occasionally, error on the unsafe side.

## ENGINEERING CLASSIFICATION OF SHALES

There is a need to develop a simple and inexpensive testing routine to classify shales with respect to their suitability for use in embankments. With this objective, representative samples of shales were collected from 15 locations within the state of Indiana (Fig. 2). These materials covered a wide behavioral spectrum, from very hard and durable to rapidly weathered into soil.

The tests conducted in the laboratory can be grouped into four categories.

1. Degradation tests measured slaking and other breakdown of the material. Because the standard tests were inappropriate for soft rocks, it was necessary to develop new ones or at least to modify existing ones. This group includes different types of slaking tests (in air, water, and sodium sulfate solution) and abrasion tests.
2. Standard soil identification tests were conducted on thoroughly degraded shales. This group included Atterberg limits, grain size distribution, and X-ray diffraction.
3. Compaction and load-deformation tests, principally California bearing ratios, were performed on as-compacted and soaked samples.
4. Miscellaneous tests included absorption-time, bulk density, and certain breaking characteristics of the materials.

All the tests did not yield useful descriptors for classifying the shale. Accordingly, only certain ones were selected for use in the recommended classification system.[1]

## TEST RESULTS AND DISCUSSION

### Simple Slaking Tests

On the basis of three tests, slaking in air, slaking in water in one cycle, and slaking in water in five cycles, all the sampled shales could be classified into three groups.

| Group | Classification | Shale |
|---|---|---|
| 1 | Severely affected by water; significant slaking | Cannelton, I-74, Paoli Y |
| 2 | Little affected by water after five cycles | Paoli X, I-65 |
| 3 | Unaffected by five cycles of water | Paoli 3, Paoli 5, Lynnville, Attica, 67A, 67B, 37A, 37B, Scottsburg, Klondike |

Those shales that slake significantly in the five-cycle test should be viewed as non-durable. If used in embankment, they should be accorded special treatment. Groups 2 and 3 perform satisfactorily in these tests, but further examination of their characteristics should be undertaken before design and construction details are specified.

### Slake Durability Tests

The values of the slake durability index for dry samples $(I_d)_d$ and for soaked samples $(I_d)_s$ are given in Table 1. An examination of the values reveals the following points.

1. For the shales that completely or partially slake in water, the slake durability index for dry samples also predicts a severe degradation in water. This is true for the Cannelton and I-74 shales.
2. For the shales with $(I_d)_d > 85$, $(I_d)_s$ is probably a better measure. If $(I_d)_s$ is between 0 and 50, the material is highly susceptible to breakdown in water. An $(I_d)_s$ between 50 and 70 represents an intermediate susceptibility to water. Values between 70 and 90 represent materials with fair to good relative durability.

[1]The original manuscript included an appendix that described the procedures for the tests selected. This appendix is available in Xerox form at cost of reproduction and handling from the Transportation Research Board. When ordering, refer to XS-51, TRB Special Report 148.

3. For shales with $(I_d)_s$ values greater than 90 (or perhaps even 85), the test does not distinguish sufficiently among the materials, and other tests are needed if such distinction is desired.

Modified Soundness Test

The results of this test, which seems more effective than others in distinguishing among the harder and more durable shales, are given in Table 2. The values of soundness index $I_s$ range from 0 to 97.2. Inasmuch as this number refers to the percentage retained on the $^5/_{16}$-in. sieve at the conclusion of the test, higher values of $I_s$ refer to more durable shales. When this test was run on a sound, medium-grained limestone, it gave $I_s = 99.2$.

On the basis of this test, the following groupings of materials are suggested:

1. If $I_s < 20$, the material is very susceptible to weathering and should probably be treated like a fine-grained soil.
2. If $I_s$ is between 20 and 50 (perhaps even 70), the material has a relatively high susceptibility to weathering, and the material should probably still be treated as a soil.
3. Materials having values between 90 and 98 are grouped as intermediate-1 and are probably affected little by weathering. Materials having values between 70 and 90 are termed intermediate-2. Both intermediate types can be superior to soil as embankment materials if given adequate treatment in the construction process.
4. If $I_s > 98$ (none was sampled), the material can probably be treated like a rock.

Compaction and Load-Deformation Tests

Table 3 gives the results at optimum moisture content and standard AASHO effort (2) for all the shales.

The comparisons of the values of as-compacted CBR, soaked CBR, and ratio of soaked to as-compacted CBR show that as-compacted CBR varied between 2.1 and 31.8, soaked CBR between 0.0 and 21.8, and soaked to as-compacted between 0.0 and 0.765. It is noted that, for the three materials showing some slaking in water, the values of soaked CBR are 0.0, 0.4, and 1.1, whereas the as-compacted CBR values are 2.1, 6.1, and 8.0. These data imply an extremely weak embankment, should these shales be saturated in service. [The breakdown of the surcharged shale sample when soaked was sufficient to produce the 0.0 value. (The authors have not seen a 0.0 CBR value reported previously.)]

The values of soaked CBR varied between 0.0 and 76.5 percent of the as-compacted CBR. As this ratio becomes small, a closer examination of the special provisions for the use of the shale is indicated, e.g., complete compaction degradation, special drainage, and encasement.

Swelling Behavior

Swelling after 96 hours of soaking was recorded. The maximum size of shale lumps used was ¾ inch, and it was thought that a few of the shale pieces might collapse and show a volume decrease upon 96 hours of soaking. However, no such settlement was noted.

For eight of 15 materials, there was almost no axial swell. At standard AASHO optimum moisture for the remaining materials, axial swell was 0.6, 1.0, 2.9, 3.2, 5.2, 5.4, and 7.8 percent. On both sides of optimum moisture content, swell was less than at optimum moisture. Swell also increased with an increase in compaction effort (molding water content constant) and therefore with an increase in dry density. This is similar to fine-grained soil results.

**Table 1. Values of slake durability index.**

| Sample | $(I_d)_d$ | $(I_d)_s$ |
|---|---|---|
| Cannelton | 24.0 | 0.0 |
| I-74 | 63.0 | 24.5 |
| Paoli Y | 86.1 | 56.2 |
| Paoli X | 88.8 | 68.7 |
| Paoli 5 | 93.8 | 89.1 |
| Lynnville | 93.8 | 87.2 |
| I-65 | 93.2 | 78.5 |
| 67B | 93.8 | 90.1 |
| 67A | 94.9 | 90.3 |
| Paoli 3 | 94.5 | 91.0 |
| Scottsburg | 94.0 | 91.1 |
| 37A | 94.8 | 93.6 |
| Klondike | 94.2 | 91.2 |
| Attica | 95.0 | 93.5 |
| 37B | 95.0 | 93.6 |

**Table 2. Results of modified soundness test.**

| Sample | Percentage Passing $^5/_{16}$-In. Sieve | Soundness Index |
|---|---|---|
| Cannelton | 100 | 0 |
| I-74 | 100 | 0 |
| Paoli Y | 84 | 16 |
| Paolix X | 69 | 31 |
| Paoli 5 | 28 | 72 |
| Lynnville | 14 | 86 |
| I-65 | 19 | 81 |
| 67B | 17 | 83 |
| 67A | 16 | 84 |
| Paoli 3 | 16 | 84 |
| Scottsburg | 15 | 85 |
| 37A | 5.5 | 94.5 |
| Klondike | 5.4 | 94.6 |
| Attica | 5.2 | 94.8 |
| 37B | 2.8 | 97.2 |

**Table 3. Results of CBR test at standard AASHO effort and optimum moisture content.**

| Sample | $\gamma_{d\ max}$ (lb/ft$^3$) | O.M.C. (percent) | As-Compacted CBR | Soaked CBR | $\frac{\text{Soaked CBR}}{\text{As-Compacted CBR}}$ | Swell (percent) |
|---|---|---|---|---|---|---|
| Cannelton | 107.8 | 14.8 | 2.1 | 0.0 | 0.0 | 7.8 |
| I-74 | 117.9 | 13.8 | 8.0 | 1.1 | 13.7 | 5.4 |
| Paoli Y | 107.4 | 16.6 | 6.1 | 0.4 | 6.6 | 5.2 |
| Paoli X | 112.2 | 12.6 | 12.0 | 3.3 | 25.7 | 2.9 |
| Paoli 5 | 117.0 | 10.1 | 19.9 | 6.2 | 31.2 | 1.0 |
| Lynnville | 115.3 | 8.7 | 12.4 | 7.8 | 63.0 | 0.6 |
| I-65 | 117.8 | 10.2 | 21.2 | 8.3 | 39.2 | 3.2 |
| 67B | 119.7 | 7.5 | 29.5 | 15.8 | 53.6 | 0.1 |
| 67A | 119.0 | 7.3 | 28.8 | 15.3 | 53.5 | 0.2 |
| Paoli 3 | 119.2 | 7.2 | 28.2 | 14.7 | 52.0 | 0.2 |
| Scottsburg | 118.2 | 6.9 | 28.4 | 14.5 | 51.0 | 0.0 |
| 37A | 119.6 | 8.2 | 30.2 | 18.3 | 60.5 | 0.0 |
| Klondike | 118.3 | 10.7 | 23.4 | 17.2 | 76.5 | 0.2 |
| Attica | 117.5 | 7.2 | 27.4 | 19.4 | 71.0 | 0.0 |
| 37B | 119.6 | 7.1 | 31.8 | 21.8 | 68.5 | 0.0 |

**Table 4. Fissility characteristics for shales.**

| Sample | Massive (percent) | Flaggy (percent) | Flaky (percent) | Fissility No. |
|---|---|---|---|---|
| Cannelton | 0 | 30 | 70 | 81 |
| I-74 | 10 | 20 | 70 | 77 |
| Paoli Y | 0 | 30 | 70 | 81 |
| Paoli X | 0 | 50 | 50 | 68 |
| Paoli 5 | 10 | 40 | 50 | 64 |
| Lynnville | 20 | 30 | 50 | 61 |
| I-65 | 0 | 50 | 50 | 68 |
| 67B | 10 | 40 | 50 | 64 |
| 67A | 10 | 40 | 50 | 64 |
| Paoli 3 | 30 | 40 | 30 | 44 |
| Scottsburg | 20 | 40 | 40 | 54 |
| 37A | 30 | 50 | 20 | 38 |
| Klondike | 0 | 50 | 50 | 68 |
| Attica | 30 | 60 | 10 | 31 |
| 37B | 30 | 60 | 10 | 31 |

An increase in swell is identified with a decrease in CBR ratio. If results are compared for those shales that give a swell of 1.0 percent or more, there is a linear trend for reduction in CBR ratio with the increase of swell.

Breaking Characteristics

The percentage by weight having massive, flaggy, and flaky proportions, as determined in the shale breaking characteristics test, is given in Table 4.

Flaky and flaggy are two characteristic conditions of fissility, and therefore a fissility index or number should be some weighted sum of the two; e.g., a fissility number could be proportional to percentage by weight flaky plus a constant times percentage by weight flaggy. The flaggy pieces were heavier than flaky pieces when the same amount of breaking effort was applied. Specifically, the weight of flaky pieces varied between 5 and 100 percent of that of the flaggy pieces, and the average weight of flaky pieces was 0.35 times the average weight of flaggy pieces.

Therefore, the fissility number was defined as the sum of percentage flakiness plus 0.35 times percentage flagginess. The values of fissility number for sampled shales ranged between 31 and 68 and are given in Table 4.

## GENERAL DISCUSSION

Several of the degradation tests may be used to distinguish among the various shales. The soaked durability index and the soundness index seem to be valuable for rating shales by their relative durability. They apparently reflect a combined effect of various important characteristics of shale, such as fissility, cementing materials, and amount and type of clay and silt sizes.

Results of compaction and CBR tests on various shales showed a wide range in the values of as-compacted CBR $(CBR)_c$, soaked CBR $(CBR)_s$, the ratio of soaked to as-compacted values R, and the peak density on the standard AASHO compaction curve $\gamma_{d\ max}$. Higher values of $(CBR)_c$ and $\gamma_{d\ max}$ indicate stronger shales. The value of $(CBR)_s$ is an indicator of both in-service strength and durability, and higher values indicate more strength and durability. Higher values of R predict more durable shales. The results of the CBR tests correlate satisfactorily with soundness index and fissility number.

The use of fissility number seems to be helpful in categorizing shales. Higher values of fissility number indicate reduced $(CBR)_c$, $(CBR)_s$, and R values. Thus those shales having higher fissility numbers display reduced durability and strength.

On the basis of four simple degradation tests, shales can apparently be classified as

1. Rock-like shales,
2. Intermediate-1 shales,
3. Intermediate-2 shales, and
4. Soil-like shales.

The flow chart for classification is shown in Figure 3.

## RECOMMENDATIONS AND SUGGESTED CONSTRUCTION PRACTICES

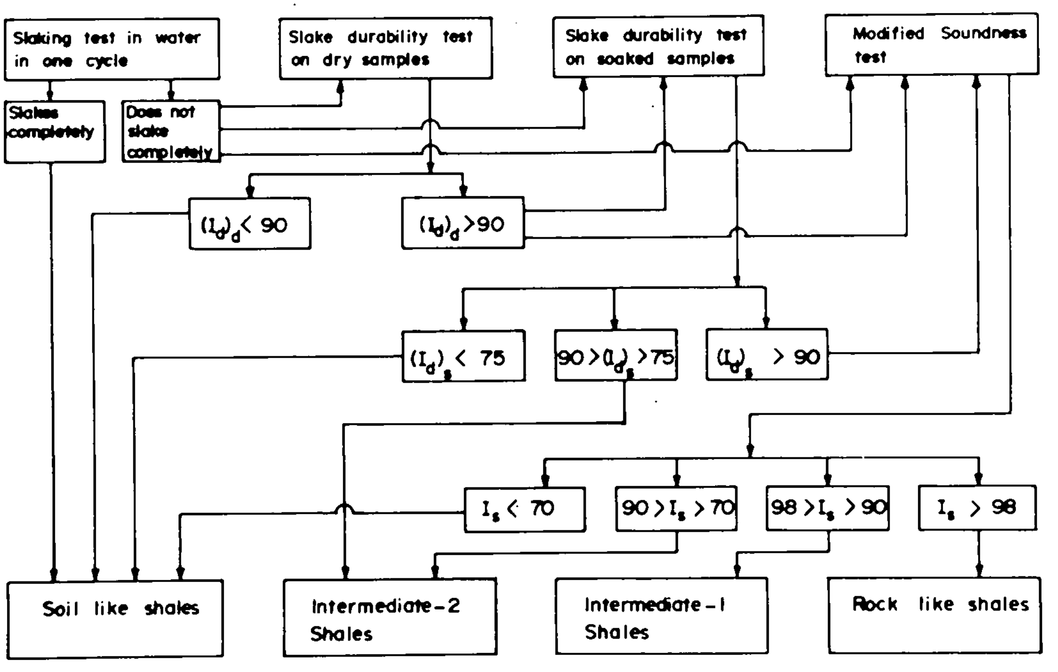
When shale is considered as a construction material in embankments, it should be viewed as a special material, i.e., something between soil and rock. It should be classified in accordance with its probable behavior in the embankment. Before actually specifying use of this type of material, the following steps are recommended.

1. Review the design and construction standards and specifications that would apply if the embankment material were (a) an average fine-grained soil or (b) an average sedimentary rock; i.e., consider the limits for the real material, which is generally intermediate.
2. Study the proposed fill material to determine whether it is homogeneous or a mixture of unlike materials, e.g., shale and limestone. There are special hazards in the latter case, and special attention is required.
3. Perform the slake durability and modified soundness tests. Classify the material in one of the four groups suggested (Fig. 3).

For the different groups of shales, the following construction practices are suggested by the authors. (These opinions were derived intuitively on the basis of observations, but without actual field tests.)

1. If the material is soil-like, it should be thoroughly broken down before use, and thinner lifts than normally specified for soil may be needed. Expansive characteristics for the shale should also be determined. (Axial swell in the CBR test is a good descriptor.) If the shale powder shows more swelling than that of ordinary clays, it should be accorded the special treatment given an expansive soil embankment, including an effective encasement of nonshale material.
2. For intermediate-1 and intermediate-2 shales, specifications should generally vary between those for soil and those for rock fills. Bigger chunks can be used. In intermediate-2 shales, it is probably necessary to have better density control and to employ an encasement.
3. A mixture of durable and nondurable material should not be used in an embankment; e.g., never mix a rock-like with intermediate-2. The two materials will degrade quite differently in service, causing potentially major problems. Only top-quality intermediate-1 or rock-like shales should be mixed with limestone or sandstone.
4. If it is not possible to separate good and bad shales, then the whole material should be treated like soil, i.e., be thoroughly broken down.

Figure 3. Proposed classification of shales for embankment construction.



## ACKNOWLEDGMENTS

Funding for this research was provided in part by the Purdue University Research Foundation and in part by the Indiana State Highway Commission, through the Joint Highway Research Project. Personnel of the Division of Materials and Tests, Indiana State Highway Commission, were quite helpful in the progress of the work.

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# gabions in highway construction

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This paper presents a short history of the use of gabions, especially in the State of Washington. Typical uses of gabions include stream control, retaining wall, and rockfall protection. Examples of each of these uses are given. Also presented is the development of Washington specifications for gabion use, and possible future modifications are discussed. Planned gabion use on I-90 at Snoqualmie Pass is discussed.

•A gabion, simply defined, is a basket of rocks! The earliest known use of gabions consisted of wicker baskets filled with stone that were piled up to form the desired structure. This evolved to the use of wire baskets (Fig. 1). There were some installations in Europe built in the last century that are still in service (Fig. 2). In this country, they have also been in use for some time but usually not under the name gabion (Figs. 3 and 4).

In Europe, gabions have been an established fact for quite some time (Figs. 5, 6, and 7). Their use in general construction is just catching on in the United States. Gabions are used primarily for two types of structures, one of which is a stream control structure such as weirs, groins, and bank protection (Figs. 8 and 9). For bank protection, gabions require much less depth and conform to local failures in the underlying bank without breach of the structure (Fig. 10). This type of structure, or a wall type of structure along a stream, should always use a fairly wide apron on the bed of the stream, as shown in Figure 10. "Mattress" types of gabions should not be placed on slopes steeper than about 2:1 because the stone filler tends to sag to the lower part of individual baskets (Fig. 11).

This brings us to the second main use of gabions, i.e., as gravity retaining walls. These are similar in application to metal bin walls but in some areas are aesthetically preferable.

The Washington Department of Highways first considered their use about 5 years ago, primarily through the efforts of Maccaferri Gabions of America. Since then gabions have been used for bank protection, retaining walls of various sizes (Fig. 12), and rockfall protection (Fig. 13). Although we have been generally pleased with their performance, we have not been pleased with their cost. This, I believe, can be attributed to two primary factors: (a) constantly revised specifications during their early development, breeding unfamiliarity among inspectors and uncertainty among contractors on what to expect; and (b) lack of construction experience, resulting in inefficient use of manpower and equipment on the part of contractors.

In regard to item a, the first available specifications (including those used by other states and the Forest Service) were those developed by Maccaferri, were quite general,

Figure 1. Typical four-unit gabion basket.

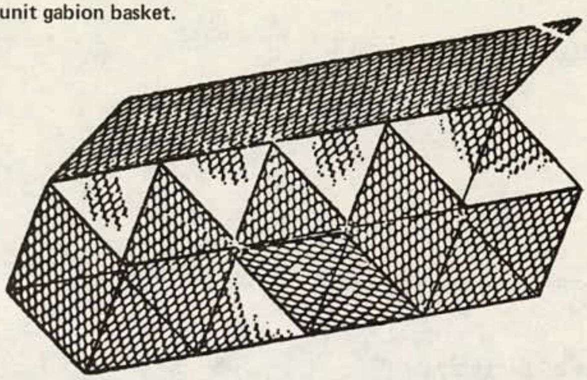


Figure 2. Installations (a) to close a breach in the bank of the River Reno built in 1893 and (b) on the River Santerno.

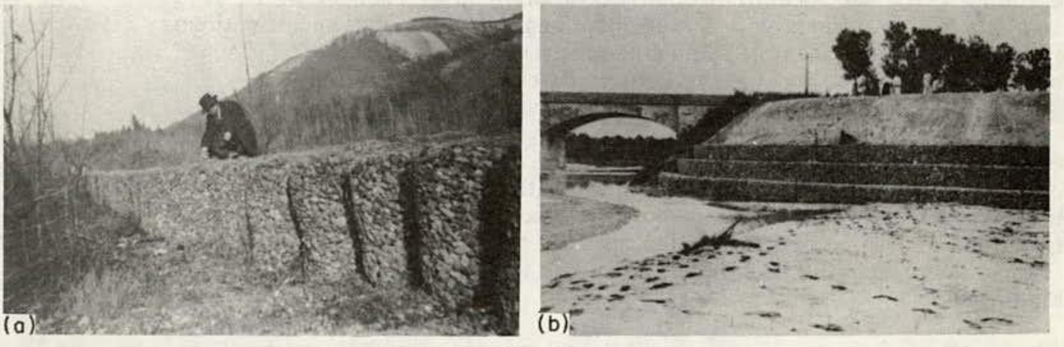


Figure 3. "Rock posts" in southeastern Washington.

Figure 4. A groin built of stone and hog wire about 1930.

**Figure 5. Extensive use in retaining a highway in the Italian Alps.**

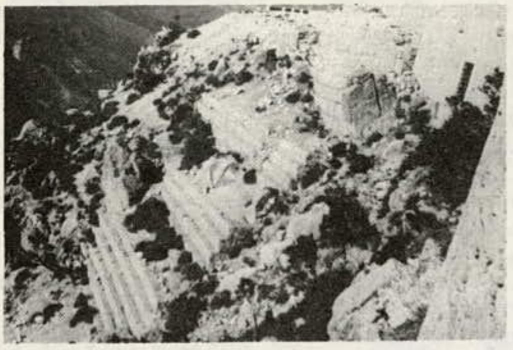


**Figure 6. Installation on the River Arno.**

**Figure 7. Restoration of a small stream in Europe.**

**Figure 8. Channel protection at culvert exit.**

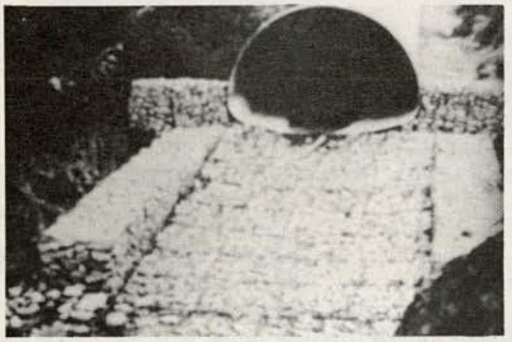
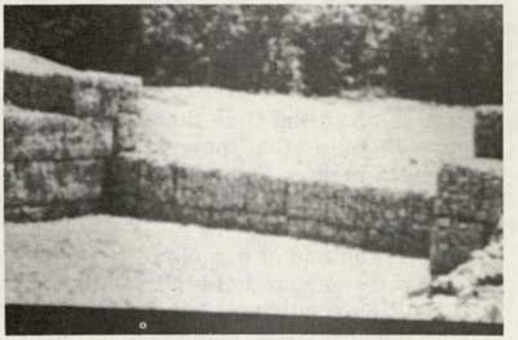


**Figure 9. Typical weir installation.**



and, understandably, were written around their product, tending to exclude competitors such as American Gabions of Browning, Montana. Examples of this are a requirement for a hexagonal triple twist mesh, wire specifications obviously written around a foreign product, and a load test that could not be met by a rectangular mesh. Maccaferri and Baekart gabions use a hexagonal mesh, whereas the American Gabion and project-fabricated gabions use a rectangular, nonclimbable fence mesh. Our interest, therefore, was to write a specification that was broad enough in certain areas to allow various methods of manufacture, yet ensure a satisfactory end product. This included reference to nonmetric measurements, a new load test, reference to standard federal wire specifications (QQ-W-461), and revision of some construction details. Also, our standard "no foreign product" specification had to be rescinded to allow the standard foreign gabion.

During this process, several gabion installations in neighboring Idaho (Fig. 14) and an extensive gabion operation in Colorado (Fig. 15) were visited. Much helpful information was gleaned on problems in Idaho and Colorado and suggested solutions. One of the big problems was cost, primarily because of the large amount of hand work in the then-accepted practice of hand-placing the stones at the face of the wall. We resolved to avoid this if we could. Another big problem was failure to specify a minimum density of the gabions or to determine the probable density of the material to be retained. This had, in at least one case, resulted in the failure of a wall because it was not heavy enough to hold the denser backfill in place. Colorado recommended a density specification, which we adopted. However, we enforced the specification by occasionally removing a completed gabion and weighing it. This is a somewhat difficult test to administer. We now allow basket filling from a container (truck) to be weighed before and after. We also recommend drastically reducing the number of these tests if satisfactory results are routine and as the inspector develops a "feel" for what a properly filled and compacted gabion looks like. Both states also strongly recommended angular stone in a retaining structure for better load transfer. This is now a part of our specification. It was, of course, recognized that this would not be necessary in a non-load-carrying structure such as bank protection. We also evolved a load test geared toward the conditions most likely to be encountered in a retaining structure. The initial specification was as follows:

A section of gabions with units having a minimum dimension of 3 feet and at least four units long shall be constructed and filled in the normal manner on a level platform having a removable center section at least 6 feet in length. The gabions shall be so situated that an end-to-end connection between units will be situated approximately over the center of the removable section. This section shall then be removed, leaving a clear span of the gabions of at least 6 feet. The gabions, including connection, shall show no sign of failure.

While the gabions are in the above condition, a single mesh wire at the bottom and near the center of the clear span will be cut, and the gabion units must still exhibit no sign of failure.

This was about the time the American Gabion people consented to try the load test. Figure 16 shows that their gabions performed beautifully. They later consented to perform a more stringent test to approximate a several-tiered wall; this test produced very good results.

