# HIGHWAY RESEARCH RECORD

**Number 327**

## Pavement Rehabilitation and Design of Overlays

8 Reports

Subject Areas

25 Pavement Design
34 General Materials
40 Maintenance, General

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# Foreword

The information published in this RECORD illustrates current good maintenance practice for the rehabilitation of bridges and pavements. Consequently, it should be particularly useful to researchers who require information for evaluating pavement condition, selecting an appropriate repair method, programming rehabilitation work, and developing construction processes and control methods for such work. Maintenance supervisors will also find useful ideas to aid them in their operations. The papers range in subject from the use of epoxy and polyester resins in repairing concrete in India to the use of liquid asphalt-stone chips sealcoat on the high-volume Pennsylvania Turnpike.

Numerous studies have shown that linseed oil-mineral spirit liquid treatments of concrete may be useful in reducing the incidence of concrete surface scaling, but Runkle points out such treatments may occasionally result in a reduction in the skid resistance of wet pavements. For that reason maintenance forces should be aware of the danger and be prepared to offset the unsafe condition either by closing the road until a safe condition is reached or by applying a skid resistant treatment. A discussor of the Runkle pavement suggested that sand applied to the freshly applied oil mixture is one promising treatment.

Pavement rehabilitation on bituminous pavement is commonly required at 5- to 10-year intervals starting as early as 5 years after the pavement is constructed. The criteria used to initiate rehabilitation treatments should be of some concern to administrators because studies have shown that the criteria used are not always valid. It is interesting to note that the Virginia State Highway Department has concerned itself with this problem and developed rules for guiding its decisions. Although these rules do not assist a supervisor to balance the need to correct road roughness or an unsightly appearance against skid resistance, Cecchini does list the priorities followed by his organization. Restoration of nonskid characteristics to road surfaces that have a predicted skid-resistance coefficient reading of less than 0.40 is given first priority. Second priority is given to resurfacing road surfaces at accident-prone locations at which the road surface is suspect, and third priority is given to other locations. It is assumed that the correction of extremely rough surfaces is not found to be a major problem in Virginia and, for that reason, their correction falls in the third priority group. Supervisors in other states might consciously or unconsciously assign a higher priority to correcting very rough pavement surfaces or accident-prone locations.

Commonly, the selection of rehabilitation treatments is made by responsible engineers on the basis of individual judgment. This results in wide variations, sometimes due to differences in the underlying pavement and other environmental conditions but also due to differences of opinion among engineers. These differences may be illustrated by the variation in practices described in this RECORD. Rohde states the New Jersey Turnpike authority has found no overlay less than $1\frac{1}{2}$-in. thick that will stand up under the heavy traffic experiences on that corridor route; yet

Klucher of the Pennsylvania Turnpike Commission reports favorable experience with light surface treatments on that heavily traveled, aging road. One can only wonder whether this represents a difference in level of service rendered to the public, or is related to the fact that portland cement concrete pavement was used initially for a pavement surface on the Pennsylvania Turnpike but asphalt concrete pavement was used for the New Jersey Turnpike, or results from some quite different reason because common bases for making the decision are not described in the 2 papers.

Two papers are concerned with heavy roller breakage of portland cement concrete pavement slabs prior to overlayment with an asphalt concrete overlay. Both Korfhage of Minnesota and Lyons of Louisiana reported favorable results from the use of roller breakage. Lyons further stated that similar good results had not been found in other tests of the process on other construction sites. Apparently a wet subgrade is required in Louisiana before the procedure is likely to ensure good results.

McCullough and Monismith outline a process by which quantitative data and rational procedures may be integrated with engineering judgment to produce an improved pavement overlay design procedure. Wide application and adoption of the proposed system in its entirety will be impeded by the difficulty maintenance supervisors will have obtaining some of the input data. Nevertheless, use of the system offers a way of ensuring that all available information is brought together during the planning process.

Ghosh reports on the materials and mixtures of epoxy and polyester resins suitable for repairing concrete roadways in tropical countries such as India.

# Contents

# Skid Resistance of Linseed Oil Treated Pavements

STEPHEN N. RUNKLE, Virginia Department of Highways

The purpose of this study was to determine what loss would incur in both wet and dry skid resistance of a portland cement concrete pavement after a linseed oil treatment, and how long it would take for the skid resistance to return to its pretreatment level. Work was limited to the testing of 3 areas on Virginia's Interstate Highway System that were treated for normal maintenance reasons. The 3 areas treated yielded 6 different test sites that were tested with the skid trailer and skid test car of the Virginia Highway Research Council. It was found that the loss and recovery of wet skid resistance varied greatly depending principally on the surface texture and cleanliness, application amount, and the surface temperature at the time of the application. Under the worst condition the skid resistance dropped to a dangerous level (skid number below 40) and remained there for 2 days. Recovery of wet skid resistance was not complete until 7 or 8 days after the treatment. The dry pavement skid resistance recovered almost completely in 4 to 6 hours after the treatment.

• DURING 1967 three sections of Virginia's Interstate Highway System were treated with linseed oil antispalling compound in an attempt to increase the scale resistance of the portland cement concrete pavement. One concern was what loss would occur in wet and dry skid resistance and what length of time would be required for the skid resistance to return to its untreated level, or at least to a safe level. Available information indicated the recovery time to be 24 hours for wet skid resistance and less time for dry skid resistance, but additional research was needed to better establish the loss and recovery of skid resistance under various conditions.

Work was limited to the testing of surfaces that were treated for normal maintenance reasons. Three areas, which yielded 6 different test sites, were tested with the skid trailer of the Virginia Highway Research Council and 2 were tested with the stopping distance method by using a car.

## TEST SITES, APPLICATION DATA, AND TEST EQUIPMENT

### Test Sites

Table 1 gives a description of the test sites and data on the application conditions. The pavement at each site has a burlap drag finish and a silica sand fine aggregate. The coarse aggregate is rounded quartz gravel for Sites 1 and 2, and granite for the remaining sites.

The surface condition rating refers primarily to the texture and cleanliness of the surface prior to the linseed oil treatment. These characteristics of the surface would be the ones most likely to affect the skid resistance inasmuch as the materials are about the same for all sites. A poor rating indicates a worn, smooth surface with a heavy layer of road film, and a good rating indicates the opposite, i.e., a well-textured and

Paper sponsored by Committee on Maintenance of Portland Cement Concrete Pavements and presented at the 49th Annual Meeting.

TABLE 1

APPLICATION CONDITIONS

| Site | Location | Length | AVD[a] | Age (yr) | Lanes | Surface Condition Rating | Application No. | Gal/ sq yd | Air Temperature | Untreated Surface Temperature | Observed Absorption Rate |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | I-64 Richmond | 1,500 ft | 875 | 1 | Passing | Excellent | 3 | 0.015 | 75-85 | 95-105 | Excellent |
| 2 | I-64 Richmond | 1,500 ft | 875 | 1 | Center | Excellent | 3 | 0.015 | 75-85 | 95-105 | Excellent |
| 3 | I-95 Emporia | 7 mi | 1,000 | 7 | Passing | Good | 1 | 0.021 | 70-75 | 70-115 | Good |
| 3 | I-95 Emporia | 7 mi | 1,000 | 7 | Passing | Fair | 2 | 0.013 | 70-80 | 90-115 | Good |
| 4 | I-95 Emporia | 7 mi | 3,000 | 7 | Traffic | Fair | 1 | 0.024 | 70-90 | 90-115 | Fair |
| 4 | I-95 Emporia | 7 mi | 3,000 | 7 | Traffic | Poor | 2 | 0.014 | 75-90 | 90-120 | Poor |
| 5 | I-95 Ashland | 10 mi | 1,500 | 7 | Passing | Good | 1 | 0.028 | 70-80 | 75-95 | Fair |
| 5 | I-95 Ashland | 10 mi | 1,500 | 7 | Passing | Fair | 2 | 0.019 | 75-85 | 95-105 | Fair |
| 6 | I-95 Ashland | 5 mi | 6,750 | 7 | Center | Poor | 1 | 0.025 | 65-75 | 80-90 | Poor |