A problem in one of our earlier contracts arose because we failed to specify the number of ties to be used to hold the various components together. It was decided at that time that the ties should equal the strength of the mesh; it has since been decided that this is overly conservative, inasmuch as load tests have shown that the units will hold up with most of the wires cut. The amount of mesh is, after all, more a function of the size of opening necessary to contain the filler. Our latest specification is as follows:

Gabion baskets shall be securely fastened to all adjacent gabion baskets to the satisfaction of the engineer. It is not intended that the fastenings to adjacent gabions equal the strength of the baskets themselves.

Figure 10. Typical riverbank installation showing ability to conform to changes caused by erosion.

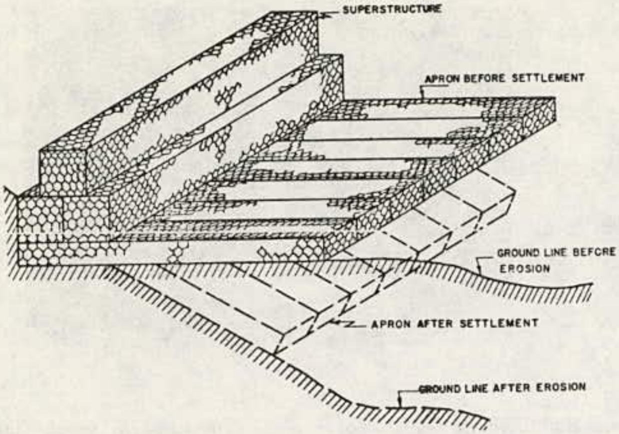



Figure 11. Results of using mattress type of gabion on too steep a slope.

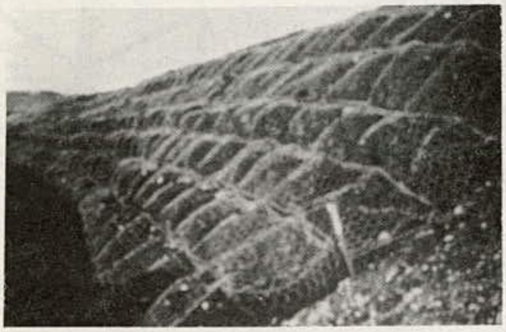


Figure 12. Wall to contain fill at bridge abutment on the Snoqualmie Pass.

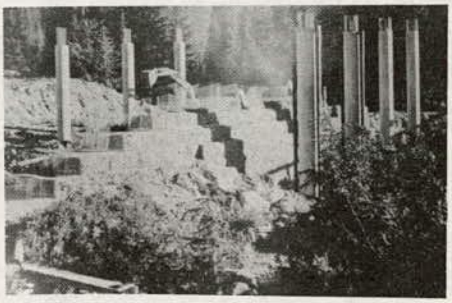


Figure 13. Protection against rockfall on Wash-12.

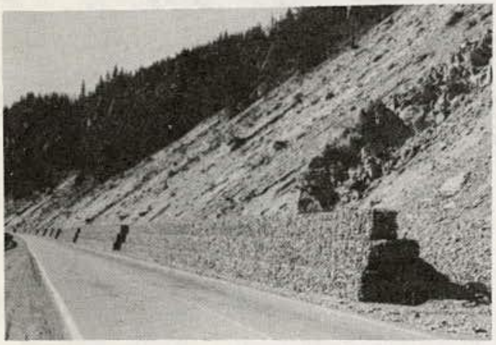


Figure 14. Gabions supporting US-95 near Whitebird, Idaho.

Figure 15. Gabion construction near Glenwood Springs, Colorado.

Figure 16. Initial test by American Gabion.

Figure 17. Artist's conception of (a) standard cantilever wall retaining a portion of I-90 near Snoqualmie Pass and (b) same location using a gabion wall.

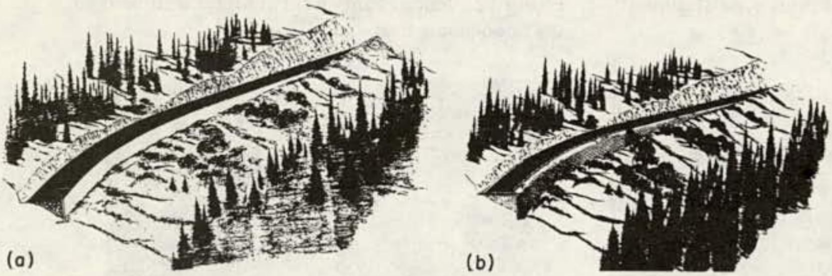



Figure 18. "Quick-Klip" joins rolls of fence wire to form gabions.

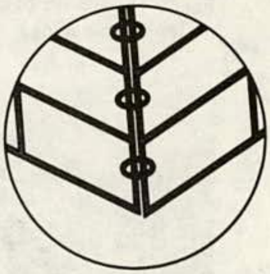
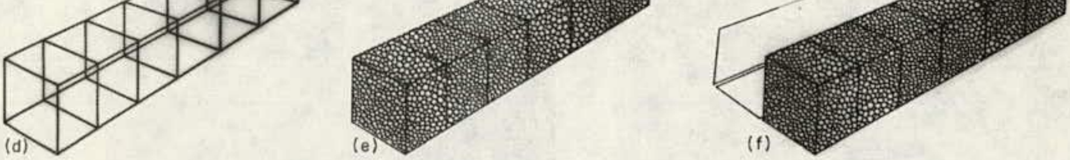


Figure 19. Fabrication of gabion: (a) roll out fence wire; (b) clip together; (c) raise sides; (d) add diaphragms; (e) fill with stone, and add lid; and (f) roll out next row.



We learned on one of our first retaining wall jobs that our design density of 115 pcf was difficult to obtain with the following usual open gradation.

| Sieve | Percentage Passing |
|---|---|
| 8-in. square | 100 |
| 6-in. square | 75 |
| 4-in. square | ≥10 |

We therefore specified additional fines to increase density.

| Sieve | Percentage Passing |
|---|---|
| 8-in. square | 100 |
| 6-in. square | 70 to 80 |
| 4-in. square | 5 to 20 |
| ¼-in. square | ≥2 |

This has not resulted in any appreciable loss of material and has helped to attain needed design density. If a conflict develops between unit weight and gradation in a retaining wall project, the unit weight must always prevail, as long as the filler remains granular and porous. However, we are now designing walls from 105 to 110 pcf, which is not too difficult to attain.

As noted earlier, we have not required hand placing of any filler material. We have had some trouble with misshapen gabions because of this, but it can be controlled with proper care, particularly if design density is not too hard to reach. We feel that the neater appearance possible with hand placing is not worth the extra cost. There certainly could be some exceptions to this, particularly in areas requiring more attention to aesthetics. Regarding aesthetics, we have found gabion structures more desirable than their concrete counterparts in areas of rugged terrain and high natural beauty, such as forest and mountain settings. In general, we will choose this type of structure over a less expensive design that does not blend so well into the natural terrain (Fig. 17).

Most recently, a project was completed for numerous rockfall barriers along Wash-12 at White Pass by using a novel gabion fabrication method developed by a mesh supplier. Essentially, it consists of fabricating the gabions in place from rolls of fence wire by using common intermediate diaphragms and specially developed stainless steel clips (Figs. 18, 19, and 20). The tedious task of installing tie wires is eliminated by increasing the number of diaphragms and adding additional diaphragms at the wall ends. There is a considerable saving on both materials and labor when this method is used, and it may help to bring gabion costs back in line with riprap and bin walls. We are still evaluating this method and intend to modify our specifications to allow this type of construction.[1]

Our most ambitious design has been for the I-90 crossing of Snoqualmie Pass in the Cascades. It has been difficult to fit a six-lane freeway into rugged terrain with minimum damage to the environment. The highway will make use of about 55,000 cubic yards of gabions, mostly retaining walls. Figure 21 shows one talus area where three lanes will be supported by a two-tiered wall (Fig. 22). This is also an area of heavy snowfall and frequent avalanches. Although other measures have been used to minimize avalanche hazard, forces due to avalanches had to be considered in the design.

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[1] Possible revised specifications were included in the original manuscript of this paper. They are available in Xerox form at cost of reproduction and handling from the Transportation Research Board. When ordering, refer to XS-50, TRB Special Report 148.

Figure 20. Completed gabions.

Figure 21. Talus area west of Snoqualmie Pass.

Figure 22. Preliminary design of walls for area shown in Figure 21.

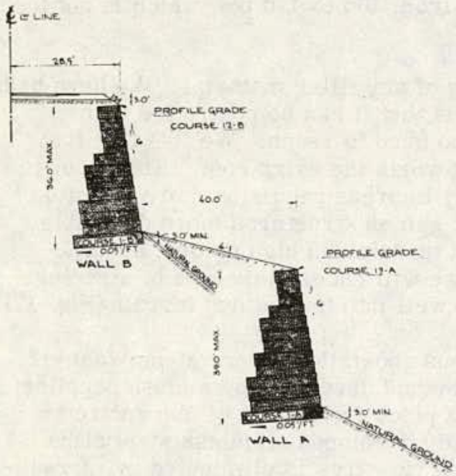



Figure 23. Avalanche chutes above I-90.

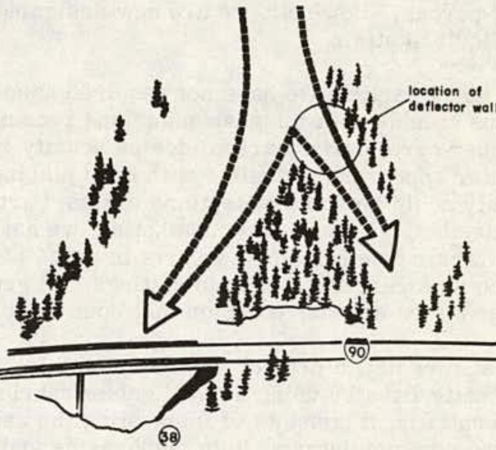
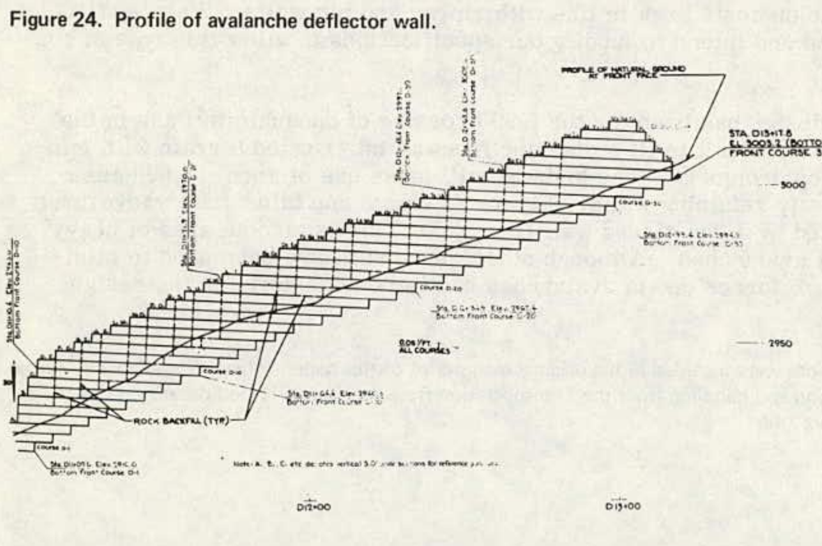



Figure 24. Profile of avalanche deflector wall.



Another unique use of gabions in this area will be as an avalanche deflector. Figure 23 shows an avalanche chute that splits above the highway, sending debris in two directions. The left leg is bridged to allow the avalanche to pass safely under the highway. A deflection wall will be constructed at the division point to contain the avalanche in this left chute (Fig. 24). This wall, of course, is designed to withstand considerable avalanche forces.

In summary, we feel that gabions do, indeed, have a place in highway construction, particularly in areas where natural-appearing structures are desirable. It appears that, with the development of newer methods and with more familiarity by the contracting fraternity, the cost of this type of construction will be competitive with other, more conventional designs.

## ACKNOWLEDGMENT

This report was written with considerable assistance from the Washington Department of Highways. Almost all the material has been gathered over the past 5 years in the author's normal duties with the department. Also, much has been drawn from the experiences and recommendations of fellow employees. Considerable advice and assistance were given by personnel of the Idaho and Colorado highway departments and from Maccaferri Gabions of America, American Gabions, and Keystone Steel and Wire. This help has been much appreciated.

The views and conclusions expressed in the report are strictly the author's and do not necessarily reflect the position of the Washington Department of Highways.

# cooling of hot-mixed asphalt laid on an insulated base

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The cooling of hot-mixed asphalt laid on an insulated base was studied to determine the feasibility of using thin insulation to permit cold-weather paving of thin mats on an existing pavement. A computer program was developed to predict the temperature distribution in the mat, insulation, and base. Bench-scale laboratory tests were conducted to verify the validity of the computer program. The computer program was then used to simulate cold-weather paving for field conditions. The results were analyzed statistically to determine the variables that significantly affect the time available for compaction. A step-wise multiple linear regression program was used to develop equations that would give the time available for compaction as a function of these significant variables and their interactions. In addition, a nomogram was constructed to predict the time available for compaction graphically. The results of this study indicated the possibility of using thin insulation for cold-weather paving.

•The basic problem with cold-weather paving is obtaining adequate compaction of the hot-mixed asphalt concrete. Rapid cooling of thin (≤2 inches) asphalt mats does not allow adequate time for compaction under marginal or submarginal environmental conditions. Because failure of asphalt concrete is usually related to insufficient compaction, it is desirable to extend the allowable time for compaction in cold-weather paving. There are a number of ways in which this goal can be achieved (1). The present study was undertaken to investigate the feasibility of using a thin layer of insulation to permit cold-weather paving.

A mathematical model for computing the temperature distribution in hot-mixed asphalt pavement after placement on an uninsulated base was described by Corlew and Dickson (2). The theoretical considerations used to develop the computer program for a pavement with insulation were essentially the same as for a pavement without insulation. However, to take into account the effect of insulation on cooling, appropriate boundary conditions were incorporated in the program. The program was designed in such a way that it could be used to predict the temperatures for cooling under laboratory or field conditions.

## EXPERIMENTATION

The test specimens of hot-mixed asphalt and asphalt base of 4-in. diameter and required height were prepared in the laboratory according to the Marshall method. To determine the density required that the specimens be weighed in both air and water. Thermal conductivities and thermal diffusivities of the specimens were determined by using a transient line source method (3).

Table 1. Properties of insulations.

| Properties | Thurane | Styrofoam FR | Styrofoam HD-300 |
|---|---|---|---|
| Thermal conductivity, Btu/hr-ft-deg F at 40 F | 0.0125 | 0.0225 | 0.0150 |
| Specific heat, Btu/lb-deg F at 40 F | 0.23 | 0.27 | 0.27 |
| Maximum operating temperature, deg F (continuous use) | 300 | 165 | 165 |
| Density, lb/ft³ | 2 | 1.8 | 3.3 |
| Range of compressive strength at 5 percent deformation, psi | 20-35 | 15-30 | 100-140 |
| Thermal diffusivity, ft²/hr | 0.0272 | 0.0463 | 0.0168 |
| Cost, dollars/ft³ | 3.36 | 2.16 | 6.48 |

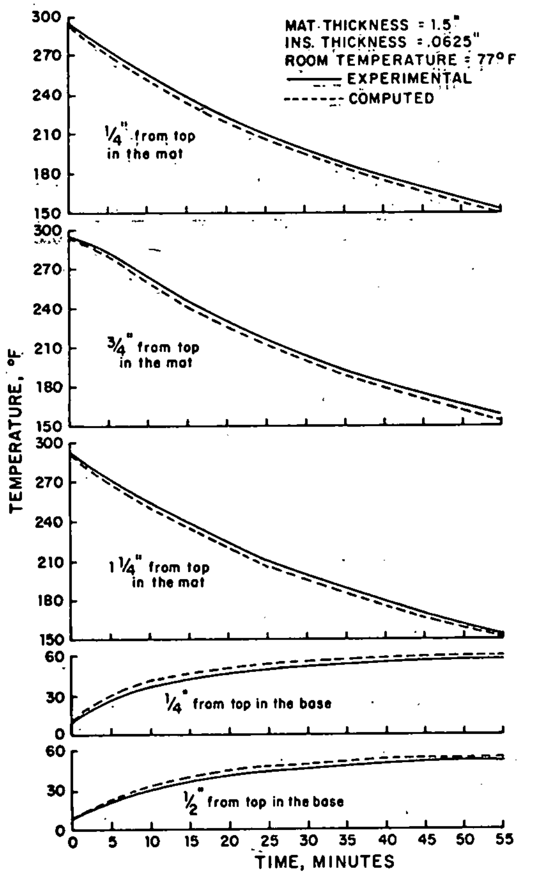



Figure 1. Comparison of experimental and computed temperatures for thurane insulation.

No. 24 gauge chromel-alumel thermocouples were located in the test specimen at a radius of 1 inch and at desired vertical distances from the upper surface. The base and mat test specimens were radially insulated with a 1.5-in. thickness of 85 percent magnesia block and glass wool insulation respectively. The top of the base specimen was covered with the desired thickness of 7-in.-diameter insulation.

For preliminary experimentations, three types of insulations, thurane, styrofoam FR, and styrofoam HD-300, were considered. The important properties of these insulations are given in Table 1.

The initial temperature of the insulation was the same as that of the base. The mat specimen, after it was laid on the insulated base, was allowed to cool from approximately 300 to 150 F under laboratory conditions. During this period, all the temperatures were recorded as a function of time by a 24-channel recorder.

## COMPARISON OF EXPERIMENTAL AND COMPUTED RESULTS

A total of 13 experimental runs were made on various combinations of mat thickness, insulation thickness, and initial base temperature values. Comparison of experimental temperatures with computed temperatures for a typical run is shown in Figure 1. The comparison of only upper base points and all the points in the mat are given. The heat wave does not significantly penetrate below a certain depth in the base; hence, a good comparison between the experimental and computed results can be expected for these points.

In general, the comparison of experimental and computed results is fairly satisfactory, which establishes the validity of the computer program. For most of the runs, it was noted that the difference between experimental and computed results was greatest at the end of the run. A small initial error magnifies during the course of time, and that may be the reason for the maximum difference at the end of the run.

## SIMULATED COLD-WEATHER PAVING

The computer program that was developed and experimentally tested was used to simulate cold-weather paving. The simulations were based on an initial mix temperature of 300 F, solar radiation of 50 Btu/hr-ft$^2$ and thermal conductivity of 0.8 Btu/hr-ft-deg F, specific heat of 0.23 Btu/lb-deg F and density of 140 lb/ft$^3$ for both mat and base.

A close examination of Table 1 reveals the difficulty of deciding which of the three insulations is best overall. From economic, heat transfer, and strength considerations, thurane falls between the two styrofoam insulations. A slight melting was noted for the two styrofoam insulations during the experimentation. From the practical standpoint, thurane would be the most important and valuable insulation in view of its higher maximum operating temperature. In light of this reasoning, we decided to use thurane for all the computer runs.

The atmospheric temperature was assumed to be the same as the initial base and insulation temperatures for all the runs. The base was assumed to be existing pavement.

For the purpose of preliminary experimentation, two values of four variables were studied: 0- and 20-mph wind velocity, 0.0625- and 0.25-in. insulation thickness, 0.5- and 1.5-in. mat thickness, and 10- and 40-F base temperature. A total of $2^4$ factorial runs were designed. The selection of the range of variables was based on practical considerations. The results of this earlier study were analyzed statistically in an attempt to determine which variables had the significant effect on the rate of cooling. Yates' technique (4) was used to analyze the effect of each of the four variables and their interactions on time for the mat to cool to an average temperature of 175 or 150 F. At the 5 percent level, the following factors were found to be significant (in decreasing order of importance): mat thickness, wind velocity, interaction of mat thickness and wind velocity, insulation thickness, interaction of wind velocity and insulation thickness, interaction of mat thickness and insulation thickness, and base temperature. At the 1 percent level, only the first five of these were significant.

Based on the analysis of the results of the first 16 runs, we decided to make 44 more computer runs. The results of a few typical runs are shown in Figures 2 and 3. From the results, it is clear that mat thickness has a pronounced effect on the time available for compaction. It should also be noted that, when a thin insulation is placed on a base, the time available for compaction increases considerably. In most cases, it was found that insulation thickness, beyond a certain value, has no significant effect on the time. Consequently, insulation thickness beyond 0.25 inch (or even 0.125 inch) would be impractical. Surprisingly, the effect of mat and insulation thickness on time is nearly linear. The base temperature does not have a significant effect on time.

In Figure 4, the temperature profiles in the mat, insulation, and base are shown plotted for a typical run. For the purpose of comparison, the temperature profiles for the similar conditions but without insulation are shown in Figure 5. Cooling is significantly retarded when insulation is used. From Figure 5, it is evident that more heat from the mat is lost to the base than to the surrounding air. Consequently, placing a thin insulation results in a marked change in the temperature profile, particularly near the bottom of the mat. This is advantageous because the compactive effort on the mat is least near the bottom (5).

In Figure 6 temperatures of different points in the mat and base with and without insulation are shown plotted as a function of time. Again the rapid cooling of the mat and heating of the base are evident. This rapid cooling is effectively retarded by the insulation, which ultimately increases the time available for compaction.

Of the 60 computer runs made, the data of 45 runs (excluding those 15 runs without insulation) were used to find a relationship between time to cool to 175 or 150 F and the four variables and their significant interactions. A computer program for step-wise multiple linear regression was used to develop this relationship. The final results are

$$\begin{aligned}\text{Time to cool to 150 F} = {} & 33.56(A) + 0.41(B) + 21.58(C) + 0.15(D) - 1.11(AB) \\ & - 2.62(BC) + 43.45(AC) - 10.01\end{aligned} \quad (1)$$

$$\begin{aligned}\text{Time to cool to 175 F} = {} & 24.76(A) + 0.31(B) + 15.44(C) + 0.0955(D) - 0.81(AB) \\ & - 1.98(BC) + 33.76(AC) - 7.12\end{aligned} \quad (2)$$

where time is in minutes and

A = mix thickness, inches;
B = wind velocity, mph;
C = insulation thickness, inches; and
D = base temperature, deg F.

Finally, in an attempt to simplify the problem, a nomogram was constructed to give time to cool to 150 or 175 F by a graphical procedure (Fig. 7).

First point G on the scale of mat thickness is joined by a straight line, GH, to a point on the scale of the wind velocity. Point I, which is the intersection of line GH with scale A, is joined by straight line IJ to point J on the scale of base temperature. The point K, which is the intersection of line IJ with scale B, is joined by straight line KL to point L on the scale of insulation thickness. Point M, which is the intersection of line KL with the time scale, gives the time for the mat to cool to 150 or 175 F. Thus

Figure 2. Time to cool to 175 F.

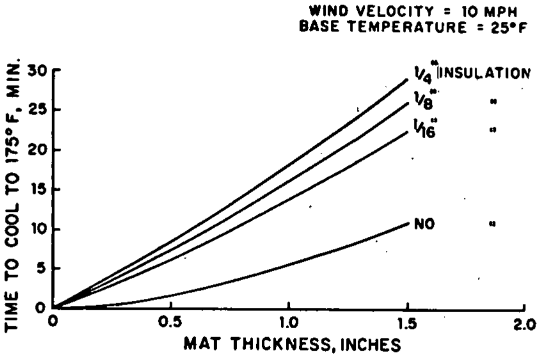



Figure 3. Time to cool to 150 F.

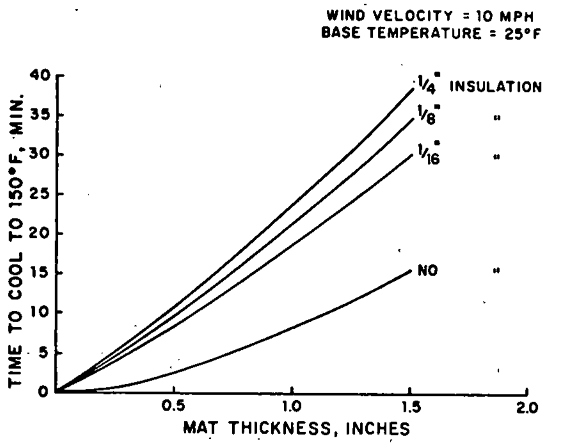



Figure 4. Temperature profile in mat, insulation, and base.

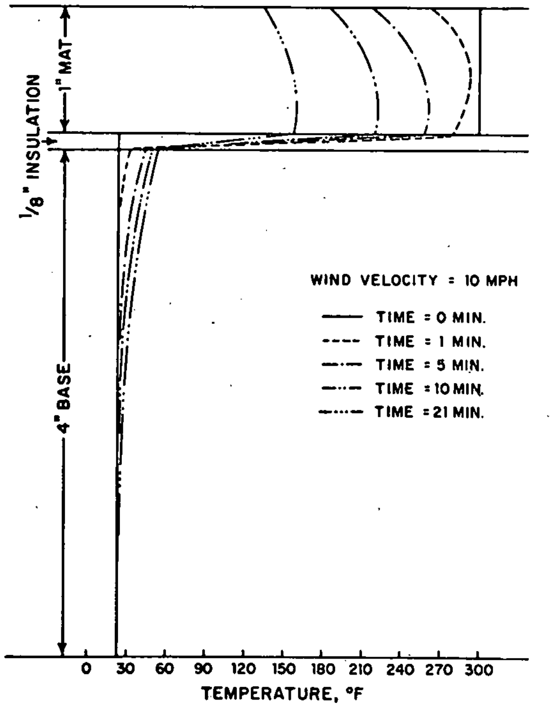
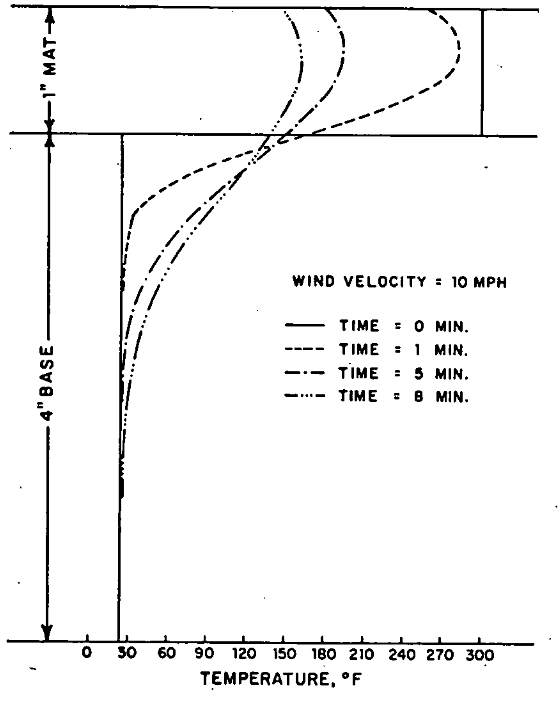



Figure 5. Temperature profile in mat and base without insulation.



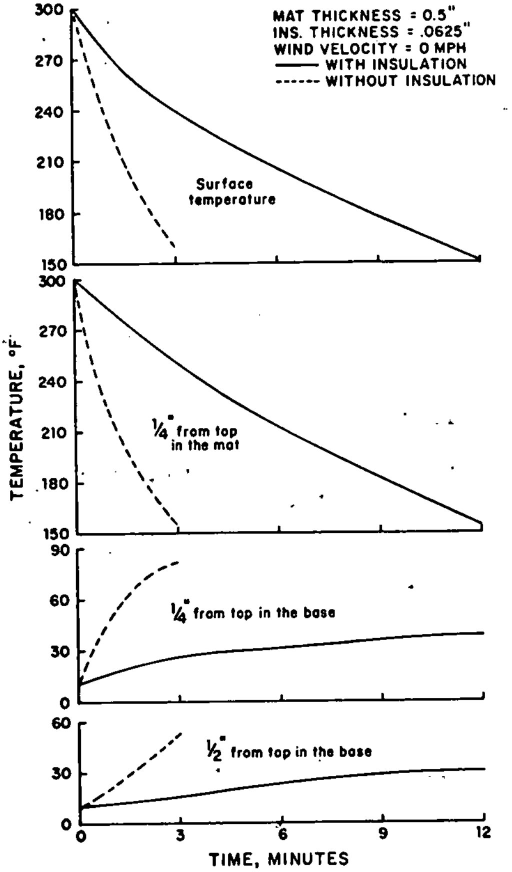



Figure 6. Temperature profiles with and without thurane insulation.

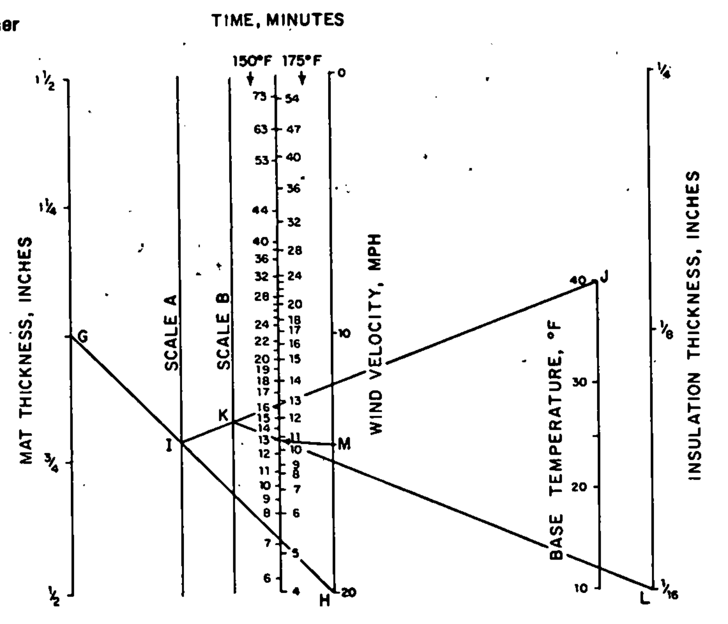



Figure 7. Nomograph for cold-weather paving.

for a 1-in.-thick mat, 20-mph wind velocity, 40-F base temperature, and 0.0625-in. insulation thickness, the time to cool to 175 and 150 F is 10.7 and 13 minutes respectively.

The results obtained by equation and nomogram are suggested for rough preliminary guidance. A maximum error of about 20 percent was noted in some cases.

## CONCLUSION

The results of this study indicate the feasibility of using thin insulations for cold-weather paving of thin mats. The insulation effectively retards the rapid cooling of thin mats, which consequently results in a considerable increase in the time available for compaction.

There is excellent agreement between the experimental and computed results, which confirms the validity of the computer program. The results of the preliminary statistical analysis are within expectation. For most cases, using insulation results in a 300 to 400 percent increase in the time available for compaction. The equations and nomogram are accurate for all practical purposes, but should be used with caution for rigorous design. To arrive at a definitive indication of the future use of insulation for cold-weather paving of thin mats requires that study be directed toward an in-depth economic investigation of the problem.

## ACKNOWLEDGMENT

The authors wish to express their sincere thanks for the support of the National Science Foundation, whose grant made this research possible.

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# part 4

# systems building for bridges

# prefabricated sandwich panels for bridge decks

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Virginia Highway Research Council

Based on the generally recognized superior strength-weight characteristics of sandwich panels, a study program was carried out to test a new type of compositely acting steel-concrete sandwich panel for use in bridge decks. Basically, the sandwich panel consists of two thin-faced plates of steel joined by a series of round shear-spacer studs welded to their inner surfaces. The core between the plates is made of lightweight concrete using expanding cement to induce a small prestress into the system. Laboratory tests were made on 10 small-scale panels, some loaded with a concentrated load and others with a uniformly distributed load. Mathematical theories developed for this type of panel show a generally satisfactory correlation between tests and theory. A number of solutions that take into account practical fabrication and erection problems are offered to illustrate how such panels can be bolted or welded, either longitudinally or transversely, across standard steel bridge girders. A comparative investigation indicated that these panels are substantially stronger and stiffer than normal reinforced concrete slabs that use the same quantity of concrete and steel.

•Since 1968, the Virginia Highway Research Council, with the cosponsorship of the Federal Highway Administration, has been engaged in the study and development of a new type of sandwich panel as might be used for prefabricated bridge decks. From the experience gained with sandwich construction in airframe and building industries, the use of sandwich construction to improve the strength-weight characteristics appeared promising. Inasmuch as bridge decks have requirements distinct from those of airframes and buildings, sandwich solutions appropriate to these requirements had to be developed. In particular, bridge decks must sustain unusually large uniform and concentrated loads, be durable under exposed conditions, and be relatively economical to construct. For this study, an additional requirement was imposed: The deck should lend itself to prefabrication to minimize the need for field construction.

After consideration of all these factors, a sandwich panel configuration (Fig. 1) was conceived. The details of fabrication and erection will be discussed later in this paper; at this time only a brief description of the panel will be presented. The top and bottom faces are thin steel plates connected intermittently by steel studs welded between them. (The protrusion of the studs on one side was done for ease of fabrication for the experimental test panels only. As will be explained later, this need not be done for prototype panels.) Side plates are welded around all edges of the panel. In the hollow between the face plates, lightweight concrete is placed to form a rigid core. This concrete can be pumped into place through small holes in the panel at either the fabricating plant or the construction site. Additionally, small bleeder holes should be provided in the top plate to ensure that during pumping all voids will be filled. (After filling, all holes in the plates would be sealed with welded steel cover plates.)

Although not absolutely essential, it is desirable to use an expanding cement in the concrete mix to induce a compressive prestress into the concrete (to reduce cracking) and a tension prestress into the steel face plates (to reduce plate buckling).

The purpose of the studs is to provide composite action between the concrete core and the steel face plates.

## STRUCTURAL INVESTIGATIONS

Before the panels were tested, it was necessary to make a study of the expanding lightweight core concrete, inasmuch as no relevant information was available on the subject. In all, 170 test specimens were cast by using 10 concrete mix designs and seven percentages of steel reinforcing. The reinforcing steel simulated the elastic restraining effect achieved by the face plates in the actual panel as the concrete tends to expand. The aggregates used were expanded shale and sand. Basically, the test specimens were concrete prisms 3 by 3 by 11 in. (76 by 76 by 280 mm), with reinforcing steel positioned along the 11-in. length. Expansion readings were taken periodically for 8 weeks after casting. All curing was under autogenous conditions to simulate the moisture-sealed conditions of the prototype panels (1, 2).

The empirical equation found to express the percentage of expansion, r, is

$$r = K(p + q)^{-m} \qquad (1)$$

where

$K = 0.012\ (w\text{-}c)^{-2.783}$,
$m = 0.488\ (w\text{-}c)^{-0.609}$,
$q = 0.003\ (w\text{-}c)^{-3.109}$,
$p$ = percentage of steel reinforcing based on gross area, and
$w\text{-}c$ = water-cement ratio.

Figure 2 shows a typical curve relating expansion to percentage of steel restraint.

After the study of the core concrete material, 10 test panels were fabricated and tested. Their dimensions are given in Table 1, and their various stud configurations are shown in Figure 3. All panels were 25 in. (635 mm) square and were simply supported on all four sides by bearings placed 23.5 in. (597 mm) apart. The first seven panels were loaded by a concentrated load at the center by using a 2-in. (50.8-mm) square bar as shown in Figure 4. Figure 4 also shows the instrumentation used—dial gauges for deflection and electrical strain gauge rosettes for strain.

Panels loaded with the concentrated force characteristically failed by shear punching as shown in Figure 5. The steel face plates bent plastically but did not rupture. Removal of the face plates revealed that the concrete core did rupture by diagonal tension in the standard failure cone pattern. The studs near the failure zone underwent some distortion but did not rupture.

Given that the object of the tests was to determine the nature of the service load or elastic behavior, instrumental data were recorded only for such linear conditions. Plots of a typical load-strain relationship are shown in Figure 6. Strain readings are given for a point on the bottom of the panel under the load.

Figure 7 shows the test setup for loading the panel with a uniformly distributed force. Note that a steel box filled with dry sand was placed above the test panel in the hydraulic loading machine. The head of the loading machine bore on a stiff plate on top of the sand, which distributed load on the test panel. Three panels were tested in this manner.

Figure 1. Cutaway of sandwich panel without concrete core.

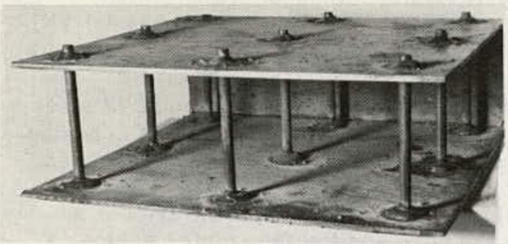


Figure 2. Typical expansion curve for expanding lightweight concrete.

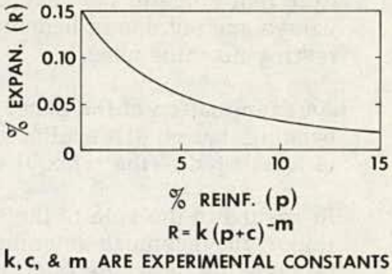



Table 1. Dimensions of test panels.

| Panel No. | Thickness (in.) Face | Core | Total | No. of Studs[a] |
|---|---|---|---|---|
| $P_1$ | 0.140 | 2.51 | 2.79 | 36 |
| $P_2$ | 0.076 | 1.348 | 1.50 | 36 |
| $P_3$ | 0.75 | 1.038 | 1.19 | 36 |
| $P_4$ | 0.076 | 1.118 | 1.27 | 1 |
| $P_5$ | 0.076 | 1.120 | 1.27 | 10 |
| $P_6$ | 0.074 | 1.032 | 1.18 | 4 |
| $P_7$ | 0.076 | 1.078 | 1.23 | 33 |
| $P_8$ | 0.075 | 1 | 1.15 | 36 |
| $P_9$ | 0.075 | 1 | 1.15 | 16 |
| $P_{10}$ | 0.075 | 1 | 1.15 | 0 |

[a]Diameter of the studs was 0.25 in. except for panel $P_3$ with a diameter of 0.125.

Figure 3. Pattern of studs in test panels

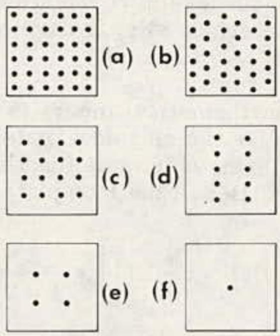



Figure 4. Test panel under concentrated load showing instrumentation.

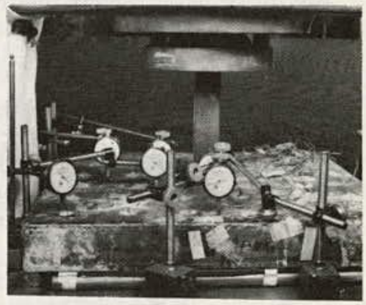


Figure 5. Failed test panel.

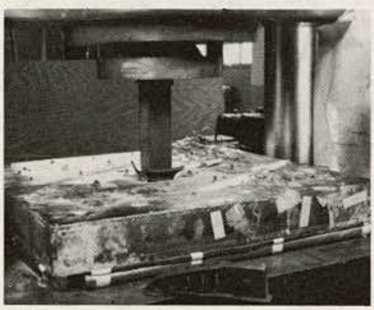


Figure 6. Load-strain readings for centrally loaded panels.

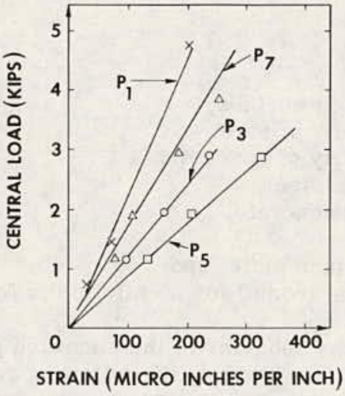



Figure 7. Test panel under uniform load.

A plot of the load-strain readings at the bottom center of the panel is shown in Figure 8. A related plot of load-deflection readings at the center of the panel is shown in Figure 9. Note that some of the readings are beyond the elastic limit, although ultimate load values are not shown because they went beyond the capacity of the 300,000-lbf (1300-kN) testing machine used.

An examination of the panel beyond the elastic limit showed some small amount of steel bending, but no discernible cracking or rupturing in the concrete. This lack of cracking is attributed to the triaxial restraint offered the concrete core by the steel on all sides.

In regard to the role of the studs at low loads, their number and placement are not too important inasmuch as composite action between the core and the face plates seems to be taken by interface friction. At loads beyond the elastic limit, interface friction breaks and the studs take over as the shear transfer mechanism. At higher loads, an increase in the number of studs causes an increase in the strength and stiffness characteristics.

Following the laboratory testing program, the sandwich panel was analyzed mathematically by using linear theory. Successful correlation of theory and test results depends on the assumption that some interface slip, S, develops between the core and the face plates (Fig. 10).

A complete derivation of the mathematical theory is given elsewhere (1, 2). Of interest is the fact that Eq. 2 is not unlike the classical plate equation used for homogeneous plates, except for a modifier term, which is a function of the slip and the respective dimensions and material properties of the face plate and core.

$$\left\{\frac{E_s}{1 - V_s^2}\left[t/2(T + t)(T + t - kT)\right] + \frac{E_c}{1 - V_c^2}(T^3/12)\right\}\nabla^4 W = P \tag{2}$$

in which

$$\nabla^4 W = \frac{\partial^4 W}{\partial x^4} + 2\frac{\partial^4 W}{\partial^2 x \partial^2 y} + \frac{\partial^4 W}{\partial y^4}$$

and where

- $W$ = deflection,
- $x, y$ = plate coordinates,
- $P$ = normal load per unit surface,
- $E_s$ = modulus of elasticity of steel,
- $E_c$ = modulus of elasticity of concrete,
- $V_s$ = Poisson's ration of steel,
- $V_c$ = Poisson's ratio of concrete,
- $T$ = thickness of core,
- $t$ = thickness of each face plate, and
- $k$ = slip factor (varying from 0 for no slip to 0.5 for full slip).

By using Eq. 2 to compare the behavior of the sandwich panels described with that of standard reinforced concrete deck slabs, the following results were found. Given the same working stress in the steel and concrete and the same quantity of steel and concrete, the sandwich panel is found to be 41 percent stronger than the concrete slab, yet it deflects 23 percent less, even at the greater load. The conclusion is that sandwich panels are substantially stronger and stiffer than comparable reinforced concrete panels, which bears out the original assumption that sandwich construction offers superior strength to weight characteristics, even for bridge decks. Inasmuch as one of the major weight factors in a normal bridge is the dead load of the concrete floor, any method of materially reducing this dead load is highly desirable.

# BRIDGE APPLICATIONS

After demonstrating that sandwich panels are structurally valid for bridge decks, we propose practical means of fabricating and erecting such panels. Several such methods are suggested. In all cases, the following design conditions are assumed:

1. The panel is attached to steel stringers or girders;
2. The top surface of the panel is skid-proofed by either an epoxy mortar overlay or a thin layer of bonded asphalt;
3. The exterior steel used is either weathering steel or normal structural steel painted for corrosion resistance;
4. The panels are prefabricated in large sections for ease and rapidity of field erection;
5. Rails, poles, curbs, and so on are made so that they can be welded or bolted to the panels; and
6. The panels are adequately braced or supported in shipment to avoid damage.

## Method I: Full-Length Longitudinal Panels Bolted to the Girders

Figure 11 shows how a longitudinal panel might be fabricated in a plant by using modern welding techniques. Note that the whole panel can be assembled in one position with all welding done downhand. Actually the panel is fabricated in a position opposite to the one it will have on the bridge. It is estimated that the steel plates will be on the order of $\frac{1}{4}$ in. (6.4 mm) thick, the studs $\frac{3}{8}$ in. (9.5 mm) in diameter and about 12 in. (305 mm) apart, and the core 4 to 6 in. (102 to 152 mm) thick. The entire panel is about 6 to 8 ft (1.8 to 2.4 m) wide and the full length of the span, or up to about 100 ft (30.5 m) maximum.

The sequence of fabrication is (a) automatic welding of the studs to the bottom plate (as with Nelson studs), (b) welding of the bolts to the bottom plate, (c) fillet welding of the side plates, (d) plug welding of the top plates to the studs in prepunched holes (the wide head on the studs allows for small fabrication errors), (e) pumping of the concrete through holes in the top plate, and (f) welding of the small cover plates over the pumping and bleeder holes.

Figure 12 shows how these panels are attached to the bridge girders by simple bolting. Such bolting not only secures the panel to the girder but also provides composite action between the girder and the deck panel. Should transverse continuity of the deck panels be desired (although not really necessary) the top face plates could be field-welded at their junctures as shown.

The voids between the panels can be hot sealed or pressure sealed with any number of materials such as bitumen, grout, urethane, or neoprene gaskets.

## Method II: Full-Length Longitudinal Panels Welded to the Girders

In Figure 13, the fabrication shown is similar to that in method I except that the top plate is bent to form the sides as well, which avoids some welding. If a press or brake is not available to bend a very long plate, the edge strip can be cut at suitable intervals for accommodation by a short press. The cut can then be welded back together after bending.

Figure 14 shows how these panels can be attached in the field by slot welding from above. Semiautomatic welders are available for continuous straight run welding of this type, which requires little manual welding. By attaching the panels to the girders in this fashion, partial composite action can be effected between the girder and the panels. If composite action is not required, simple tack or intermittent welding of the panel to the flange can be done, with a savings in field welding.

Figure 8. Load-strain readings for uniformly loaded panels.

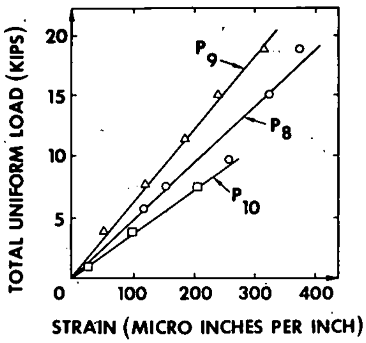



Figure 9. Load-deflection readings for uniformly loaded panels.

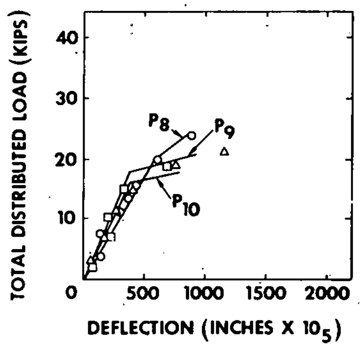



Figure 10. Slip at interfaces.

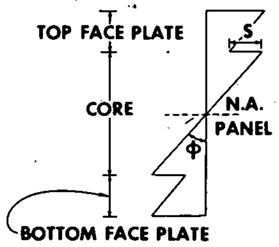



Figure 11. Method I longitudinal panel.

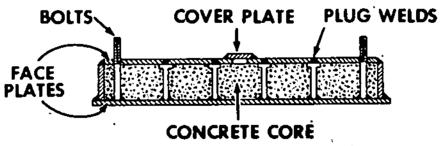



Figure 12. Attachment of panels to bridge (method I).

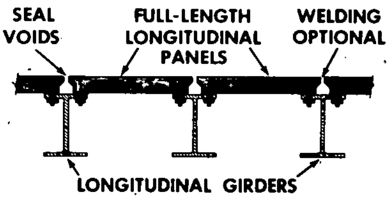



Figure 13. Method II longitudinal panel.

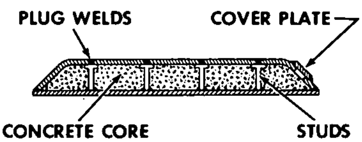



Figure 14. Attachment of panels to bridge (method II).

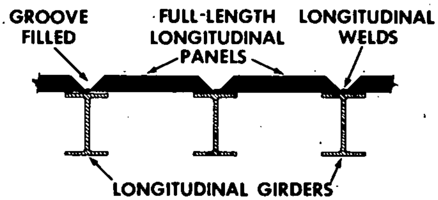



Figure 15. Method III transverse panel.

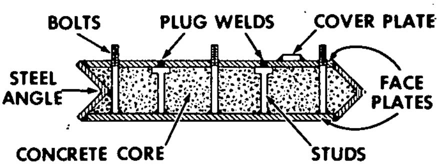
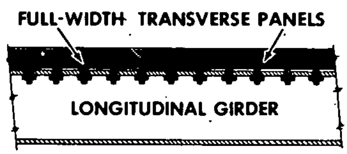



Figure 16. Attachment of panels to bridge (method III).



If the panels are topped with a wearing surface of asphalt, the grooves between panels can be filled with the same material at the same time to level out the roadway surface.

Method III: Full-Width Transverse Panels Bolted to the Girders

The basic method of fabrication for transverse panels shown in Figure 15 is similar to that described in method I except that the long side pieces are rolled angle sections rather than flat plates. The angles provide a shear transfer joint when the panels are erected on the girders. (Note that the sharp leading edge of the angle is to be ground off so that it can fit snugly with its mating angle as shown in Fig. 16.) Field welding of this joint is optional.

Pumping and bleeder holes are located in regions away from the bolts so that they will not interfere with attachment to the girders.

These panels can be 4 to 8 ft (1.2 to 2.4 m) wide and up to 100 ft (30.5 m) long, or the full width of the bridge.

Figure 16 shows how these transverse panels are mated and attached to the girders by bolts. For ease of erection, holes in the girder flanges should be enlarged or slotted. It is recommended that the transverse cross slope (for drainage) be along an arc of a circle rather than $\wedge$ so that the natural flexure of the panel can adjust to the curvature more easily. However, even with an arc, tapered washers would probably be needed to secure a good connection between the panel and the girder flange. The panels, if properly gripped, provide for composite action between the girder and the deck. Transverse continuity of the deck panels would, of course, be automatically established.

## ACKNOWLEDGMENTS

Most of the laboratory and mathematical analyses for this project were done by Raghupati S. Sinha under the general direction of the author. Without his valuable assistance, the validation of the structural behavior of the sandwich panels would have been lacking.

Funds for this research were generously provided by the Virginia Highway Research Council and the Federal Highway Administration. However, the opinions, findings, and conclusions expressed in this report are those of the author and not necessarily those of the sponsoring agencies.

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# use of prestressed, precast concrete panels in highway bridge construction

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A beam and slab bridge that makes use of precast, prestressed panels was investigated. For this type of bridge, both the panels and the beams are precast and prestressed in the casting yard. In the bridge structure, the panels span the transverse distance between beams and serve as forms for the cast-in-place portion of the deck. They remain in place to become an integral part of the continuous structural slab. Composite action is obtained when the deck elements and the beams are bonded together by the cast-in-place concrete. Tests were made to determine the ability of this type of bridge to distribute wheel loads in a satisfactory manner and to behave as a composite unit. A full-scale, simple span, prestressed panel concrete bridge was constructed and structurally tested in the laboratory. The bridge was subjected to cyclic applications of design loads and finally to static failure loads. It performed satisfactorily under all load conditions. Several bridges of this type have been in service in Texas for 10 years and have performed well.

•The relatively new type of concrete bridge slab discussed in this paper is constructed of prestressed, precast panels and conventionally reinforced, cast-in-place concrete. This type of construction, shown in Figure 1, reduces construction time and cost.

The precast panels serve as forms for the cast-in-place portion of the deck and remain in place as an integral part of the deck. This eliminates installation and removal of forms and falsework on the underside. The cast-in-place deck is mechanically connected to the beams by the stirrups that extend into the deck as in conventional construction. The cast-in-place concrete is placed over the surface of the panels and into the space above the beams; thereby, all elements are bonded together to act as a composite unit. The cast-in-place concrete serves as the riding surface of the deck.

Two major deviations from conventional beam and slab concrete bridge construction exist in a prestressed panel bridge: bonding of a new concrete to old concrete with sufficient strength to develop an adequate structural connection at the interface and inclusion of transverse joints made by butting adjacent prestressed panels together. The latter created some question on the capacity of the slab to accomplish longitudinal distribution of wheel loads and to resist internal stresses. This transverse joint extends from top to bottom of the prestressed panels but does not extend into the cast-in-place concrete. Present AASHO bridge specifications do not deal directly or by implication with these two structural details.

Figure 1. Prestressed, precast concrete panel bridge.

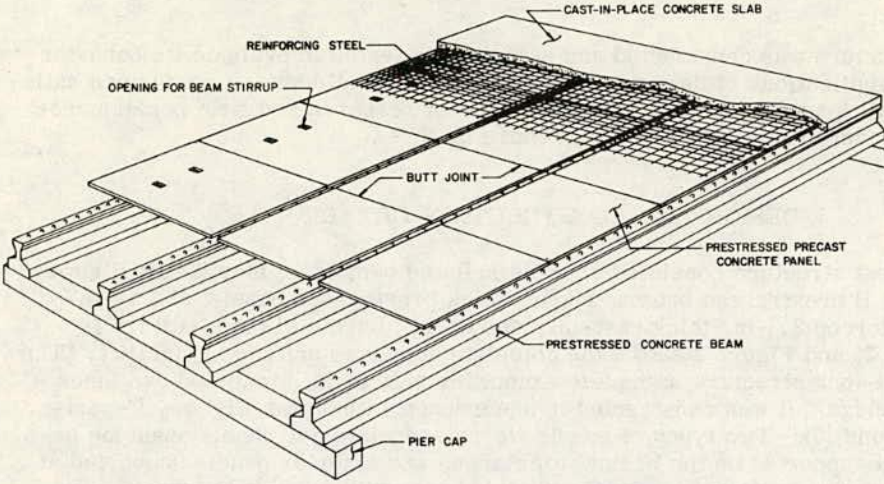



Figure 2. Layout of prestressed panel bridge.

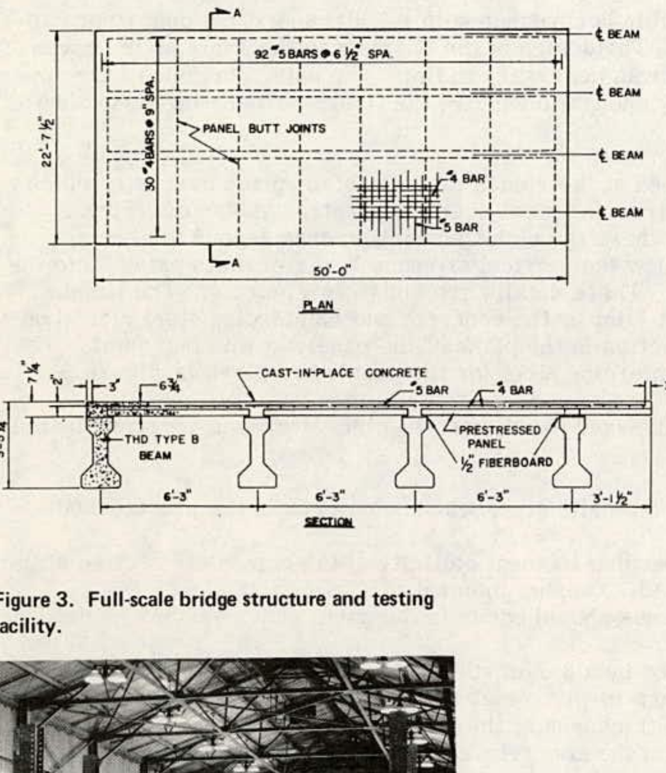
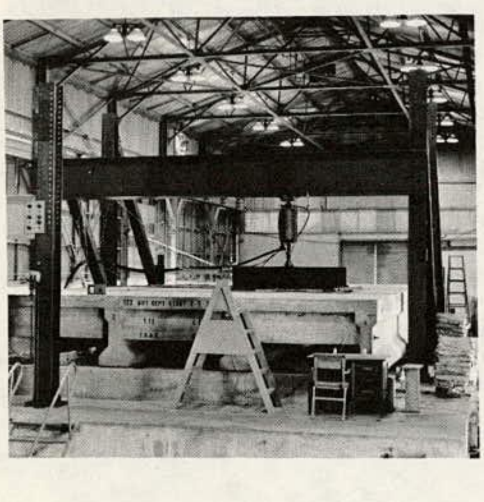



Figure 3. Full-scale bridge structure and testing facility.



A full-scale structure was constructed and structurally tested to evaluate its behavior under repeated applications of design loads and static failure loads. It performed satisfactorily under all load conditions. Further details of research and field performance of prestressed panel bridges are given elsewhere (2, 3, 4).

## DESIGN AND CONSTRUCTION DETAILS

The full-scale test structure consisted of two simulated bent caps, four Texas Highway Department type B prestressed beams, 3¼-in.-thick prestressed panels, and a conventionally reinforced 3½-in.-thick cast-in-place deck. Layout of the structure is shown in Figure 2, and Figure 3 shows the complete structure and testing facility. The 23-ft-wide, 50-ft-long structure, complete except for side rails, simulated two lanes of a four-lane bridge. It was constructed in accordance with Texas Highway Department specifications (5). Two types of panels were used: interior panels spanning between beams and supported on the beams' top flanges and exterior panels supported at one end on a beam and continuous over the outside beam with a 2½-ft overhang. The panels became an integral part of the continuous deck when the top portion of the deck was cast.

The entire structure was designed in accordance with AASHO specifications, where applicable, for an HS20-44 loading. The design of the beams was the same as in conventional construction, except that it was necessary to finish the outer portions of the upper surface of the top flange smoothly enough to receive the fiberboard seating strip shown in Figure 2.

The prestressed panels were joined at their ends by the cast-in-place concrete, which engaged a 3-in. extension of prestressing steel over the prestressed beams (Fig. 4). At the outside edge of the bridge where the slab was cantilevered beyond the beam, holes were cast in the panel to allow the vertical stirrups in the beam to extend into the cast-in-place portion of the slab. These details are shown in Figure 4. The panels were joined at the transverse butt joint by the concrete and reinforcing steel placed on top of them. There was no connection in the plane of the panels at this butt joint. Dimensions and details of the reinforcing steel for the panels are given in Figure 5.

The design of the composite prestressed panel, cast-in-place deck was governed by the following requirements:

1. Under construction loads, no tensile stresses should occur in the prestressed panel,
2. The minimum transverse bending moment capacity of the composite section should be greater than or equal to the AASHO design moment (3.77 kip-ft/ft), and
3. No transverse tensile stresses should occur in the panel under service loads.

These requirements were satisfied by a 3¼-in.-thick panel with prestressing as shown in Figure 5 and a 3½-in.-thick cast-in-place slab with the transverse reinforcing shown in Figure 2. Although the design thickness of the composite slab was 6¾ in., the actual slab measured 7 in. Properties of the concrete are given in Table 1.

It was assumed in the design that all elements of the structure would act as a composite unit. This assumption required that all elements of the structure be bonded together in a suitable manner to transfer all stresses across the interface between the cast-in-place deck and the prestressed panels and at the slab-beam interfaces. At the interfaces, the same proven methods used in conventional beam and slab bridges were employed. Three methods, used as test variables, were employed to bond the cast-in-place concrete to the top surface of the prestressed panels (Fig. 6). Z-bars (Fig. 7) were used to provide both shear and tensile bond over a selected portion of the deck. In another area, portland cement grout was thoroughly brushed onto the rough surfaces

Figure 4. Detail of slabs resting on (a) interior beam and (b) exterior beam.

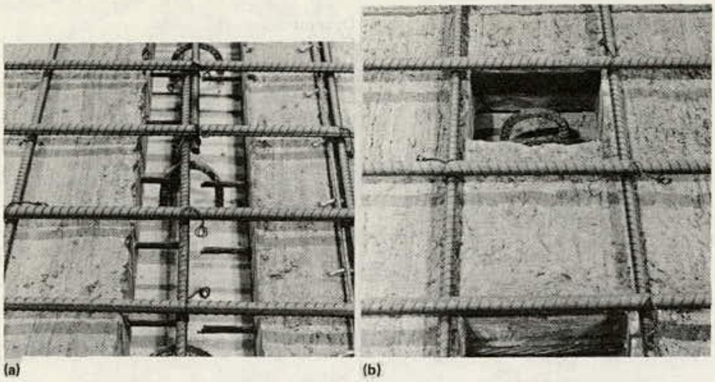



Figure 5. Details of (a) interior panels and (b) exterior panels of full-scale bridge.

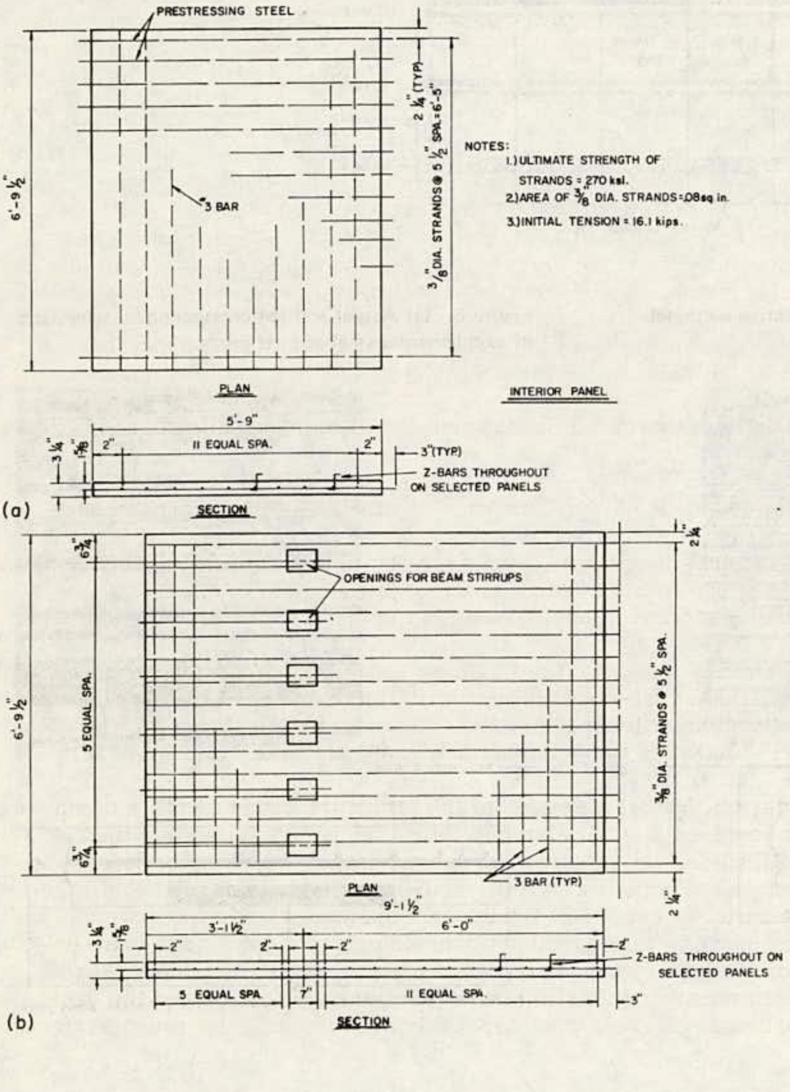


**Table 1. Concrete properties.**

| Item | Date Cast | Release Strength (psi) | Compressive Strength (psi) | Tensile Strength (psi) | Dynamic Modulus of Elasticity (psi × $10^6$) |
|---|---|---|---|---|---|
| Prestressed beams | 10-29-70 | 4,810 | 7,590 at 28 days | | 6.19 |
| | 10-30-70 | 4,880 | 7,130 at 28 days | | 6.19 |
| Prestressed panels | 12-10-70 | —[a] | 8,550 at 316 days | 640 at 480 days | 5.65 |
| Cast-in-place deck | 2-25-71 | | 5,970 at 240 days | 490 at 400 days | 5.23 |

[a]Data not available.

**Figure 6. Location and identification of variable sections.**

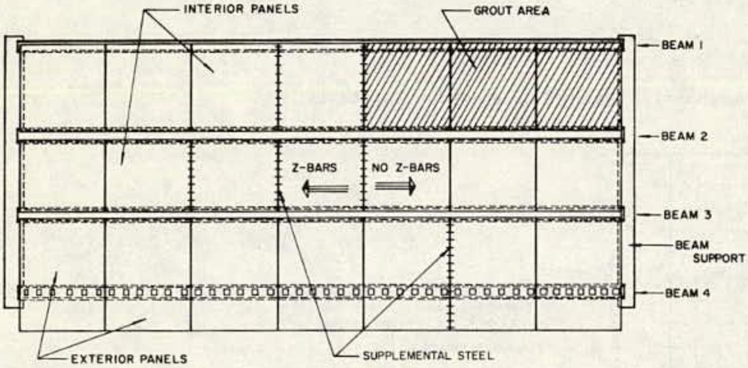



**Figure 7. (a) Actual and (b) cross-sectional schematic of Z-bars.**

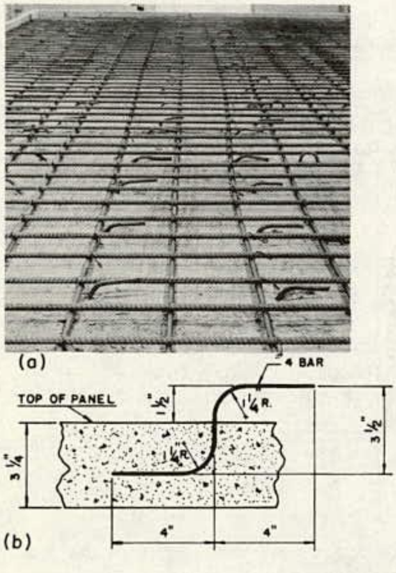
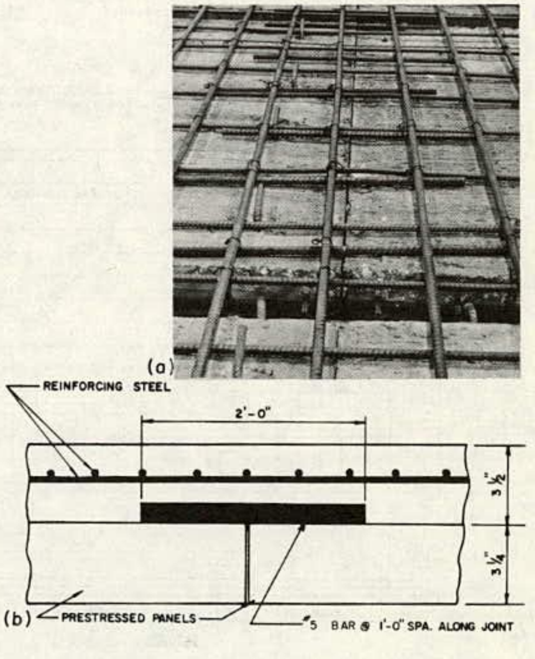



**Figure 8. (a) Actual and (b) cross-sectional schematic of supplemental steel at butt joints.**



of the panels to serve as a bonding agent. The cast-in-place deck was placed over the grout. There was no special treatment over the remainder of the deck. The locations of these areas on the structure are shown in Figure 8. The surface of the panels was thoroughly cleaned with water from a hose and nozzle and then damp-dried shortly before placement of the cast-in-place concrete. The grout was applied on the selected panels immediately before concrete placement. The progress of the grout brushing operation was regulated so that the grout did not dry before placement of the concrete.

At selected transverse butt joints, supplemental bars were placed on the surface of the panels and extended across the butt joint (Fig. 8). They were intended to aid in transferring a wheel load across the panel joint and in distributing it in the longitudinal direction of the bridge.

## INSTRUMENTATION

Instrumentation was planned to detect any breakdown in the overall performance of the full-scale bridge and to reveal any local failure that might develop in the vicinity of the applied loads. The structure was instrumented with mechanical gauges for measuring deflection and for detecting relative movement between elements and with electrical resistance gauges for measuring strains in the beams and deck. Strain gauges were mounted on the top of the slab, bottom of the prestressed panel, and on the top and bottom of the beams. The slab gauges were in areas of maximum shear and bending to provide information that would indicate bond failure between slab and panel if such developed at the gauge (Fig. 9).

Linear motion dial gauges were installed to detect any relative vertical motion between abutting prestressed panels and to detect relative transverse and longitudinal movement between the prestressed panels and beams. Relative vertical movement between adjacent panels would indicate either a vertical crack through the cast-in-place slab above the panel joint or bond failure between the panel and cast-in-place slab. Either of these vertical movements would indicate a local deficiency in the structure. Any relative horizontal movement between the beam and the slab would indicate a failure of the bond betweeen these two elements.

## LOADING SYSTEM

Two types of loading arrangements were used to simulate design loads. Simulation of axle loads was accomplished with the hydraulic ram and loading pad arrangement. The two pads representing the dual wheels of a single axle of a design HS20 truck were 12 in. by 20 in. in plan and spaced 6 ft on centers. A loading beam spanned between these two pads and the loading ram was positioned at midspan of the beam. A hydraulic testing machine operated a ram for both the static and dynamic axle loadings. The system produced a nearly sinusoidal loading for these particular tests.

Simulation of a wheel load rolling across a transverse butt joint between prestressed panels was accomplished with two hydraulic rams acting on loading pads positioned on opposite sides of and adjacent to the transverse joint. The load alternated between the two rams, and one ram loaded and unloaded while the other remained inactive. The pulsator used to produce this alternating wheel loading produced a nearly trapezoidal load-time trace.

The static failure load tests were conducted by using the same load pad but with a 400-kip hydraulic ram substituted for the dynamic loading ram.

The procedure for evaluation of the behavior of the structure under cyclic loading was as follows:

Figure 9. Layout of electrical resistance and beam deflection gauges.

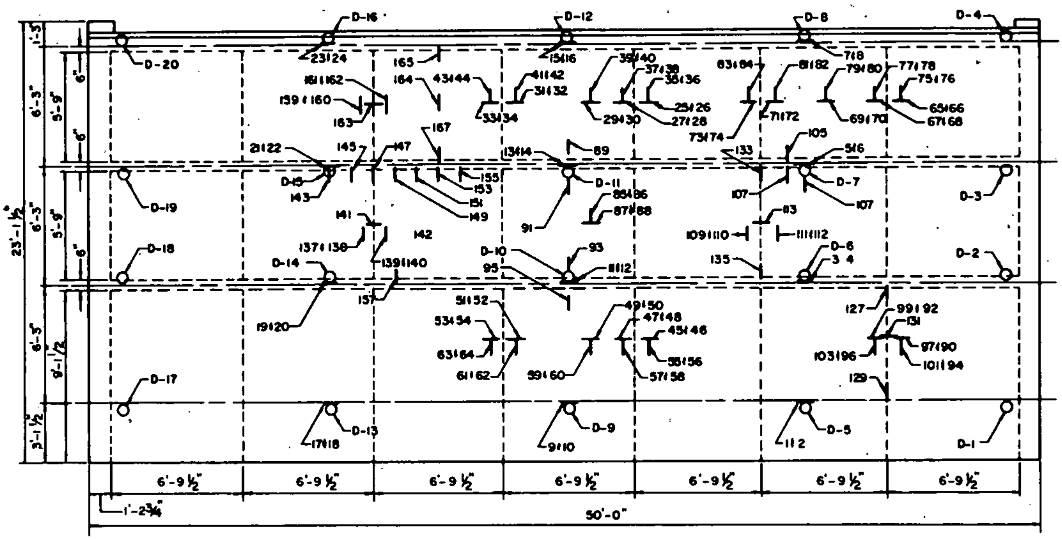
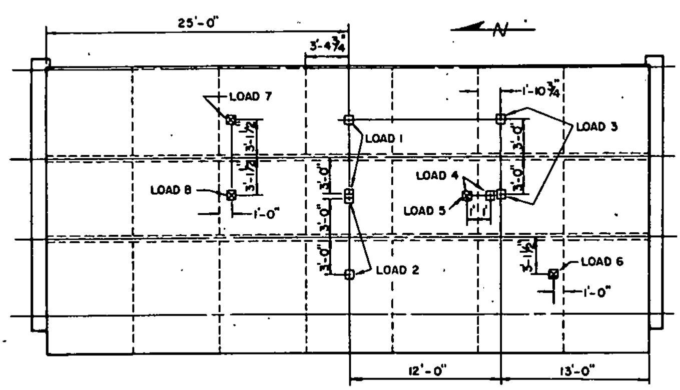


Table 2. Loads applied to structure.

| No. | Type | Purpose |
|---|---|---|
| 1 | Cyclic axle | To determine behavior of bond between panel and cast-in-place concrete with Z-bars and dowels, but no grout |
| 2 | Cyclic axle | To determine behavior of bond between panel and cast-in-place concrete with Z-bars, but no dowels or grout |
| 3 | Cyclic axle | To determine behavior of bond between panel and cast-in-place concrete with no Z-bars or dowels; one wheel pad was on grouted area, and the other was not |
| 4 | Cyclic, alternating wheel | To determine ability of deck to support load simulating wheel crossing joint between two panels with no dowels or Z-bars |
| 5 | Static wheel | To determine failure load of the structure in an area with no dowels or Z-bars |
| 6 | Static wheel | To determine failure load of the structure in an area with no dowels or Z-bars |
| 7 | Static wheel | To determine failure load of the structure in an area with Z-bars, but no dowels |
| 8 | Static wheel | To determine failure load of the structure in an area with Z-bars and dowels |

Figure 10. Position of load applications.



1. Determine the response to a static design load by reading the strains and deflections at all gauge locations.
2. Subject the structure to a number of cycles of load.
3. Again determine the response to static load.
4. Visually inspect the structure each time the static load is applied to determine whether any form of distress has occurred.
5. Compare the responses to static loads obtained in 1 and 3 above to determine whether any distress has occurred in the structure.

## PROGRAM OF TESTS

The structure was subjected to cyclic design loads and, after completion of these, to static failure loads. In the application of the cyclic loads, the condition of the structure was determined by periodically measuring its response to static load. Gauge readings under application of a static load were made before the cyclic loading, at predetermined intervals during loading, and after loading at each load position.

The loading plan, designed to accomplish a complete evaluation of the structures, is given in Table 2. The positions of the loads on the full-scale bridge model are shown in Figure 10. Loads 1 through 3 were cyclic loads and simulated an AASHO design axle load plus impact of 41.6 kips. Load 4 was a cyclic load and alternated on either side of a panel butt joint to simulate an AASHO design wheel load plus impact of 20.8 kips rolling across the joint. Loads 6, 7, and 8 were static failure loads.

## RESULTS AND DISCUSSION

### Repetitive Load Tests

The condition of the structure during cyclic loading was monitored by periodically determining structural response to a static load. A change in the response to a static load was considered to be the result of a change in the structural integrity of the bridge. Comparisons of beam deflections under static load before loading and after 2 million cycles indicated that no distress was caused by the loading. This is further supported by the fact that no slippage occurred between the beams and the slab as determined by the relative displacement dial gauges between those elements.

Experimentally measured strains at the upper and lower surface of the slab at locations in the proximity of the simulated wheel pads are given in Tables 3, 4, and 5. The generally close agreement between values obtained from the static load response tests (48 kips) made before and after cyclic loading indicate that no distress was caused by the loading.

Strain readings made before and after application of load 4, a 20.8-kip cyclic alternating wheel load, are given in Table 6. It is observed that the average ratio of strain readings after cyclic loading to readings before cyclic loading is 1.00 for data obtained with load on the north ram and 0.97 with load on the south ram. Closer observations of the individual gauge readings do not indicate any consistent trends in the data for load on the north ram, but do indicate a possible trend in the data for load on the south ram. The after-before ratios are consistently low for the top gauges and high for the bottom gauges. However, the largest difference in strains is only 22 $\mu$in./in., and this is not considered to be conclusive evidence of distress. No distress was observed visually, and none was indicated by data from static failure load test 5.

Prior to application of load 3, minute cracks were discovered above some transverse joints between panels. The widths of these cracks were measured with a microscope and were found to be 0.002 in. and less. These cracks were not found upon inspection

**Table 3. Experimental strains for load position 1.**

| Gauge Placement | No. | Strain ($\mu$in./in.) Before | After |
|---|---|---|---|
| Longitudinal | 25 | -11 | -15 |
| | 26 | -26 | -17 |
| | 27 | -14 | -13 |
| | 28 | -20 | -14 |
| | 29 | -56 | -56 |
| | 30 | +1 | -2 |
| | 31 | -16 | -14 |
| | 32 | -19 | -15 |
| | 33 | -15 | -14 |
| | 34 | -16 | -13 |
| Transverse | 35 | -18 | -23 |
| | 36 | +14 | +26 |
| | 37 | -39 | -27 |
| | 38 | +20 | +33 |
| | 39 | -74 | -76 |
| | 40 | +61 | +74 |
| | 41 | -28 | -24 |
| | 42 | +25 | +36 |
| | 43 | -39 | -38 |
| | 44 | +18 | +24 |

**Table 4. Experimental strains for load position 2.**

| Gauge Placement | No. | Strain ($\mu$in./in.) Before | After |
|---|---|---|---|
| Longitudinal | 45 | -9 | -5 |
| | 46 | -15 | -14 |
| | 47 | -3 | -4 |
| | 48 | -17 | -14 |
| | 49 | -32 | -32 |
| | 50 | +7 | +1 |
| | 51 | -7 | -7 |
| | 52 | -23 | -19 |
| | 53 | -23 | -24 |
| | 54 | -23 | -19 |
| Transverse | 55 | -21 | -21 |
| | 56 | +22 | +24 |
| | 57 | -31 | -27 |
| | 58 | +47 | +34 |
| | 59 | -77 | -62 |
| | 60 | +75 | +76 |
| | 61 | -32 | -29 |
| | 62 | +31 | +36 |
| | 63 | -11 | -12 |
| | 64 | +25 | —[a] |

[a]No data.

**Table 5. Experimental strains for load position 3.**

| Gauge Placement | No. | Strain ($\mu$in./in.) Before | After |
|---|---|---|---|
| Longitudinal | 65 | -4 | -10 |
| | 66 | -8 | -14 |
| | 67 | -6 | -11 |
| | 68 | -10 | -18 |
| | 69 | -12 | -18 |
| | 70 | -13 | -21 |
| | 71 | -13 | -18 |
| | 72 | -3 | -20 |
| | 73 | -16 | -25 |
| | 74 | +3 | +3 |
| Transverse | 75 | -7 | -14 |
| | 76 | +13 | +9 |
| | 77 | -12 | -20 |
| | 78 | +17 | +13 |
| | 79 | -52 | -59 |
| | 80 | +53 | +51 |
| | 81 | -50 | -58 |
| | 82 | +66 | +56 |
| | 83 | -54 | -57 |
| | 84 | +50 | +40 |

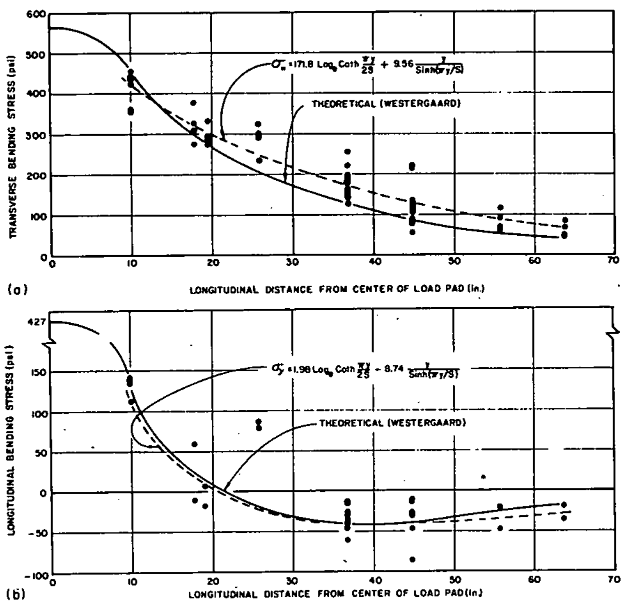



**Figure 11. Comparison of theoretical and experimental bending stresses in (a) transverse and (b) longitudinal directions.**

**Table 6. Experimental strains (in $\mu$in./in.) for load position 4.**

| Gauge | North Ram Before Loading | North Ram After 2,000,000 Cycles | North Ram After/Before | South Ram Before Loading | South Ram After 2,000,000 Cycles | South Ram After/Before |
|---|---|---|---|---|---|---|
| 109 | -94 | -97 | 1.03 | -71 | -60 | 0.84 |
| 110 | +74 | +74 | 1.00 | +56 | +63 | 1.13 |
| 111 | -76 | -70 | 0.92 | -100 | -78 | 0.78 |
| 112 | +62 | +63 | 1.02 | +81 | +92 | 1.14 |
| 113 | -88 | -92 | 1.05 | -99 | -95 | 0.96 |
| Avg | | | 1.00 | | | 0.97 |

after conclusion of load 2. Some cracks were in the vicinity of a panel joint near the north end of the bridge, far removed from the loads, and it is believed that they were caused by shrinkage, thermal strains, or both, and not by load. A limited number of core samples taken after completion of the testing program indicated that some of the cracks extended as deep as 2 in. below the surface, lacking at least 1½ in. of penetrating through the cast-in-place slab.

Comparisons of stresses computed from measured strains obtained during the static load response tests with those predicted by theory (6) are shown in Figure 11. Stresses were computed from the measured slab strains by using the plane stress relationships:

$$\sigma_L = \frac{E}{1-\mu^2}(\epsilon_L + \mu\epsilon_T) \tag{1}$$

$$\sigma_L = \frac{E}{1-\mu^2}(\epsilon_T + \mu\epsilon_L) \tag{2}$$

where

$\sigma_L$ = stress in the longitudinal direction of bridge,
$\sigma_T$ = stress in the transverse direction of bridge,
$\epsilon_L$ = strain in the longitudinal direction,
$\epsilon_T$ = strain in the transverse direction,
E = 52.3 and 56.5 psi for cast-in-place and prestressed panel concrete, and
$\mu$ = 0.15.

As expected, these stresses were observed to consist of a combination of benging stresses and in-plane or membrane stresses in the slab resulting from overall bending of the entire structure. These components of stress were separated mathematically in the following manner. The values of the component of bending stresses attributable to bending of the composite structure, at the top and bottom surface of the slab, were assumed to be proportional to the distances of these two surfaces from the neutral surface of the composite unit. The components of stress resulting from local bending of the slab were arbitrarily assigned equal values, of opposite sign, at the top and bottom surface of the slab. This allowed a unique solution for the distribution of the total stresses into the two components. Stresses computed from strain measurements made both before and after cyclic loading in each case were used for these computations.

Relationships presented by Westergaard (6) with adjustments for 50 percent edge fixity of the slab were used in arriving at the theoretical curves. These stresses on both the top and bottom surfaces were assigned the same sign as the bending moment at that point—compression on the top surface being positive moment. Curves were fitted to the experimental data by the method of least squares to facilitate comparisons between experimental and theoretical values. The function chosen to fit the experimental values was based on the form of the theoretical expressions for bending moments:

$$\sigma = C_1 \log_e \coth \frac{\pi y}{2s} + C_2 \frac{y}{\sinh \frac{\pi y}{s}} \tag{3}$$

where $C_1$ and $C_2$ were constants determined from the least squares fit. Equation 3 is shown in dashed lines in Figure 11. The maximum difference between the theoretical curve and the least squares fit to the data for transverse bending is about 45 psi and is less than 15 psi for longitudinal bending. It is further noted that at points close to the load, where maximum stresses occur, comparisons are very good.

Static Failure Load Tests

The static failure loads were applied in 10-kip increments, and strain and deflection

Figure 12. Crack pattern that developed during static failure load tests.

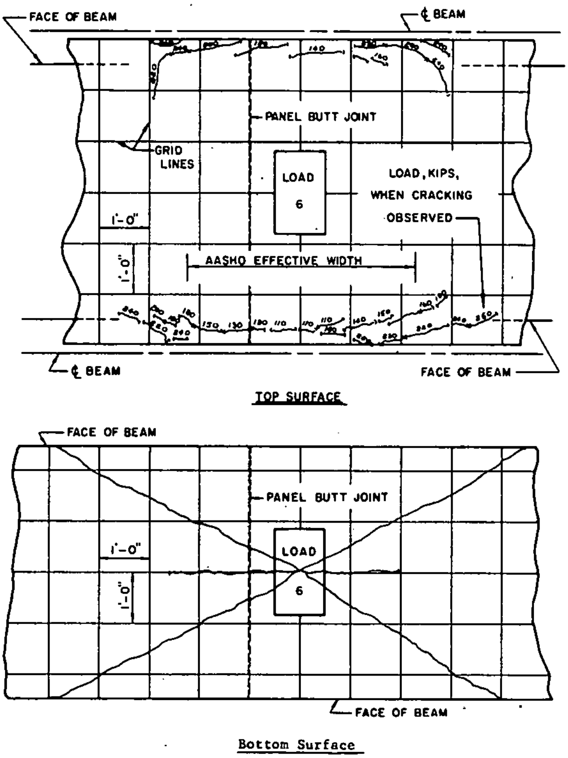
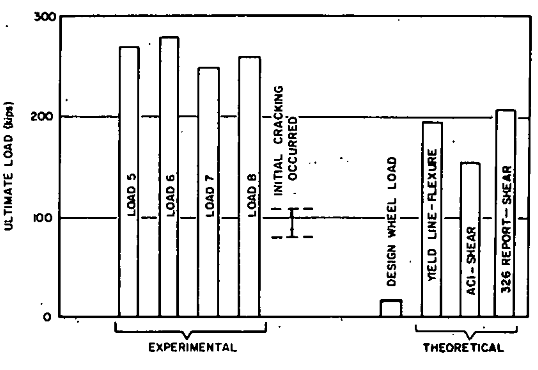



Table 7. Summary of static failure load test results.

| Load Position | Date Tested | Cracking Load (kips) | Failure Load (kips) | Failure Mechanism | Remarks |
|---|---|---|---|---|---|
| 5 | 3-9-72 | 90 | 270 | Punching shear | Significant flexural distress had occurred |
| 6 | 2-17-72 | 110 | 280 | Punching shear | Significant flexural distress had occurred |
| 7 | 4-6-72 | 120 | 250 | Punching shear | Truncated pyramid did not fully develop in adjacent panel |
| 8 | 3-23-72 | 80 | 260 | Punching shear | Significant flexural distress had occurred |

Figure 13. Comparison of experimental and theoretical failure load values.



gauge readings were made after each load increment. Cracking in the concrete in each test was first detected on the upper surface of the slab. These cracks occurred in the negative moment regions, on either side of the load, above the inside faces of the adjacent beams. The cracking load varied from 80 to 120 kips for the four tests. Cracking was observed on the lower surface of the slab, under the load pad, at a load 20 to 30 kips higher than the load causing the first top surface crack. With additional loading, the cracks on both surfaces grew until the load reached 220 to 240 kips. At this load, the rate of progression of the cracks slowed considerably, almost ceasing to extend but opening wider with additional load. The upper surface cracks generally extended parallel to the beams and eventually began turning away from the beam into the span of the slab being loaded. The lower surface cracks crossed to form an X under the load and extended until they reached the beams. A third crack, much shorter in length, passed through the center of the X and extended parallel to the beams. Additional load increments were applied until a punching shear failure occurred in each case. A typical pattern of the flexural cracking is shown in Figure 12.

The ultimate failure surface formed a truncated pyramid, typical of a punching failure in a slab. In tests 5, 6, and 8, the surface showed no apparent influence of the panel butt joint. However, in test 7, one segment of the surface intersected a panel joint and did not develop in the adjacent panel.

A summary of the static failure load test results is given in Table 7. A comparison of experimental and theoretical values of failure load is shown in Figure 13. A yield line analysis, using a two-fan failure mechanism, was made to determine the theoretical load (7). This mechanism and its dimensions were selected on the basis of the cracking pattern that was observed in the tests. In this analysis, the slab was transformed into an equivalent orthotropically reinforced slab (7). The ultimate load was computed to be 195 kips. Because the failure mode obtained experimentally was not a flexural failure, it can only be said that the ultimate flexural failure load for this slab was greater than the values attained when the punching shear failure occurred. It was expected that the predicted flexural failure load from the yield-line analysis would be greater than the 280 kips obtained experimentally, particularly since the analysis results in an upper bound solution. However, this is seen not to be the situation.

The first and most obvious factor that could have caused the actual flexural strength of the slab to be greater than that predicted by the yield-line analysis is the enhancement of the strength of the slab by in-plane compressive stresses. Such stresses existed in the longitudinal direction of the slab because the entire structure bent as a composite unit. Another source of in-plane compressive stresses has been observed in lightly reinforced slabs where the failure mechanism is confined to an interior portion of the slab (7). With application of a concentrated load and partial development of the failure mechanism, in-plane extension of the slab occurs in the area of the failure mechanism. This extension is restrained by the surrounding portion of the slab, and compressive in-plane stresses are thereby created in the area of the failure mechanism. This phenomenon, in a rigidly restrained slab, was observed by Wood (7) to increase the flexural strength of a lightly reinforced slab by 10.9 times. This same phenomenon also enhances the punching shear capacity of a slab.

The AASHO code relationship between a wheel load and the maximum slab bending moment for this structure is $M = 0.18P$. If this relationship is used along with the ultimate moment capacity of the section, an ultimate wheel load can be calculated. This load is somewhat meaningless because the load-moment relationship is intended for an elastic slab, and the relationship would be disrupted by yielding and redistribution of stresses. Nonetheless, such a calculation results in an ultimate load of 123 kips.

The shear strength of concrete slabs is a very complex subject and at present is handled with semiempirical methods of analysis. The primary difficulties are the lack of understanding of the behavior of concrete under multiaxial states of stress and the inability to determine the state of stress at any given point in a concrete slab. An analysis of the slab studied here is further complicated by the use of both prestressed and conventionally reinforced concrete. The American Concrete Institute code provisions for slabs and footings specify that the nominal shear stress for two-way action (neglecting the capacity reduction factor) be computed by $v_u = v_u/bd$, and this shear stress is specified not to exceed $4\sqrt{f_c'}$. If the average value of compressive strength for the slab from Table 1 is used, this method predicts the ultimate load, $V_u$, as 140 kips if a panel butt joint is assumed and 156 kips if no joint is assumed. The experimental failure surface did not intersect the panel joint in tests 5, 6, and 8, and the results of these tests should be compared with the predicted value of 156 kips. It should be realized that the ACI code provision is a simplified design equation intended to provide a lower bound on the ultimate load. An equation (8) that more closely approximates the lower bound of experimental data is

$$v_u = 4(d/r + 1)\, f_c' \tag{4}$$

where

d = effective depth of the slab and
r = side dimension of the loaded area.

This equation results in an ultimate shear strength of 457 psi if the average compressive strength for the slab is used. The resulting ultimate punching shear loads are 186 and 210 kips for an assumed joint and no assumed joint respectively. Experimental values, given in Table 7, exceed these theoretical values by 25 to 35 percent. Experimental values given in the ACI committee report (8) exceed the theoretical values by 0 to 100 percent.

Three bridges of this type, located in Grayson County, Texas, were built in the early 1960s and have been serving satisfactorily since then. A visual inspection of these bridges was conducted in the first phase of this study (2), and crack patterns on the deck surfaces were mapped. Cracking above the panel joints was found on only two of the bridges. These cracks were rather extensive in some areas and extended to about half the depth of the cast-in-place concrete. Although they are undesirable from a durability standpoint, especially in severe climates, they have not damaged the structural integrity of these bridges. Load tests were performed, and core samples were taken from one of the bridges to determine the condition of the bond between prestressed panels and cast-in-place deck. This examination indicated no further distress in the bridges nor any signs of bond failure.

## SUMMARY AND CONCLUSIONS

A single-span, full-scale, prestressed panel bridge was evaluated experimentally in the laboratory. The structure was subjected to cyclic applications of design load plus impact and to static failure loads. It performed satisfactorily under all test conditions.

Theory presented by Westergaard (6) (the basis of present design specifications) predicts local bending stresses in the slab of the structure studied with reasonable accuracy if in-plane stresses resulting from spanwise bending of the entire structure are ignored.

Two million applications of simulated design axle load plus impact were accomplished at three locations on the bridge structure. The bond at the interface between the prestressed panels and the cast-in-place concrete performed without any indication of distress under these cyclic loads.

Two million cycles of design wheel load plus impact alternating on opposite sides of a panel butt joint were applied at one location on the structure without causing distress. Satisfactory performance was exhibited by the bridge slab when subjected to static failure loads. The lowest value of cracking load measured experimentally was 3.8 times the design wheel load plus impact, and the lowest measured ultimate load was 12 times the design wheel load plus impact.

The failure surfaces that developed in the static failure tests intersected and continued across the panel to the cast-in-place interface and were not influenced by the interface.

The following conclusions were drawn from the results of the experimental work reported.

1. The bond at the interface between the prestressed, precast panels and the cast-in-place concrete performed without any indication of distress under cyclic design loads and static failure loads.
2. No distinction in performance among those areas of the deck with mechanical shear connectors (Z-bars), with grouting treatment, and without special treatment could be made after 2 million applications of design load or static failure load tests.
3. Wheel loads were transferred and distributed across transverse panel butt joints in a satisfactory manner. Those joints with supplemental reinforcing gave no indication of superior performance under 2 million applications of design wheel load or static failure loads when compared to those joints without supplemental steel.
4. With this type of construction, some small transverse cracking in the cast-in-place deck over panel butt joints is to be expected as a result of thermal and shrinkage effects. Such cracks have not been found to be detrimental to the overall performance of a bridge.

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# bridge replacements with precast concrete panels

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Methods of replacing existing distressed bridge deck slabs with precast concrete panels are presented. The general nature of the problem is explored, and the nature of force transfer in a bridge deck system is examined. The various methods of connection between adjacent slab panels and between the slab and stringer are then developed on a rational basis. Eight types of slab-stringer connections are detailed. Many of these would develop composite action. Some connections are welded, and others are bolted. Several types of shear connectors are used. A construction method is suggested that (a) presents a rehabilitated structure that is compatible in strength to the original, (b) provides rapid construction that causes minimal interference to normal traffic, and (c) allows full traffic capacity to be maintained during peak periods.

•Distress of cast-in-place concrete bridge decks is a widespread problem. The nature and severity of the problem and the difficulty of obtaining a satisfactory solution are recognized by various highway agencies. A rapid, structurally sound, and economic method of replacing distressed bridge decks is needed for all highway systems.

It is generally recognized that any viable method of rapid bridge deck replacement should make use of prefabricated units. The three major requisites considered for development of the proposed bridge deck replacement system are as follows:

1. The finished structure should be compatible in strength to the original;
2. Construction should be rapid and should cause minimal interference with the normal traffic; and
3. Full traffic capacity should be maintained during peak periods.

With these conditions as guidelines, the New York State Thruway Authority initiated a program of research and development, which included prototype construction. The research included a comprehensive review and evaluation of available methods of industrialized bridge deck construction that use prefabricated components (1-8). The results of this evaluation are not included, but the connection details presented reflect this research. This paper summarizes the results of the research and development program up to the beginning of the prototype construction.

## NATURE OF THE PROBLEM

A typical bridge is shown in Figure 1. The bridge has horizontal and vertical curves,

Figure 1. Typical bridge span.

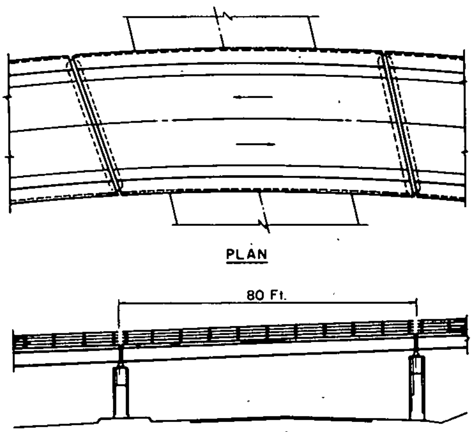



Figure 2. Basic types of load transfer in bridge deck and stringer system.

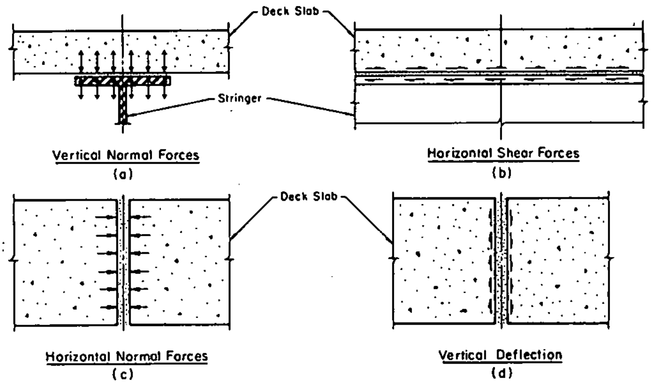



Figure 3. Transverse slab joints: (a) type F-F and (b) type M-F.

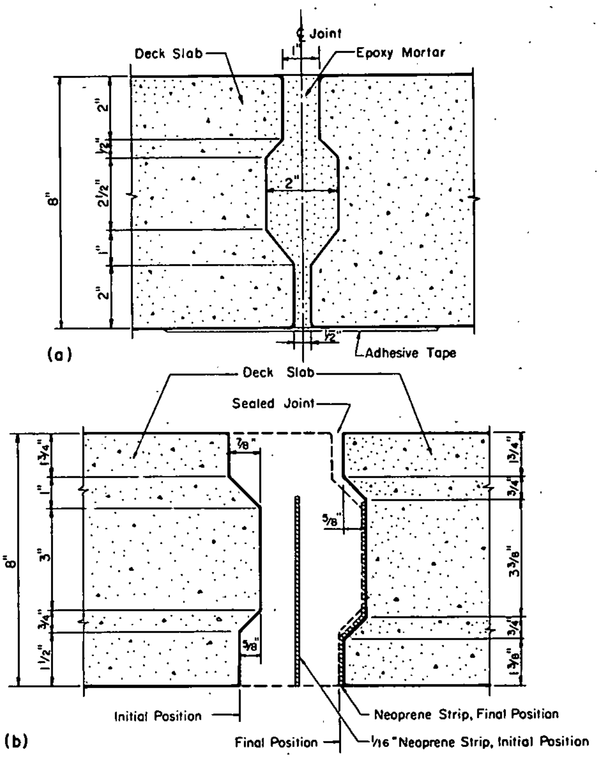
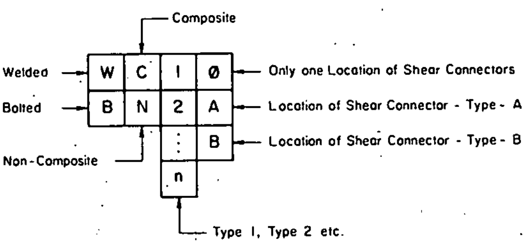



Figure 4. Connection system designations.



a skewed span, and a super-elevated roadway. The deck-stringer action is composite. Presumably traffic cannot be removed from the bridge during the reconstruction period. There are three distinct but interrelated problem areas: (a) developing a structurally suitable deck replacement system, (b) finding a safe, fast, and economical method of removing the existing deck slab, and (c) developing a construction procedure that causes the least interference to the traffic both on the bridge and under the bridge, which could be a roadway, a railway, or a waterway.

The greatest structural difficulty is to achieve proper matching, contacts, and connections among the various elements. Given normal tolerances of manufacturing, prefabrication, and construction and the realities of existing conditions, components do not fit together the way they are designed to. Moreover, if the existing deck is composite, it is probable that the replacement deck will also have to act compositely, especially inasmuch as the loading on many bridges might have increased since their construction. Requirements for composite action and construction time constraints impose further difficulty in developing a proper deck and stringer connection system.

Understanding the nature and basic mechanics of load transfer in the deck and stringer system would help us develop replacement methods on a systematic and rational basis. Types of load transfer are shown in Figure 2. The dotted area shown between two elements is a hypothetical force transfer medium. Any device used must be able to duplicate its function. The following load transfers are required.

Vertical normal forces [Fig. 2(a)] must be transferred between the slab and the top of the stringer. This is a basic type of transfer required for composite and noncomposite decks. The dead load and the wheel loads taken by the slab must be transmitted vertically down to the stringer (heavy arrow heads). Vertical upward forces caused by the negative reaction from the slab, which is continuous over several stringers in the transverse direction of the bridge, also have to be transmitted (light arrow heads). A general holding down action is also required to avoid lifting up and bouncing of the slab.

Horizontal shear forces [Fig. 2(b)] must be transferred between the bottom of slab and top of stringer. This is required mainly for composite action. Forces due to braking and wind also have to be transferred. Because the force transfer media [Fig. 2(a) and 2(b)] are placed in the same location, i.e., between the slab and stringer, they must be compatible.

Horizontal normal forces [Fig. 2(c)] must be transferred at the transverse joint between two adjacent precast slabs. This is required for composite action only. By virtue of this force transfer, the precast slab will be effective as the compression flange of the composite deck-stringer system.

Vertical deflection [Fig. 2(d)] compatibility must be maintained at the transverse joint between the two adjacent precast slabs. This is required for both composite and noncomposite action to provide a proper riding quality and to maintain the integrity of the wearing surface. Transfer of bending moment is not necessary. Therefore, the empirical AASHO formulas for longitudinal bending moment in the slab need not be used for design of these slabs. Force transfer media [Fig. 2(c) and 2(d)] must be compatible.

## PROPOSED SLAB AND STRINGER CONNECTION METHODS

A series of slab and stringer connection methods have been developed for composite and noncomposite action, and these meet the requirements stipulated previously.

### General Features

Transverse Joint—Two types of transverse joints are shown in Figure 3. Joint type F-F

Figure 5. Types of deck slab-stringer connections.

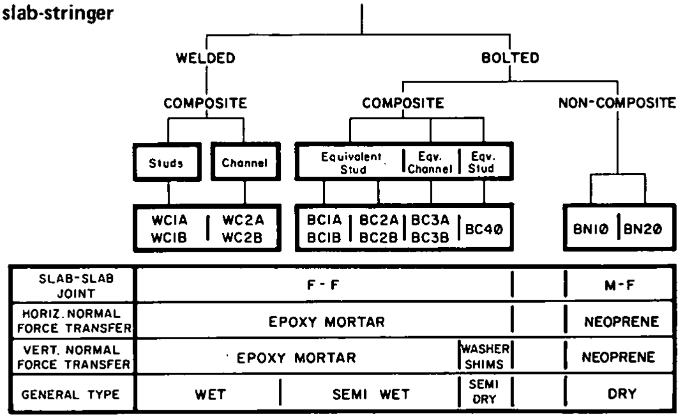



Figure 6. Connection type WC1A.

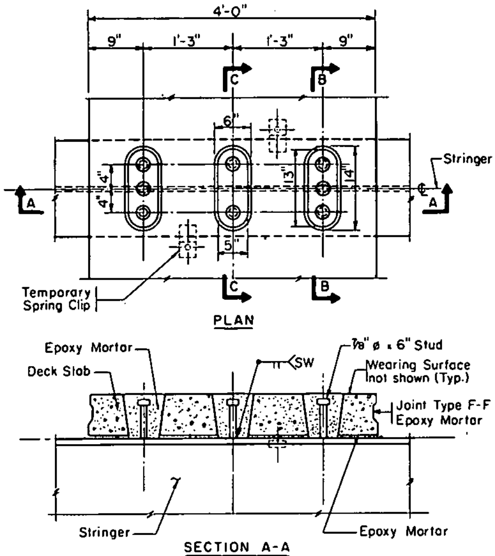



Figure 7. Connection types WC1A and WC1B.

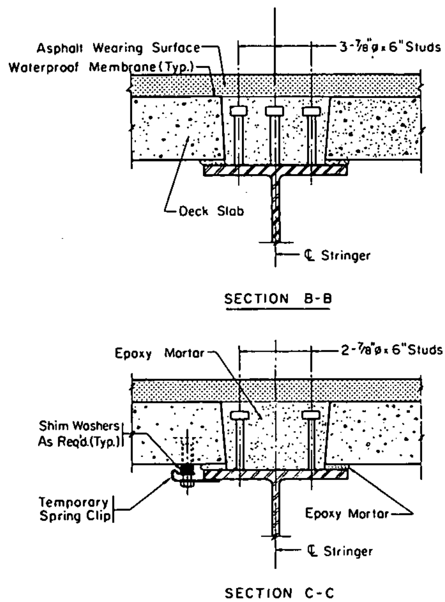



Figure 8. Connection type WC1B.

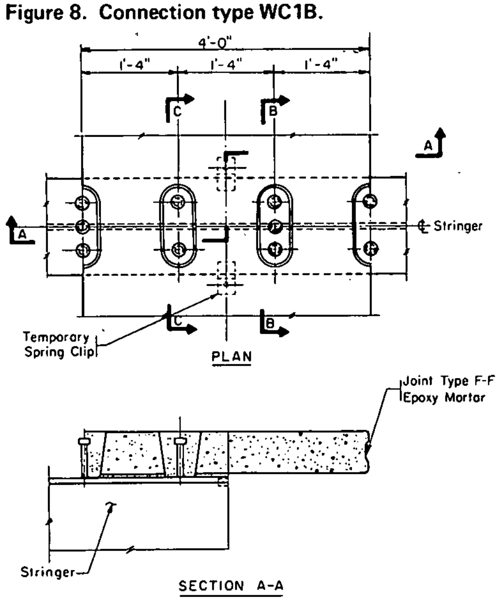
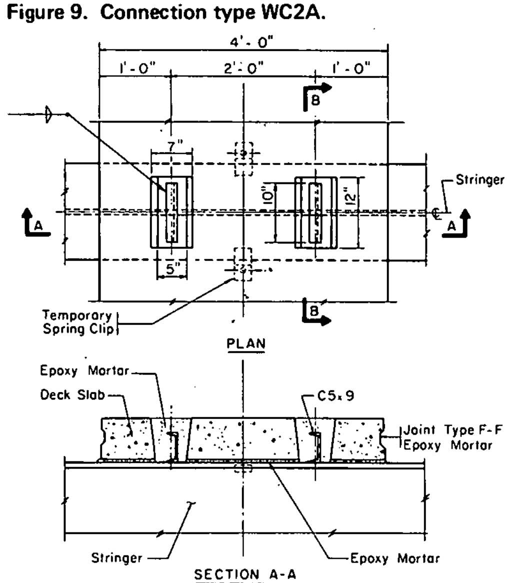



Figure 9. Connection type WC2A.



is suggested for all composite decks. Adjacent slabs are placed side by side as shown. Half-inch spacers can be used. Adhesive tape is applied to the bottom of the joint. Epoxy mortar is poured from the wider opening at the top and is allowed to seek its own level and set. Joint type M-F is suggested for noncomposite deck. This detail can be used with or without the use of posttensioning in the longitudinal direction of the bridge.

Connection Types and Designation—Over the years, the welded stud and welded channel shape have become the standard shear connectors used in cast-in-place composite deck construction. Because the precast deck is expected to duplicate the action of a cast-in-place deck, it would be logical to consider details that make use of these proven shear connectors. Systematic modification of these shear connectors results in other types of connection systems. Because horizontal shear transfer through the shear connectors plays a key role, the connection types are designated on the basis of the method of fastening these connectors to the stringer and the type of connectors. The principle used in designating the connection system is shown in Figure 4. The various proposed methods of deck slab stringer connection are shown in Figure 5. The designations will be evident as the various types of connections are explained below.

It was desired to develop a completely dry composite system. This, however, has not been achieved. A completely dry system is available for noncomposite deck only.

## Slab-Stringer Connections

Type WC1A—This type of connection is a welded composite connection using field-welded studs. The essential features of this system are shown in Figures 6 and 7. The top of the stringer is accurately marked to position the studs to fit the tapered pockets in the slab, and the studs are welded in place. (Number of shear studs per 4-ft width can easily be varied, if required by design.) The top of the stringer is then thickly "buttered" with a nonflowing or gel type of epoxy mortar. The precast deck slab is then lowered on the stringer, next to an adjacent deck slab that has been placed before. The slab presses down on the epoxy mortar. The temporary clips are bolted to fasten the slab to the stringer, and the transverse joint is taped. The stud pockets and transverse joint are filled with poured-in epoxy mortar.

Construction proceeds to the next slab as the epoxy sets in the previous one. A completed deck will transmit all the joint forces mentioned earlier. Although the epoxy mortar pressed between the deck and the stringer is capable of transmitting horizontal shear, this is not relied on, and the shear studs are designed to take mechanically the full horizontal shear. Also, the tapered pockets provide the hold down force. This construction method is common to many of the connection types mentioned later.

Type WC1B—This is a variation of the previous method and is shown in Figure 8. Here, the stud lines are shifted in the longitudinal direction of the stringer. Two stud lines are placed at the transverse joint, resulting in an increased effective area of the slab in the transverse direction, which may be desirable.

This type of variation of type B from type A, by shifting the studs in the longitudinal direction of the stringer, is common to several of the methods of connections (Fig. 5) and will not be described in detail for all the applicable cases.

Type WC2A—This is a welded composite connection type 2-A that uses field-welded channels (Figs. 9 and 10). In certain cases a field fillet weld may be more reliable than the stud weld. Type WC2B is shown in Figure 11. Type WC2A is recommended over the type WC2B because, in the former, there are at least two connectors within the width of the precast slab.

Type BC1A—This is a bolted composite connection type 1-A (Figs. 12 and 13). The

**Figure 10. Connection types WC2A and WC2B.**

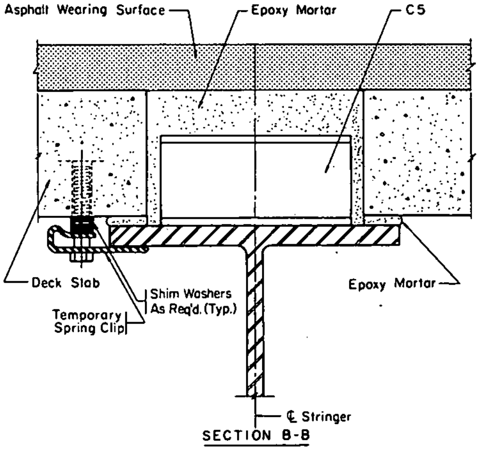



**Figure 11. Connection type WC2B.**

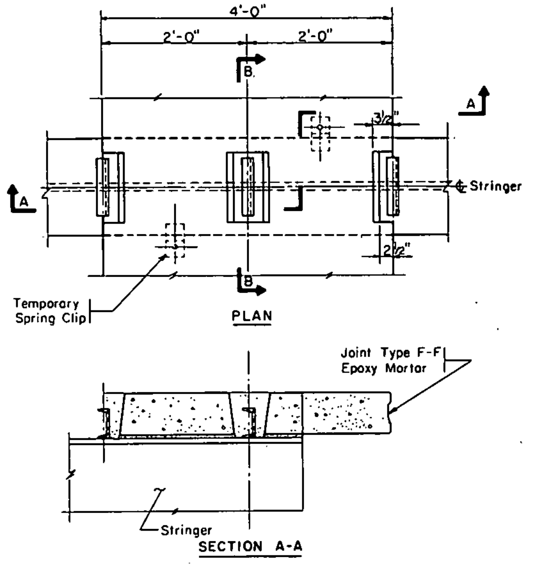



**Figure 12. Connection type BC1A.**

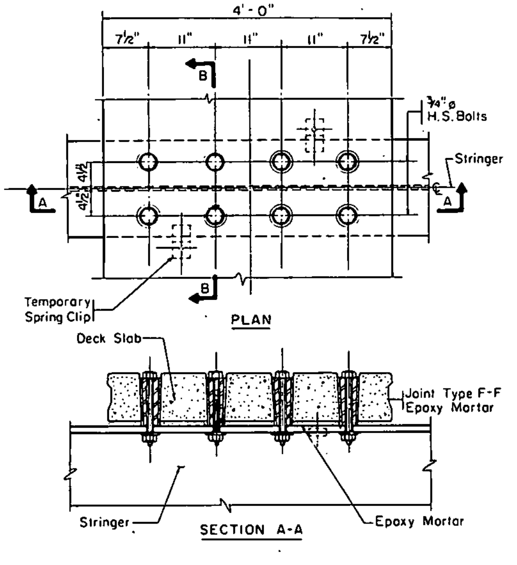



**Figure 13. Connection types BC1A and BC1B.**

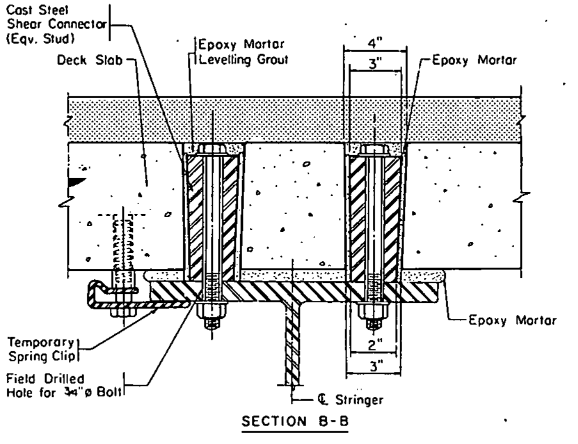


Figure 14. Connection types BC2A and BC2B.

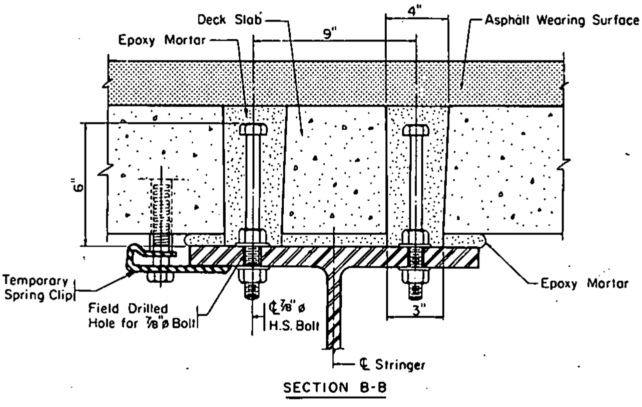



Figure 15. Connection type BC3A.

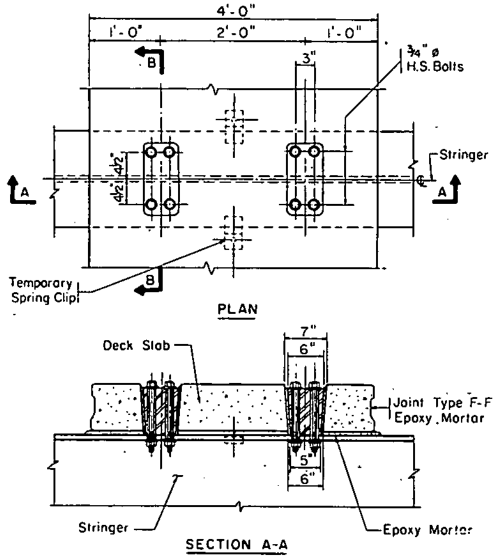
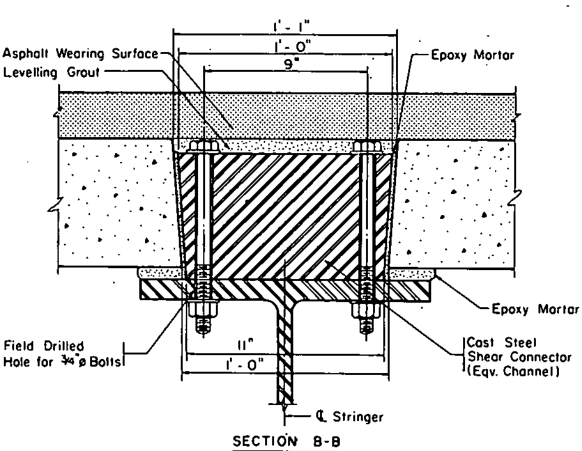



Figure 16. Connection types BC3A and BC3B.



holes in the top flange of the stringer are drilled in the field. The tapered bushing types of cast steel shear connectors are "buttered" at the exterior surface with a thick epoxy mortar and are pushed down into the tapered pocket in the precast deck slab. A high-strength steel bolt fastens the bushing to the stringer flange. The assembly effectively acts as a shear stud. Type BC1B (not shown) is recommended over type BC1A.

Type BC2A—Type BC2A is similar to type BC1A and is shown in Figure 14. The bushing is not used. The high-strength bolt itself with two nuts serves as the shear connector (9). The tapered pocket in the precast slab is filled with poured-in-place epoxy mortar. Type BC2B (not shown) is recommended over type BC2A.

Type BC3A—This is the bolted equivalent of the welded channel shear connector and is shown in Figures 15 and 16. The construction procedure is similar to that of type BC1A. Type BC3A is recommended over type BC3B (not shown).

Type BC40—For type BC40 the connection between the slab and stringer is completely dry (Fig. 17). The pipe sleeves with top and bottom plates are integrally cast with the slab. The holes in the stringer flange are drilled after the slab is in place. The space between the slab and the stringer, if any, is snugly filled with shim washers. The slab is then tied down with high-strength bolts.

Type BN10—This is a slightly modified version of a method originally developed at Purdue University (3) and later used in two Indiana bridges. The system is noncomposite (Fig. 18). Use of pressed-down epoxy mortar is suggested. If the rail clips are tack-welded to the stringer flange, the precast slabs would have to be posttensioned in the longitudinal direction of the bridge in order to hold the slabs together.

Type BN20—This bolted noncomposite connection (Fig. 19) is suggested as an alternate to type BN10. In detail, it is similar to type BC40. Fewer bolts are used, and the bottom plate attached to the pipe sleeves is not needed. Use of interference-body bolt will eliminate the need of posttensioning the precast slabs in the longitudinal direction of the bridge, as used in the Purdue tests and the Indiana bridges. Neoprene shims are used to keep the system completely dry; epoxy mortar can be used if desired.

## SEQUENCE AND METHOD OF CONSTRUCTION

The proposed method and sequence of construction are shown in Figures 20 through 27.

During most of the construction, traffic could be maintained at least in one lane. During peak periods, construction could be stopped to maintain two full lanes of traffic. The bridge will also be open during any peak period when construction is adjourned.

The key to the proposed construction method is a movable hinged plate assembly shown in Figures 28 and 29. The elements of the assembly can be prefabricated, brought to the site in parts, and assembled at site before construction begins. The hinge plate assembly can be used as standard equipment in deck replacement construction of several bridges.

With the method described, the time of shipping the precast deck slabs to the site, removing the existing deck slab, and placing the new slab can be scheduled to result in optimum use of manpower, material, and time available and to cause the least interference to the traffic.

## PROTOTYPE CONSTRUCTION

Plans for bridge deck replacement using the results of the research and development

Figure 17. Connection type BC40.

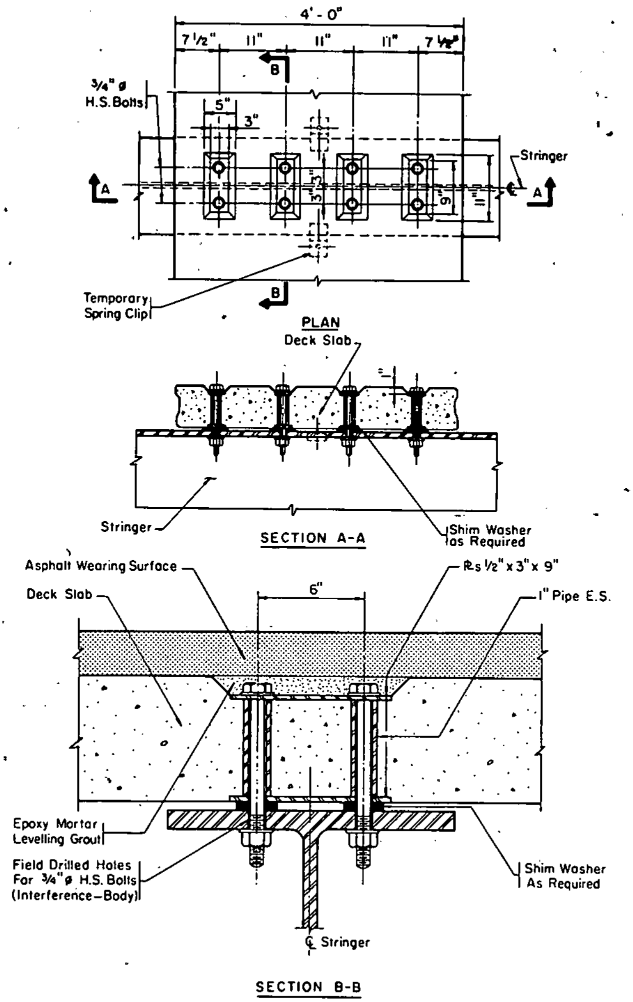



Figure 18. Connection type BN10 (modified Indiana version).

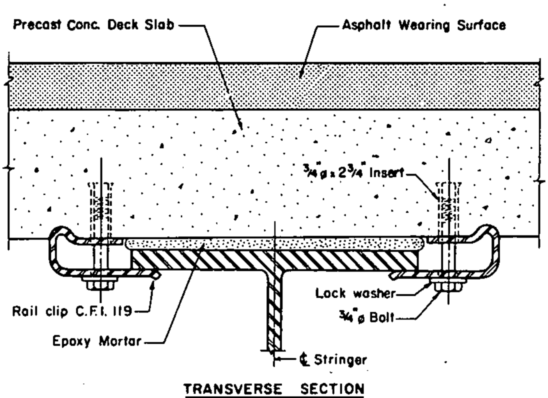


Figure 19. Connection type BN20.

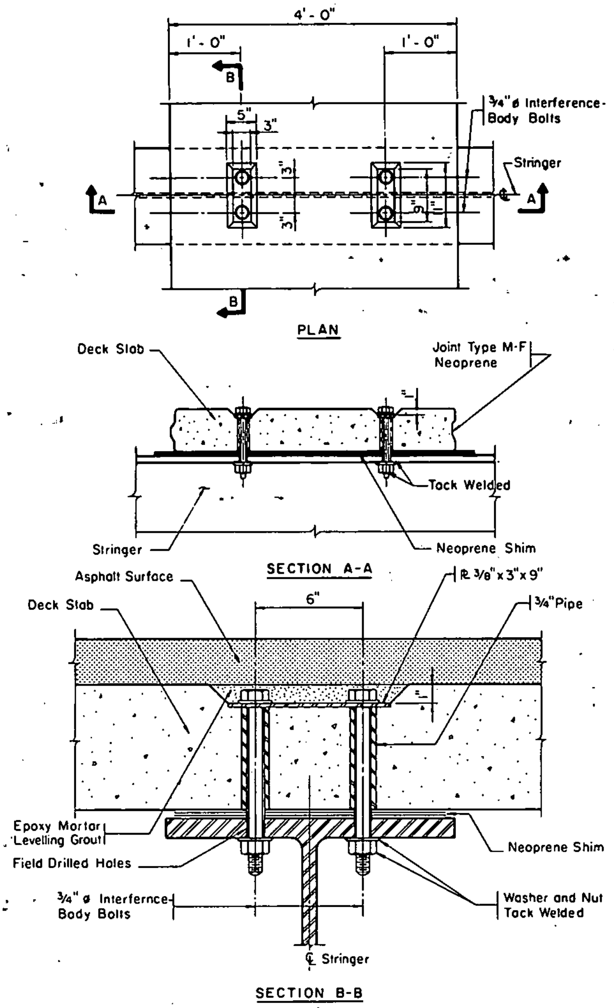



Figure 20. Typical two-lane bridge deck slab.

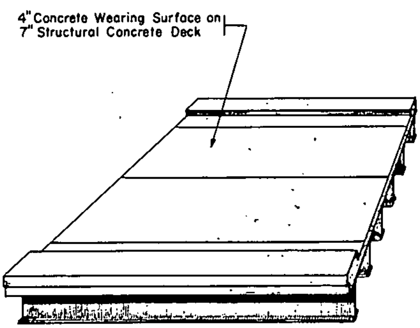
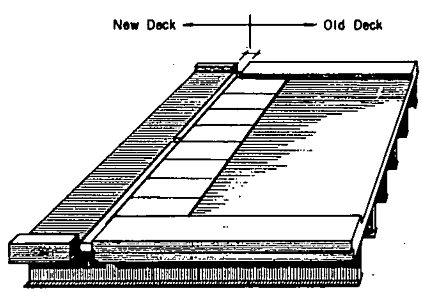
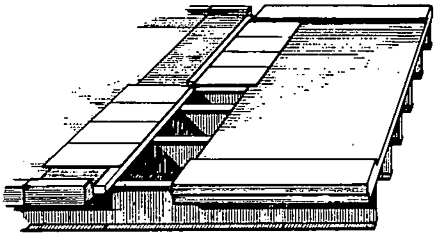



Figure 21. Hinged plates in place.

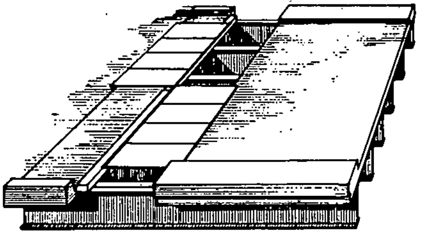



1. Cut and remove slot through deck across roadway for placement of hinge assemblies.
2. Install hinge assemblies and deck plates. Attach plates at hinge and rest other end on old deck.

Figure 22. Deck removal.

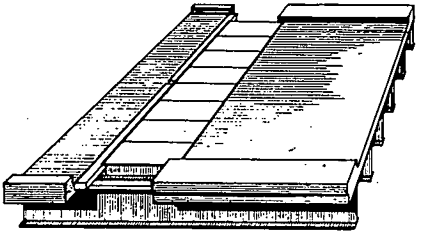


3. Maintain traffic on one lane during work period.
4. Remove length of deck under plates, opening individual plates as required.



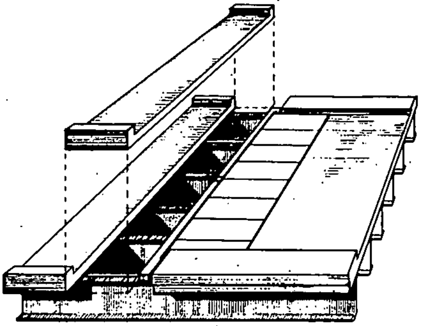
5. Shift traffic to opposite lane.
6. Remove remainder of deck.

Figure 23. Maintenance of traffic during construction.



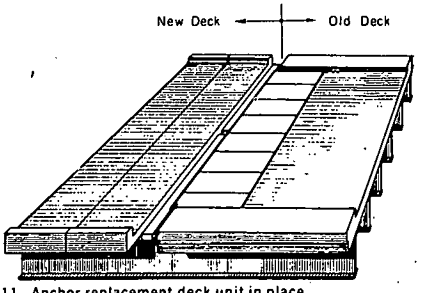
7. During peak traffic hours and when construction is suspended maintain traffic across open deck with steel plates in closed condition.

Figure 24 Replacement deck inserted.



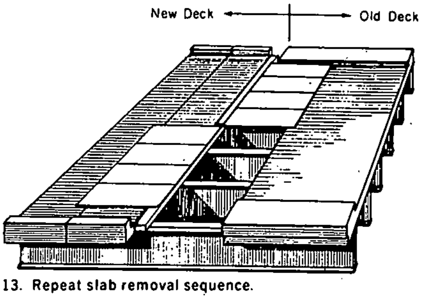
8. Move hinge plates forward, leaving opening in deck. Half length of assembly can be moved at a time to maintain traffic.
9. Prepare the surfaces as required for the connection method to be used. Shear connectors can be attached to stringers and epoxy mortar can be buttered on the top of the stringers at this stage. Traffic can be maintained on one lane by turning the plates over by 180°.
10. Drop in replacement slab unit.

Figure 25. Replacement deck unit connected.



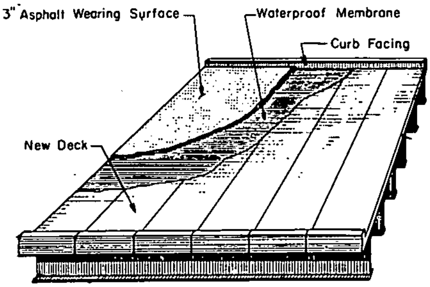

11. Anchor replacement deck unit in place.
12. Install shear connectors. Complete the deck to stringer connection and the transverse joint.

Figure 26. After one section of deck is completed, the process is repeated.




13. Repeat slab removal sequence.

**Figure 27. Finishing of completed deck replacement.**




14. Add finishing accessories when weather permits.

**Figure 28. Hinge plate assembly.**

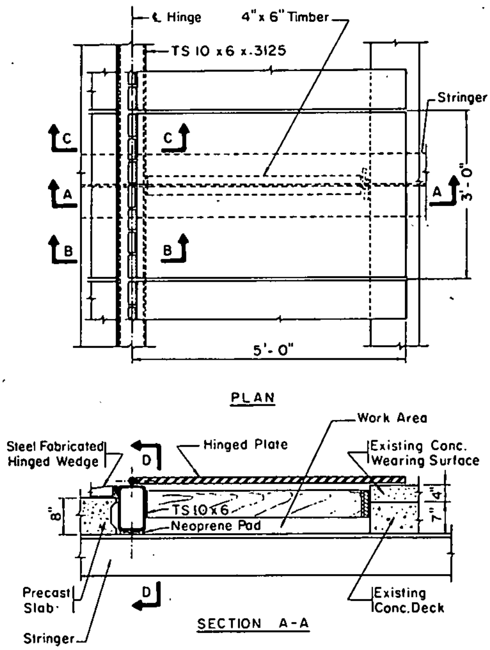
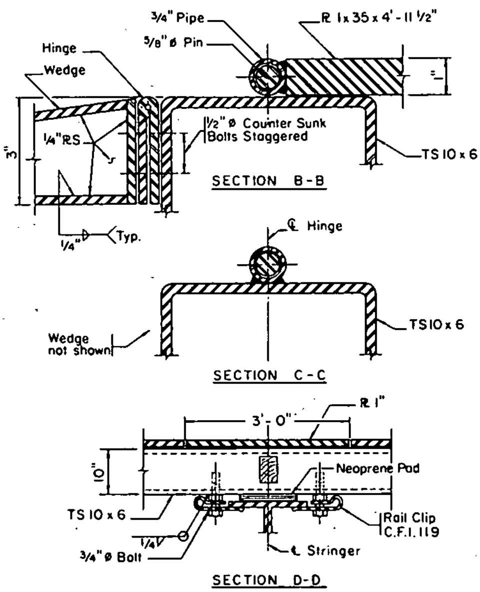



**Figure 29. Hinge plate details.**



are in progress at the New York State Thruway Authority. Contract plans have been developed for one span of a straight, square bridge to be used as a prototype to test several of the various types of connections developed and the general method and sequence of construction. In connection with this work, tests of the epoxy mortar were conducted to evaluate shrinkage properties, ability to set under vibratory loads, and stud pull-out resistances.

In addition, contract plans are being developed for a second prototype bridge similar to the one shown in Figure 1. This prototype will introduce the problems of skew and curvature. On the basis of these prototype construction projects, it is anticipated that a standard method applicable to most of the New York State Thruway bridges can be established.

## ACKNOWLEDGMENT

The research and development program is being executed under the direction of John P. Pendleton, chief engineer of New York State Thruway Authority, and John A. Tiesler, executive director of New York State Thruway Authority.

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# systems building for steel bridges

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This paper describes component bridges used in Europe and South America. These bridges are designed on the sectional principle wherein individual parts are standardized and are fabricated with such accuracy that they can be interchanged and used on any point of the bridge. Three types of component bridges are discussed: fast-assembly bridge of the Rheinhausen type, D (from the German word for triangle) bridge, and SKB bridge. Discussion of both design and erection of these bridge types is presented.

•Basically, there are two types of construction in steel bridge building: (a) bridges designed for clearly defined, unchanging conditions, built only once, and normally having a certain amount of time available for design, procurement, shop fabrication, and field erection; and (b) bridges designed for several purposes and varying conditions, for multiple reuse in different locations, for quick delivery and short erection time. Examples of the latter type are

1. Temporary road or rail crossover above a subway construction pit;
2. Temporary parallel diversion of road or railroad traffic while a permanent bridge is being built or replaced;
3. Temporary relief of brief traffic peaks during a trade fair, exhibition, or sporting event;
4. Quick elimination of danger points such as level crossings between road and rail traffic; and
5. Emergency replacement of bridges destroyed or damaged by a disaster.

Such tasks can only be fulfilled by bridge equipment that is kept in store, is quickly supplied, erected, and dismantled, and is suitable for reuse without loss in material.

## COMPONENT BRIDGES

The first question is, What features must bridging equipment have in order to meet these requirements? The bridge must be designed on the sectional principle with maximum standardization of individual parts. By means of jigs and fixtures, the elements must be fabricated with such accuracy that they can be interchanged and used at any point of the bridge. Elements must connect and disconnect in the simplest way. This is best achieved by fitted bolts. The holes for fitted bolts have to be shop-drilled to the final nominal size. It must be possible to combine the individual elements into overall structures for many uses. The overall structure must have variable cross section, span length, bearing arrangement, and vertical and horizontal curvature.

After these general considerations, the next questions are, Where have bridges of this type been used? Have they performed well?

Various countries have developed and tested different kinds of bridge systems. The following are three types developed and tested by Krupp in West Germany:

1. The fast-assembly bridge of the Rheinhausen type, a bridging system for road traffic;
2. The D bridge, a bridging system for road traffic; and
3. The SKB bridge, a bridging system for rail and road traffic.

Fast-Assembly Bridge of the Rheinhausen Type

The dimensions of the fast-assembly Rheinhausen bridge system are based on bridge class 30 of DIN 1072. The standard lower chord design of the main girder permits span lengths of up to about 30 m. If the lower chord and possibly the roadway plate are specially strengthened greater span lengths can be obtained without difficulty. The Rheinhausen fast-assembly bridge is a deck bridge of all steel construction, with a lightweight steel deck. The completed bridge is formed by a beam grillage rigid in longitudinal and transverse directions and continuous over all supports. In operation, the roadway panel is jointless over the total bridge area. Web spacing of the structure is 3 m. Cross girders are provided at 6-m intervals. The fast-assembly bridge can be erected with or without walkways. Although the fast-assembly bridge is designed for temporary use, its design fully corresponds to that of permanent bridges.

The basic elements of the fast-assembly bridge are the main girders. They have an open cross section and consist of the top roadway panel with roadway ribs in transverse arrangement, the web plates, lower chords, and a welded-in cross beam in the center girder. Four variations are available: two-web or one-web design, each 12 m long, and two-web or one-web design of 6-m length. Main girders 6 m long are mostly installed at the bridge ends. They can be delivered with a uniform web height and also with raised web. A raised web design is advantageous in the case of a low ramp height. For all main girder types the web plate height is uniformly 1 m.

For a single-lane cross section, a two-web main girder element is used. If the useful lane width of 3 m is not sufficient, the roadway can be widened to about 3.20 m by enlarging the element and widened further to 3.80 m by the addition of a special curb. Two-lane and multilane cross sections are achieved by combining the necessary number of one-web main girder and two-web girder elements. In this manner useful roadway widths of 3, 6, 9, 12 m, and more are produced. In-between roadway widths are possible, too. For the fast-assembly bridge at Hannover, two-lane widths of 7.50 m were required. They were built by arranging two 3.20-m-wide two-web main girders at the outer borders of the cross section, with an intermediate plate of 1.10-m width. This cross-sectional design also offers the advantage of a two-lane cross section 7.50 m wide, which can be divided into two 3.20-m-wide single-lane roadways.

The main girder elements are normally straight units. If bridges are to have curvatures in the plan view or elevation, this curvature is approximated by a bent polygonal course with bends at the joints spaced 12 m apart. Its accuracy is sufficient for practical purposes. Curvatures in elevation have large radii, and the bends at the joints can be achieved by wedge-shaped drilled web butt straps. Curvatures in the plan view, however, may have very small radii. In this case, a wedge-shaped drilled butt strap arrangement may no longer be enough. Special wedge plates will have to be arranged at 12-m intervals to form the curvature needed. The dimensions of these wedge plates are variable, and they can be adapted to any curvature. As compared with a main girder uniformly curved, the design of a polygonal approximation of the curvature as stated would offer a great benefit so that, in many uses of the equipment with considerably different conditions of curvatures, only butt splices and wedge plates would have to be exchanged, whereas all main girder elements and cross girders would be used without any modification.

The versatility of this fast-assembly bridge design may be seen from the following five examples.

In Barcelona, a fast-assembly, 144-m-long, four-lane bridge was delivered. It had a useful roadway width of 12 m. The spans ranged from 13 to 18 m. In its plan view the bridge was straight. The maximum gradient was 10.6 percent. A synthetic coat was applied to the roadway. The weight of steel was 330 tons. This fast-assembly bridge served to cross a railroad line where the previous level crossing had resulted in interruptions of traffic.

A fast-assembly, 131-m-long, four-lane bridge with a useful roadway width of 12 m was erected in Caracas. The span lengths range from 27 to 37 m. In its plan view the bridge is straight, and its maximum gradient is 7.85 percent. The fast-assembly bridge crosses a small river and a highway and is intended to be in use for a long time. An asphalt coat was chosen for the roadway.

In Rotterdam a fast-assembly bridge was temporarily used to cross a road-building site. Its total length was 53.60 m, and it had a useful roadway width of 9 m and a span length of 26.80 m. Because of its short-term use, a synthetic coat was applied.

The most convincing example of the versatility offered by the fast-assembly bridge system in many locations is the bridge across the Aegidientorplatz in Hannover. This fast-assembly bridge has a central two-lane part of 7.50 m of useful roadway width and separate single-lane descents of 3.20 m useful width. The total length of the bridge is 726 m, 504 m allotted to the single-lane descent branches and 222 m to the two-lane central bridge. The span lengths range from 11 to 28.50 m. The maximum gradient is 6 percent; the crossfall of the deck ranges from 1 to 3 percent. In its plan view the bridge is greatly curved. The center angle of the arc is about 80 deg, and the radius of curvature is about 90 m. Because of its great length and the horizontal curve, the bridge length was subdivided into three independent sections separated by expansion joints. The roadway coating is a 1-cm-thick layer of mastic and a 2-cm-thick melted asphalt layer. The steel superstructure weighs 1.023 tons and the supporting structure 175 tons.

The fast-assembly bridge was erected to relieve the overloaded Aegidientorplatz during rush hours by a parallel overpass route in the second plane for the main flow of traffic. Another factor contributing to the bridge design was the subway construction site planned for the Aegidientorplatz region in the near future. The scheduled time of bridge use is at least 10 years. Erection work was carried out on 5 consecutive weekends. Road and tram traffic was interrupted on these weekends only in the actual erection area, whereas in the remaining sections the traffic was maintained. The bridge has been in service successfully for 4½ years. It is used with great frequency by all traffic including trucks and buses, although the at-grade routes are still available. The melted asphalt coat suppresses most of the traffic noises. Although the roadway is elevated to the window level of adjacent flats, there have been no complaints from residents about any noise nuisance.

The last example of this series is the fast-assembly bridge at Duisburg, Marientorplatz. It is 144 m long and is of two-lane design with 8.50 m of useful roadway width. Individual span lengths vary from 21 to 30 m. In its plan view the bridge is straight. The gradient ranges from 0.8 to 5 percent. The one-sided deck crossfall is 1.6 percent. The roadway coating is a 2-cm mastic layer and a 3-cm melted asphalt layer. The steel superstructure weighs 380 tons and the supporting structure 30 tons. The fast-assembly bridge is used as a temporary descent ramp from a permanent elevated roadway system in the second plane to the existing roadway level. The scheduled time of use is 5 years. Afterward the structure will be used at another location. Erection was carried out without any appreciable traffic impediments. Tram traffic below the bridge only had to be interrupted for a few hours on a Saturday night.

## The D Bridge

The name D bridge is derived from the word Dreieck, i.e., triangle. The D bridge is designed for road and tram traffic. It can be of single- or double-story design. Its concept has been based on span lengths up to 95 m having a roadway width of 6 m, with loads based on bridge class 30 of German DIN 1072. The D bridge is a through bridge with trussed main girders. Its principal elements are the main girder triangle formed by two diagonals and the associated chord member and the loose single chord. The roadway main girder is built up from these elements. It may be single-story with a system height of 2.135 m, two-story with a system height of 4.27 m, or three-story with a system height of about 6.5 m. In all types of the D bridge design, one or even two bearing trusses per main girder may be arranged side by side. Additional chord reinforcements permit another increase in the allowable span length.

Except for the bridges equipped with a flat roadway, D bridges are provided with a lower wind bracing, whereas through bridges have an additional upper bracing, if necessary.

Several deck types have been developed for the D bridge: (a) timber decking on longitudinal and cross girders, (b) hollow structural steel plates on cross girders, and (c) flat steel decking without cross girders, of extremely low overall depth, extending from main girder to main girder. For the last type of roadway, special lower wind bracing is unnecessary.

The D bridge can be installed by the usual erection methods of structural engineering. A particularly advantageous and quick method is the launching of the D bridge. The total bridge can be completely assembled on one side of the obstacle and then launched into place. The launching nose required for this procedure can also be assembled from D bridge elements. About 50 D bridges have been placed in service in West Germany since 1960.

## The SKB Bridge

The last example for standard bridge types is the so-called SKB bridge. It is a bridging system for single-track railroad traffic. Because the bridge has a deck cover that can also be used by road vehicles, it can simultaneously serve for road traffic. The SKB bridge is a further development of the SKR bridge, which was designed by Krupp in cooperation with the pre-war Deutsche Reichsbahn. Since May 1945, 39 SKR bridges have been set up in the present territory of the German Federal Railways. The SKB bridge, too, was developed by Krupp jointly with German Federal Railways.

The standard design of the SKB bridge is the through bridge with trussed main girders. Truss panel length is 6 m. The trussed main girders can be single-story with 6-m system height or double-story with about 12-m system height. As a single-span girder design, single-story bridges are used up to a span length of 84 m in railroad traffic (in road traffic up to 120 m), and two-story bridges up to a span length of 120 m in railroad traffic (in road traffic up to 162 m) in accordance with German specifications. The main elements of the roadway main girder are the chords, diagonals, and posts, all available in lightweight or heavy construction.

The roadway of the SKB bridge consists of the cross girders arranged in the panel points at 6-m intervals, the longitudinal girders, and the three cover panels. Single-story SKB bridges are provided with a lower wind bracing; for two-story SKB bridges the wind braces may be arranged in both the lower and upper chords.

So far the SKB bridge is the sole sectional bridge system in which deflection from permanent load, bolt tolerance, and a quarter of the live load can be compensated. SKB bridges, therefore, do not sag; they comply with all conditions for full-speed traffic.

The SKB bridge can be erected both by the cantilever method and by launching. To launch the bridge requires that stationary sets of roller boxes be used. The carrying capacity of the roller sets is 200 to 800 tons. The weight of the cantilever structure is minimized during launching by use of a special launching nose on the launching tip, which also consists of standardized SKB elements.

## CONCLUSION

The three bridge systems described, the fast-assembly bridge of the Rheinhausen type, the D bridge, and the SKB bridge, fulfill all conditions for a system bridge stipulated in the beginning of the paper: They consist of standardized single elements; these elements are interchangeable; and they can be combined in a structure of high variability and versatile use. They can be delivered early, quickly erected, simply dismantled, and easily reused. Because the number of single elements is limited to as few as possible, stockage and storage, too, can be carried out at a small expenditure. The bridge parts can be stored either with the manufacturer or at any of the road-building authorities' storeyards. In addition to the typical buying and selling transactions, leasing transactions are also possible for short-term use of a system bridge.

# part 5

## innovations in pavement and bridge maintenance

# mastic asphalt concrete

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Mastic asphalt concrete was first placed in the United States on a main-line pavement in Pennsylvania in 1972. This paper describes that placement. The equipment used was specially constructed inasmuch as placement of mastic asphalt concrete required a propane-heated screed, chip spreader, and a crimper roll, which were not previously available in the United States. Also, the plant had to be modified to produce the asphalt at 410 to 450 F. Based on the experience gained on this project, advantages and disadvantages of mastic asphalt concrete placing are discussed.

•Mastic asphalt concrete, or Gussasphalt, is a paving material new to the United States. It was placed for the first time in this country on a main-line pavement in Pennsylvania during September 1972. Mastic asphalt concrete is a mixture that can be poured or cast in place and requires no rolling or compaction. The resulting mass is essentially voidless and is considered highly durable. The properties of conventional asphalt pavements are essentially governed by the qualities of the mineral aggregates. This is not true, however, of mastic asphalt concrete in which the mineral aggregates are considered to be of lesser influence. The pourable characteristics of the mastic asphalt concrete are instead controlled by the composition of the fine mineral filler and the properties of the asphalt cement. These are mixed to produce a mastic type of binder that holds the larger aggregates and supports the load imposed by vehicular traffic.

The purposes of the research program in Pennsylvania were to evaluate the mixing and placing characteristics of mastic asphalt concrete, to evaluate the immediate and long-range performance of the paving system, and to determine the cost-performance relationship of this system as compared to PennDOT's conventional, high type of bituminous concrete pavement (ID-2A).

## EQUIPMENT AND MATERIALS

The equipment used to place the mastic asphalt concrete was specially constructed and was not previously available in this country. The entire paving train moves on rails and consists of three major components: a propane-heated screed, followed by a chip spreader, which is trailed by the crimper roll. One German Linhoff Kocher, an agitated, heated transporter, was mounted on an American truck body. Two other American-built transporters were patterned after the German Kocher and were fabricated to allow for a slightly larger hauling capacity.

The mastic asphalt concrete for this project consisted of a Trinidad and conventional asphalt cement blend, gravel fine aggregate, gravel coarse aggregate, and a limestone filler. The asphalt blend consisted of 20 to 25 percent natural Trinidad asphalt blended with a standard AC-20 having a 40 to 50 penetration grade. The fine and coarse aggregates were gravel, and the filler was a graded limestone material. The cover chips, also gravel, were precoated by using 1 percent of the Trinidad-asphalt blend.

## PLANT MODIFICATIONS

The plant had to be modified to produce the mastic asphalt concrete. The addition of a large amount of filler required a means of preheating the material before it was added to the pug mill. A heated screw assembly, added above the plant, was designed to maintain the temperature generally around 325 F. A portable bin, located near the dust collector storage system, was used to store and provide the needed filler. An agitation system was also needed for the asphalt storage tank. This was required to prevent the settlement of the fine material present in the natural Trinidad asphalt. The asphalt storage temperature was maintained between 400 and 405 F by means of circulation.

## PROCEDURE

### Plant

The mastic asphalt was produced at temperatures that ranged from 410 to 450 F. Mixing was accomplished by adding the coarse aggregate, fine aggregate, and filler into the pug mill. All materials were dry-mixed for 10 to 30 seconds, 20 seconds being the most appropriate dry-mixing time. The Trinidad-asphalt blend was then introduced, and wet-mixing was continued for 60 seconds. The initial design required an asphalt content of 9.5 percent, but this was gradually reduced to 8.9 percent. The initial design called for an addition of 25.4 percent filler but was finally adjusted to 23.7 percent.

### Field

The completed mix was dropped directly from the pug mill into the heated trucks and transported to the paving train. The hot mastic asphalt was dumped onto the fine binder in windrows and allowed to flow in front of the heated screed. The entire paving train rode on rails, but wood forms were nailed to the fine binder to the desired width between the metal rails. The hot mastic asphalt concrete was screeded to the desired depth as the paving train moved forward.

The screeding operation was followed by the power-driven chip spreader, which spread the precoated chips at a rate estimated between 10 and 15 pounds per square yard. The final operation of the paving train was a spiked roller or crimping device which indented the cover chips into the hot mix and created a waffle effect on the surface. The spikes provided $\frac{3}{8}$-in. square indentations on $1\frac{1}{4}$-in. centers. The paving train could move at 3 to 9 feet per minute but averaged about 4 to 5 feet.

Joints were formed by saw-cutting the hardened material at the end of each day's production. Joints needed during the daily production were constructed by hand-cutting or other methods inasmuch as the material had not hardened sufficiently to allow saw-cutting. Traffic was restricted for a 24-hour period, and the finished pavement surface was power-broomed prior to its opening to remove the excess chips.

## OBSERVATIONS

Although some difficulties were naturally encountered during the first-time construction of mastic asphalt concrete, these were corrected as the job progressed, and, generally, a fairly good pavement was achieved. The most difficulty centered on the inability of the plant, transport trucks, or both to produce a steady flow of material. This was improved considerably as the job progressed. The project demonstrated that a successful mastic asphalt concrete could be mixed and placed and also provided valuable experience that should contribute to improvement of future projects. Based on the observations of this project, the following advantages and disadvantages are noted.

Advantages

1. The mastic asphalt concrete, with its 0 void content, has the potential for a long service life, free from the usual surface defects such as potholing, raveling, and surface oxidation.
2. The longer service life will reduce maintenance costs and may produce substantial savings despite the greater initial costs.
3. The construction season could be extended by cold-weather paving.
4. The system will produce a durable and highly skid-resistant pavement.
5. Because mastic asphalt concrete is impervious to water, it could provide a waterproof resurfacing system for concrete bridge decks.
6. The longer service life of the system would require fewer resurfacings of a pavement section and thus would reduce the inconvenience to traffic, particularly on high-volume roads.

Disadvantages

1. There are currently no manufacturers of the required paving equipment in this country. This will discourage widespread use of mastic asphalt concrete and keep its cost high until paving equipment can be produced or acquired.
2. Some bitumen plants may not want to produce the material because of the plant modifications required.
3. The present paving equipment must ride on rails over the work area.
4. There are safety hazards associated with placing the extremely hot material. With a temperature in excess of 400 F, severe burns could be caused if workmen were to come in contact with the asphalt.
5. Heated trucks with agitation are necessary for a uniform mastic asphalt.
6. Because the mastic asphalt concrete pavement has to be closed to traffic for 12 hours, this almost limits its use to four-lane pavements where traffic can use the other lanes during construction or to those pavements that can be completely closed. This condition requires more elaborate traffic control than that for placement of conventional asphaltic pavement.

# steel-fiber-reinforced concrete

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Steel-fiber-reinforced concrete was used as a bridge deck overlay in Pennsylvania. This paper discusses the mixing and placing of that overlay on a single-span, continuous-truss bridge. Aside from a delay caused by the necessity for emergency bridge repair elsewhere, problems in placing the overlay included formation of wire balls in the concrete, nonuniformity of concrete loads delivered, and erratic strength test results. It is recommended that, on the steel-fiber-reinforced project to be undertaken by Pennsylvania, control sections be provided to afford more meaningful evaluations of concrete performance.

•Steel-fiber-reinforced concrete is a system wherein steel wires or fibers having a diameter of 0.010 to 0.025 inch and a length of ½ to 2 inches are incorporated into a conventional cement concrete mixture. The system provides many advantages over conventional concrete and promises to provide a solution to concrete pavement and bridge deck overlay problems. Steel-fiber-reinforced concrete has been used on a number of installations throughout the country on airport runways, arterials, and bridge deck surfaces.

It has been reported that the addition of steel fibers to concrete increases the flexural strength from two to three times and more than doubles the compressive strength. The impact resistance is nearly three times higher, the concrete is twice as abrasion-resistant, and spalling resistance is several times greater. The advantages of steel-fiber-reinforced concrete are its (a) much greater resistance to cracking, (b) superior resistance to thermal shock, (c) ability to provide design strength with thinner sections, which results in material savings, (d) elimination of conventional reinforcing with associated labor, and (e) lower maintenance requirement and longer life. The resistance to salt scaling is supposedly equal to that of conventional concrete. Rusting of the steel wire does not present any structural problems, and it should not adversely affect the surface appearance.

This report covers the first of two proposed research projects in Pennsylvania using steel-fiber-reinforced concrete. The purpose of the study is to evaluate the mixing and placing characteristics of the fiber-reinforced concrete and to observe the long-range performance of the system as a bridge deck surface overlay. The structure covered in this report is the bridge on LR-250 in New Cumberland over the Yellow Breeches Creek at the York and Cumberland County line. The other structure, proposed for the 1973 construction season, will be in Allegheny County.

## THE STRUCTURE

The structure on LR-250 was built in 1936 as a single-span, continuous-truss bridge. It is 154 feet long and 44 feet wide including 2-ft right and left drain gutters. The reinforced concrete deck was a two-course single slab member consisting of a 7½-in. bottom course and a 2½-in. wearing course fixed to riveted plate end dams.

The concrete deck surface had previously been covered with a plant-mixed bituminous wearing surface that, in recent years, had required frequent patching. Deterioration of the wearing surface had progressed to the extent that surface cracks and pot holes were prevalent, and the riding surface was significantly impaired.

## CONSTRUCTION DETAILS

Because of the nature of the contract, field inspection was required by district construction forces. It was understood, as a result of the preconstruction meeting, that the project should be governed by the applicable PennDOT specification requirements. The delineation of control in the field however was uncertain as the job progressed and, because of this, control was not so good as it should have been.

The project was designed in close cooperation with personnel from the Battelle Development Corporation and the U.S. Steel Corporation. These personnel were also present during the mixing and placing operations to provide guidance and assistance. The steel fibers for the project were provided, at no cost to the department, jointly by Battelle and U.S. Steel. Mix design data are given in Table 1.

The project was awarded, and preparation work began during the week of August 7, 1972. Preparation consisted of stripping the bituminous material from the concrete deck and removing any weak, unsound, or deteriorated concrete from the deck surface as specified in the contract. It was only during this process that the wearing course on the old deck surface became apparent. Arrangements were made with the contractor to remove the entire concrete wearing surface from the deck. At this time, an emergency arose: Bridge repair work was necessary on two major arteries. The contractor was asked to cease further deck preparation activities and to immediately place the fiber-reinforced concrete overlay. At this time approximately 50 percent of the wearing surface had been removed (Figs. 1 and 2).

On August 25, a 2-in.-thick, single-slab member was placed between the 2-ft gutters over the unstripped area, and an average of 6⅛ inches was placed over the stripped areas because of the removal of deteriorated concrete in the bottom course.

The formation of wire balls in the concrete, which is a constant problem with transit mixers, was minimized by batching the fine and coarse aggregates, then adding the steel fibers, 70 percent of the mixing water, and the cement, and then adding the remaining water. The fibers were delivered in 20-lb boxes and were manually spread and separated on a conveyor belt charging into the truck mixer.

The first transit mixer arrived at the job site at 9:30 a.m. The weather was sunny, the ambient temperature was 72 F, and the relative humidity was 82 percent. A total of 79 cubic yards was delivered to the job site; the hauling distance was 3 miles. With the exception of the first load, the capacity of the transit mixers was restricted to 6 cubic yards.

The prepared surface had been wet sufficiently, and a neat cement slurry was applied immediately before the fiber-reinforced concrete was placed (Figs. 3 and 4). The finishing equipment consisted of a strike-off unit and a trailing bridge unit that supported

Table 1. Concrete mix design.

| Item | Amount |
|---|---|
| Ingredients/yd³ batch | |
| Cement, lb | 752 |
| Water, lb | 368 |
| Air entrainer (6.5 percent), oz | 5 |
| Retarder, oz | 24 |
| Aggregate, lb | |
| Fine[a] | 1,870[b] |
| Coarse[c] | 610[b] |
| Fibers | 200 |
| Aggregate, percentage by volume | |
| Fine | 74.0 |
| Coarse | 23.3 |
| Fibers | 2.7 |
| Fibers/yd³, percent | 1.52 |
| Slump, in. | 3.25 |
| Weight, lb/ft³ | 140.72 |

[a]Fines modulus = 2.90.
[b]Saturated, surface dry.
[c]100 percent passing ½-in. sieve.

Figure 1. Bridge deck with concrete removed.

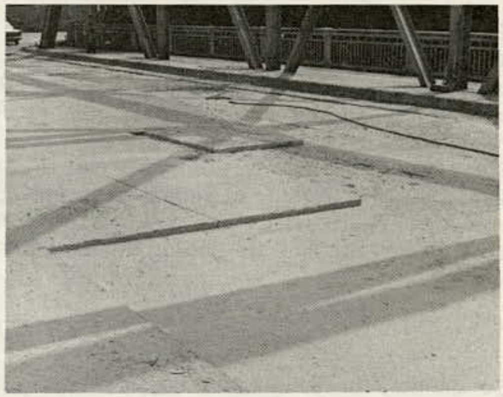


Figure 2. Deck layout.

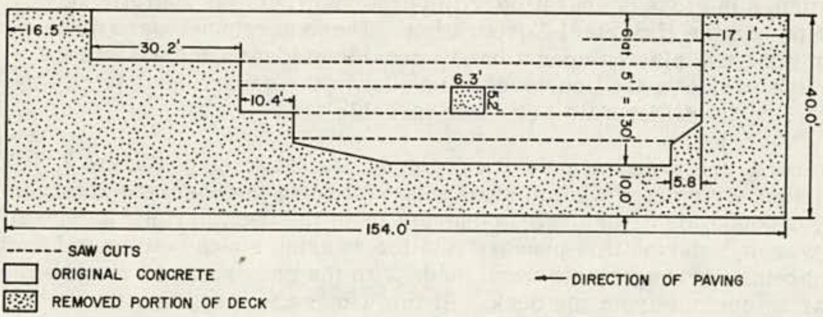



Figure 3. Cement slurry and finishing machine.

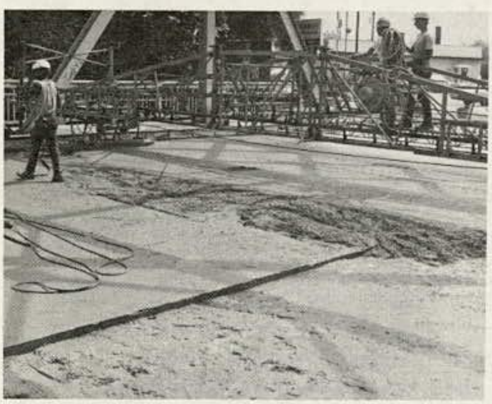


Figure 4. Steel-fiber-reinforced concrete.

Figure 5. Finishing operation.

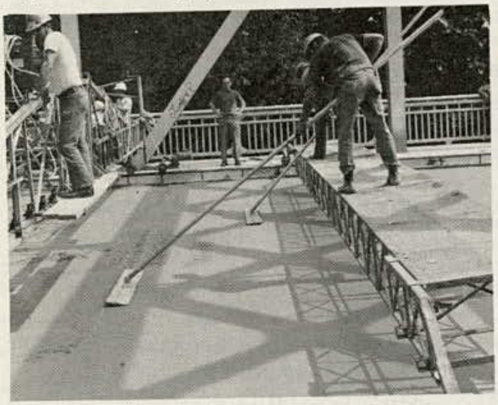


Table 2. Results of slump and air tests.

| Test | Plant High | Plant Low | Plant Avg | Jobsite High | Jobsite Low | Jobsite Avg |
|---|---|---|---|---|---|---|
| Slump, in. | 6½ | 3¼ | 4½ | 6.0 | 3¼ | 4¼ |
| Entrained air, percent | 8.0 | 4.8 | 6.4 | 7.4 | 4.8 | 6.5 |

Table 3. Strength test results.

| Age (days) | Compressive Wet-Cured | Compressive Dry-Cured | Flexural Wet-Cured | Flexural Dry-Cured |
|---|---|---|---|---|
| 3 | | 2,891[a] | | |
| 4 | | 3,970[b] | | |
| 7 | 2,617 | 2,476 | 520 | 879 |
| 8 | | 5,217[b] | | |
| 14 | 4,333 | 3,360 | | |
| 21 | | | 667 | 1,022 |
| 28 | | 4,881 | | 798[b] |

[a]Avg of three tests. [b]Avg of two tests.

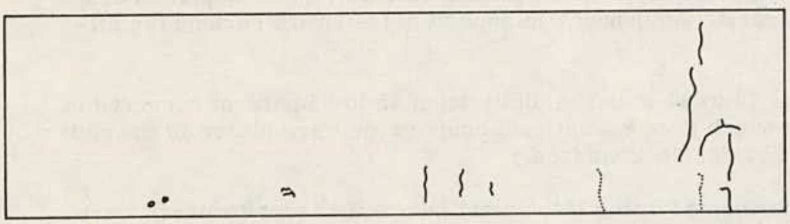
Table 4. Linear traverse data.

| Item | Quantity |
|---|---|
| Specific surface, in.$^2$/in.$^3$ | 0.571 |
| Spacing factor, in. | 0.009 |
| Total traverse length, in. | 80.00 |
| Entrained air[a], percent | 4.65 |
| Entrapped air, percent | 0.55 |
| Total air, percent | 5.20 |
| Avg fiber spacing, in. | 0.88 |

Note: Coarse aggregate was carbonate with narrow gradation, no cracking, and good bond. The cement paste was glassy and had no internal cracking, no fine or coarse aggregate pullout, and good paste quality.

[a]Entrained air considered adequate.

Figure 6. Crack patterns.



............ RECORDED AT 13 DAYS
——— RECORDED AT 38 DAYS
o CAVITIES, 1/2-1 INCH DIAMETER

SCALE: 1 INCH = 20 FEET

the hand finishers (Fig. 5). Both units rode a tube rail located in the gutter area. The mechanical efficiency of the finishing units left something to be desired.

The concrete was discharged directly from the transit mixers onto the existing deck surface ahead of the strike-off unit. Hand finishing was accomplished with aluminum floats and straightedges; a bristled broom was used for the final transverse textured finish. Curing was achieved by hand spraying a white-pigmented liquid curing compound.

Although a design slump of 3¼ inches was specified, the actual slump varied from a low of 3½ to a high of 5¾ inches; the average of 13 truck loads was 4¼ inches.

The first batch of concrete mixed at the plant contained excessive water, which was corrected by the addition of another yard of ingredients. Tests at the jobsite produced a 4-in. slump and an air content of 7 percent, but the consistency during discharge varied considerably. Additional, intermittent charges of water were not uncommon, and water was sprayed on the concrete during finishing because of poor workability.

The third load was excessively wet throughout its discharge. Slump and air tests were omitted. Although the latter portion of the fourth batch was extremely wet and variations in consistency continued from truck to truck and within a single batch, there was an improvement in consistency and desired slumps in the remaining nine truckloads. Steel fiber balls, measuring from 1 to 3 inches in diameter, occurred frequently, but, because the batch plant had a limited supply of steel fibers, it was necessary to use all concrete as delivered. These balls were removed from the concrete wherever they were visible.

The ambient temperature increased steadily during the day, and there were some lengthy delays between truckloads. The midday temperature reached a high of 92 F, the relative humidity was 47 percent, and there was a considerable amount of direct sunlight. At noontime, with less than one-third of the required concrete in place, a 1-hour delay between truckloads occurred. Concrete temperature during the placing period ranged from 85 to 98 degrees.

As concrete placement progressed, the existing surface was rewet, and the neat cement slurry was applied. Frequently, this appeared to be considerably thin-textured. The 684-$yd^2$ area was completed around 7:00 p.m.; the final surface texture appeared to be of fair quality. Additional curing compound was applied to the entire surface the following day.

Ambient temperatures from 73 to 90 F and humidity from 45 to 85 percent occurred on the following 3 days during which time 8 additional cubic yards were placed in the gutter areas, which completed the bridge deck surface.

Slump and air tests were performed at the batch plant before the transit mixers were dispatched, and the same tests were also duplicated at the jobsite on concrete sampled after final mixing. A summary of these tests is given in Table 2.

## TEST RESULTS

One of the anticipated advantages of adding steel fibers to a conventional concrete mixture is the higher flexural and compressive strengths. Although higher-than-usual strength tests were achieved, these were not so great as anticipated. Although the slump and entrained air test results were generally favorable, the erratic strength test results given in Table 3 are disappointing and are perhaps indicative of the nonuniformity of the delivered concrete. Minimum PennDOT strength requirements are as follows:

| Age (days) | Compressive Strength (psi) | Flexural Strength (psi) |
|---|---|---|
| 3 | 3,000 | 550 |
| 28 | 3,750 | 660 |

The primary function of the steel fibers is to curb the development of cracks and to prevent the propagation of any microcracks that occur in the concrete matrix. It is therefore important that the steel fibers be added in sufficient quantity and in a manner designed to provide an average spacing of 0.5 inch or slightly less. A petrographic analysis was performed by PennDOT on samples obtained from a 28-day beam specimen. The fiber spacing and other relevant characteristics of the concrete are given in Table 4.

## COST DATA

The contract was awarded at a bid price of $215.00 per cubic yard for in-place concrete (all preparation work included), which amounted to approximately $25.00 per square yard. Traffic control costs were assumed by the department and were not included in this price. As mentioned before, the steel fibers were donated to the project. The cost of the steel fibers would normally be expected to be about 18 to 25 cents per pound and this, in addition to a batching cost of approximately $3.00 per cubic yard (costs for this project), would run the cost of steel-fiber-reinforced concrete to twice that of conventional concrete. A uniform, 2-in.-thick, steel-fiber-reinforced concrete overlay (8-bag mix) could be placed for about $4.00 a square yard for material costs only.

Although steel-fiber-reinforced concrete will never approach the cost of a conventional mix, the increased use of this product together with research leading toward use of larger sized aggregate and less cement will contribute to lower unit costs.

## RECENT INSPECTIONS

The condition of the project was observed at 7-, 30-, 60-, and 90-day intervals. As of this date, no major defects have developed; however, some cracks are evident as shown in Figure 6. Most of these have developed at areas where the concrete overlay varied abruptly in thickness from 2 to 5 inches or more because of the concrete removal pattern. These cracks measured from 0.01 to 0.03 inch in width and did not appear too severe at this time. The surface finish is generally smooth (little or no texture), and the riding quality is uneven.

## CONCLUSIONS AND RECOMMENDATIONS

It is premature to make predictions on the ultimate performance of the fiber-reinforced concrete, particularly on the basis of this installation. It is certain, however, that the advantages of using steel-fiber-reinforced concrete will be evident in many areas of highway construction. The principal disadvantage is the cost, but this can be justified by reduced maintenance and longer service life.

The project selected for construction next season will be closely monitored to provide additional research data relative to the performance of this system.

Recommendations at this time are limited; however, it would be desirable in future installations to provide control sections using conventional concrete overlay methods or other overlay systems to provide a more meaningful evaluation.

# innovative maintenance procedures in texas

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Texas Maintenance Practices Committee

Innovative maintenance procedures and maintenance materials used by the Texas Highway Department and developed by district maintenance personnel are briefly presented. Procedures discussed involve maintenance methods and equipment associated with pavement and shoulder repair including rapid repair of bridge deck failures, maintenance equipment with hot storage capabilities, and unsurfaced shoulder repair with mechanized operations. The use of a one-man, truck-mounted herbicide sprayer; mechanized mulching and seeding operations for erosion control; mechanized litter gathering equipment; and a culvert cleanout process using a dog are also discussed. Specialized materials used in Texas maintenance operations include lightweight aggregate mixtures for seal coats, overlays, plant-mixed seals, bridge deck overlay, and hot-mixed, cold-laid patching materials. A brief description of these practices together with limited economic data is presented for comparison with other state highway department practices.

• Funding for highway maintenance operations makes up a significant part of the total highway budget. As new roadways are constructed and existing roadways are reconstructed to handle the ever-increasing demands, maintenance expenditures must be increased to maintain these facilities at an acceptable level of service. Furthermore, as the highways in existence become older, their maintenance requirements will increase. Thus comprehensive decision-making systems have been developed and management studies have been performed by a number of state highway departments.

The Texas Highway Department and the Texas Transportation Institute initiated a maintenance study in 1970 to identify and develop maintenance methods, to develop a maintenance priority rating system, and to implement the study on a trial basis. Early work in the study has been aimed at the development of maintenance methods. Results from this phase of the research project indicate that certain innovative maintenance procedures exist in certain highway districts, which could be of benefit to all other districts.

District personnel in charge of maintenance and representatives from the Texas Highway Department Maintenance Operation Division and the Texas Transportation Institute formed five maintenance method panels. These panels are responsible for the development of maintenance methods and quality standards in the following general areas:

1. Base and subgrade repair,
2. Bituminous surfaces, shoulders, and approaches,
3. Portland cement concrete surfaces,

4. Roadside maintenance, and
5. Structures.

Innovative items from these general areas will be discussed in this paper.

It is recognized that the so-called innovative maintenance materials, equipment, and methods described in this paper may be standard practices in other states. However, certain other sections of the U.S. or even Texas maintenance forces are not aware of these ideas as applied to maintenance. The Texas Maintenance Practices Committee strongly encourages the exchange of information like that described in this paper at all levels of responsibility.

Selection of the innovative ideas presented in this paper is based on two primary factors.

1. Does the new practice reduce the total cost of roadway maintenance?
2. Does the new practice increase safety?

Total cost of roadway maintenance includes the initial cost of the material and operation and a monetary measure of how well the operation satisfies the needs of the driving public. Safety of both the driving public and the maintenance forces must be considered when maintenance operations are developed.

Specific maintenance materials, equipment, and methods will be briefly discussed. These discussions will include crew size, equipment requirements, and cost data where available.

## MAINTENANCE PRACTICES

### Pavement Maintenance

Materials—Both bloated (lightweight) and nonbloated synthetic aggregates have been used for seal coats in Texas since 1961. It is estimated that 10,000 lane-miles of this type of highway surface have been placed to date. In addition to the conventional full-lane-width seal coat, strip seals and smaller patches have been placed with synthetic aggregate in a sealing type of operation.

Hot-mixed asphalt concrete containing synthetic aggregate as the coarse fraction has been used since 1963. Field evidence exists that suggests that synthetic aggregate hot-mixed overlays and plant-mixed seals can be considered as premium materials because of their prolonged and initially high skid resistance and stability characteristics.

Recent research conducted in Texas established the methodology for the manufacture and placement of hot-mixed, cold-laid synthetic aggregate asphalt mixtures. Successful field trials have demonstrated that the material is a good winter and summer patching material that provides necessary skid resistance and stability.

Conventional maintenance practices can be used for the placement of all the synthetic aggregate-asphalt mixtures described. Certain precautions, however, should be taken under some conditions and are adequately described in references that can be obtained from the Texas Highway Department or the Texas Transportation Institute.

Edge Repairs—During the initial phases of the farm-to-market road-building program, many miles of 18- to 20-ft-wide pavement were constructed without surfaced shoulders. These roadways are now subject to much heavier loads and a higher traffic volume than originally expected. Also, farmers driving farm equipment often use these roadways and, in attempts to allow automobiles and trucks to pass, pull off to the edge. Thus raveled pavement edges and pavement drop-off often result. Considerable difficulty has been experienced in maintaining the edges.

The cost of repairing pavement edges by the hand method involving hot-mixed, cold-laid bituminous patching materials averaged 18 to 24 cents per linear foot. A new procedure was developed that reduced the cost to 4.5 to 6.5 cents per linear foot. These costs include labor, equipment, and material.

The innovative maintenance method proceeds in the following way: The pavement edge is cleaned, and low areas are replaced with flexible base; RC-250 tack coat is applied at a rate of 0.07 gallon per square yard. A string line or pilot line is used to produce a straight edge. A slide box is used to spread the bituminous-treated mixture in the designated area. The slide box is attached to a dump truck by a hopper through which material is introduced (Fig. 1). A workman on the ground controls the flow of material into the slide box with a gate device while another workman breaks up lumps and clods that the raker teeth in the slide box do not break. The workmen are in a protected area beside the truck and somewhat away from traffic. Next a brooming operation is used to correct an over-width application of the bituminous-treated material applied outside of the area that has been tacked. The rolling operation is performed with a steel drum roller (Fig. 2).

Materials other than hot-mixed, cold-laid bituminous mixtures can be used in this operation; however, little bleeding has been noticed with this material and the heavy tack coat applied to the pavement edge.

Repair of pavement drop-off is another troublesome maintenance activity in which a more mechanized approach has been developed. A Hebbronville, Texas, maintenance construction foreman used a "hopper spout and strike-off box" assembly attached to a dump truck to perform in 3 hours with a seven-man crew a job that would normally take a 17-man crew 9 hours to perform. This innovative edge repair method using crushed base material prepared at a prescribed moisture content consists of two maintenance men, four truck drivers, and one loader operator.

The strike-off box is made primarily from old grader blades and contains an adjustable gate. Workmen are able to work in a relatively protected area, and compaction can be accomplished by using the loaded trucks or suitable compaction equipment.

The specialized equipment required for this operation can be built at a cost of $200 and requires about ½ hour to install. Use of this equipment effects a significant cost saving.

Pot Hole Repair—Repair of pot holes and other forms of localized distress can most successfully be accomplished with hot-mixed asphalt concrete materials. Use of these materials is limited by their availability and the requirement that they be kept at an elevated temperature for a prolonged period of time. To make use of hot-mixed asphalt concrete mixtures, some of the highway districts in the large metropolitan areas of Texas are making use of a "hot box." Automatically controlled burners keep the material at the desired temperature, if necessary overnight. Normal patching crews place the material at a cost of about $50 per ton.

The use of the "hot box" in cold or wet weather is especially beneficial, for the truck-mounted device can heat or dry the repair area prior to patching. Its use will increase in Texas.

## Roadside Maintenance

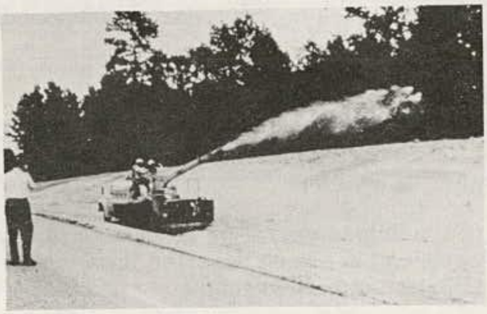
Roadside maintenance operations include truck-mounted spraying, mulching operation, litter removal, and culvert cleanout.

Figure 1. Pavement edge repair.

Figure 2. Steel drum roller used to repair pavement edge.

Figure 3. Herbicide sprayer.

Figure 4. Mulching operation.



Herbicide Spraying—A truck-mounted herbicide spray rig with three spray bar attachments has been used effectively to spray 30 to 50 miles of farm-to-market roadways in an 8-hour day (Fig. 3). One operator is required to perform all duties and, once the operator becomes experienced, this device will have a greater production than a two-man crew using a trailer-mounted unit. Thus, use of this spray rig can reduce costs by 25 to 50 percent.

When spraying around expressway rails and delineator posts, the truck normally travels at 2 to 3 mph. The operator can spray the edges of paved shoulders, flip a master switch, and spray delineators as he passes by. If a hard-to-reach area requires spraying, the operator can reach the area with a hand boom from the truck cab. The boom is equipped with a squirt nozzle. This same boom has been used to spray riprap, culverts, drain ditches, and the like.

Ant beds may be treated by changing to a boom designed for that purpose and by applying an appropriate chemical.

Attachments that have been used successfully with this piece of equipment include curb spraying attachment, pavement edge spraying attachment, delineator spraying attachment, culvert spraying hose, bridge abutment spray hose, right-of-way post spraying hose, and shoulder spraying bar.

Because the operator is away from the spray bars and is not likely to come in contact with the spray materials, operator safety is improved. An amber revolving light and fluorescent flags mounted on high-level stands provide protection for the truck and operator.

Basic equipment on this 1- to 1½-ton stake-body truck includes a 550-gal fiberglass tank, pumps, pressure regulator, tachometer, strainers and filters, and electric control panel. The spray truck should be washed with water daily and with detergent twice weekly as part of a preventive maintenance program. Corrosion-inhibiting agents added to the spray are helpful in reducing equipment maintenance.

Seeding and Mulching Operation—A mechanized seeding and mulching operation that requires an eight- to 10-man crew has been used in Texas (Fig. 4). The area to be planted is cultivated to the desired depth (a recommended minimum of 4 inches), and a mixture of perennial grass seeds and fertilizer is applied with a truck-mounted hydroseeder or other device that distributes the seed and fertilizer in water through a pressure distribution system. The amount of fertilizer (16-8-8 or 16-20-0) depends on local conditions; however, quantities on the order of 400 pounds per acre are common.

The mulch consists of a mixture of hay and emulsified asphalt distributed pneumatically on the seeded area at a rate of approximately 2 tons per acre. Dry hay is introduced into the trailer-mounted mulching machine whereupon it is shredded. An emulsified asphalt is then added to the shredded hay at a rate of approximately 0.05 gallon per square yard to tack the hay to the slope.

The combined seeding and mulching operation is capable of covering 10 acres per day. A limited number of successes have been reported in which the seed, fertilizer, hay, and emulsion have been applied simultaneously.

Litter Removal—Comparative costs of litter removal using mechanized equipment versus hand methods have been obtained in Texas. Two mechanized devices, the "can gobbler" (Fig. 5) and "litter gitter" (Fig. 6), have been employed. Mechanized devices are capable of removing 6 cubic yards of litter during 8 hours of operation at a cost of $12 to $14 per cubic yard. One maintenance man is required for the operation of the mechanized equipment and an additional crew member may be required for traffic control, removal of litter from the mechanized device, and transport to disposal area.


Figure 5. Can gobbler.


Figure 6. Litter gitter.


Figure 7. Fido and his rope trick.

In general, the roadside is not cleaned so well by mechanized devices as by hand picking; however, an acceptable appearance is obtained. Often dead grass and loose rock may be picked up by these devices. They greatly reduce the capacity of the machines and, in the case of one of the machines, create a problem in the litter sacks that must be periodically removed and dumped.

Hand removal of litter with 1-, 2-, and 3-man crews costs \$11, \$18, and \$22 per cubic yard respectively. Production per day for these crews is on the order of 4 to 5 cubic yards of litter depending on the density of the litter. Thus it would appear that, based on data collected in several trial areas of the state, a two-fold savings can be realized under certain situations, depending on the density of the litter.

Culvert Cleanout—Removal of silt and other materials from culverts is a major problem in many areas of Texas. The usual culvert cleaning operation requires stringing cables through these drainage channels and use of scoops or augers to remove the silt. Stringing of cables through these culverts is a major problem. Augering tools were tried with limited success; flooding techniques using blocks of wood to carry the cable were tried but required as many as 3 days to string a cable in a single culvert. Two workers placed only a few hundred feet of cable a day before they employed Fido, a dog belonging to one of the workers. Fido carries the cable through the cramped culverts (Fig. 7) and has increased production to as much as 7,800 feet per day. These cables are now routinely left in the culverts; however, occasionally some are stolen or washed away, and Fido is called on to perform his duties again.

Only on two occasions did Fido refuse to go through a culvert. It is suspected that rattlesnakes take shelter in the cool shady culverts during the summer months.

## Maintenance of Structures

Bridge Deck Repair—On several occasions, parts of bridge decks have completely collapsed. These decks must be quickly repaired. A method using fast-setting cements has been developed; traffic can use the bridge in the afternoon following repair in the morning.

The method of repair includes the removal of damaged concrete if it is determined that breakout extends back to sound concrete in all directions. Pneumatic hammers can be used for this operation, and we sandblast to clean reinforcing steel. If it is determined that additional support under the bridge deck is desirable, short sections of 12-in. I-beams can be erected horizontally between beams or girders to support 8-in. longitudinal I-beams, on which ⅛-in. metal plates for bottom deck forms can be supported. If additional support under the bridge decks is not deemed necessary, a metal or wood bottom deck form can be secured to the deck reinforcing bars.

The concrete breakout area should be painted with an epoxy adhesive, and a rapid-setting special cement should be placed. This cement will set within a few hours. Approximately 1 cubic yard of concrete can be placed per day in 56 man-hours of labor.

For repair of smaller areas, certain epoxy materials have been used with some degree of success. This is a rapid repair type of operation.

Bridge Deck Overlay—Asphalt concrete overlays on bridge decks have been utilized in the Dallas-Fort Worth area for a number of years. This overlay system consists of the application of aggregate seal coat to the portland cement concrete bridge deck after areas of destruction have been repaired. The seal coat asphalt may or may not contain a natural rubber additive. The seal coat application is followed by about 2 inches of asphalt concrete overlay. This overlay material contains lightweight aggregate as its coarse fraction and may or may not contain asbestos fibers and rubber as an additive. These overlays have performed exceptionally well since installation in 1968.

## CONCLUSIONS

Innovative maintenance materials, equipment, and methods have been briefly described. Detailed information on this operation can be obtained from the Texas Highway Department, Maintenance Operations Division, File D-18, Austin 78701. It is realized that improvement in the methods described in all probability is practiced in several states, and it is hoped that this paper will stimulate an interchange of ideas in all areas of highway maintenance.

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The organizational units and the chairmen and members are as of December 31, 1973.

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