[a]Obtained by applying an estimated percentage for traffic in one direction and one lane to the total AVD determined by the Virginia Department of Highways.

clean surface. As would be expected, because the pavement was only a year old, the surface conditions at Sites 1 and 2 were excellent. At Emporia, on Sites 3 and 4, the surface condition was good in the passing lanes and fair in the traffic lanes. The surface in the traffic lanes at Site 4 had a slightly heavier road film and also was worn smoother. The rating prior to the second treatment was poorer because of the film of oil left from the first treatment. Site 5 was similar to Sites 3 and 4, although the surface at Site 5 showed more scaling and deterioration. Site 6 had a heavier layer of road film than any of the other sites, and for this reason was given only one application of linseed oil.

## Application Data

Table 1 also gives application data for all sites. In all cases the material, which was a 50 percent linseed oil and 50 percent kerosene mixture, was applied with a standard, commercially available liquid fertilizer distributor. Traffic was allowed on the treated sections 4 to 8 hours after the application.

There were several different delay times between applications for the 6 sites. For Sites 1 and 2 there had been 2 treatments totaling 0.040 gal/sq yd 15 months before the third treatment. For Site 3 the first and second treatments were applied on the same day with only a 2- to 4-hour delay between treatments. For Site 4 there was a 3-day delay between the first and second treatment for three-fourths of the application area and a 2-hour delay for the remaining one-fourth of the area. Rain fell hard for approximately 2 hours soon after the first treatment on the part that had a 3-day delay. Site 5 had a 24-hour delay time between the first and second treatments.

The temperatures given are self-explanatory. The temperature of the treated surface usually ranged from 8 to 10 deg above that of the untreated surface. The high surface temperatures at Sites 3 and 4 caused the material to lose its volatiles sooner than desirable, and thus "set up" on the surface. This was particularly true for the second application at Site 4.

The observed absorption after each treatment refers to the times required for most of the oil to be absorbed into the surface, with 4 to 6 hours being termed good; 6 to 10 fair; and 10 or more poor. It can be seen that the absorption is clearly dependent on the surface condition, as would be expected. On the surfaces that were relatively worn and had a heavy road film, the absorption was poor and the oil tended to "set up" on the surface. As stated earlier, this was especially true where there was a high surface temperature during and after the treatment.

## Testing Equipment

The skid test car was a 1964 Plymouth with ASTM E-17 test tires. In testing, the car was brought to the test speed of 20 mph, the gears disengaged, the brakes locked, and the distance to stop recorded on a Wagner stop meter. The skid number was computed by using the equation SN = V2/30S, where SN = skid number, V = velocity in mph, and S = stopping distance in ft.

The skid test trailer was a locked-wheel drag device that measured force on the brake anchor pin. It, too, was equipped with the standard E-17 test tire. Testing was done at a constant speed of 40 mph, by utilizing the trailer's self-watering system, and by locking the left wheel only.

Although all of the wet skid resistance data were collected with the trailer, the most important information was the skid numbers obtained with stopping distance automobile tests. Therefore, the latest correlation between Virginia's skid trailer and stopping distance car was used to convert the trailer data to predicted car values. This correlation included sets of 5 tests at 40 mph for each vehicle on 38 separate sites of various surface types, including portland cement concrete. The correlation resulted in the equation $y = 0.58x + 13.23$, where y = car skid number and x = trailer skid number, with a standard error of estimate of 3.52. The relationship is shown in graphic form in Figure 1.

Also shown in Figure 1 are the good, fair, and poor skid resistance ranges as they have been delineated in Virginia for the past several years (1). Between 40 and 50 is considered fair, above 50 good, and below 40 poor. Reference will be made to these classifications later in this paper.

Substantiation of the correlation was obtained at Sites 3 and 4, where car stopping distance tests were run during the rain on the untreated pavement in the passing lanes and treated pavement in the traffic lanes. Trailer tests run subsequently indicated car values of 50 on the untreated area and 39 on the treated area. The actual values obtained with the car were 50 and 40 respectively.

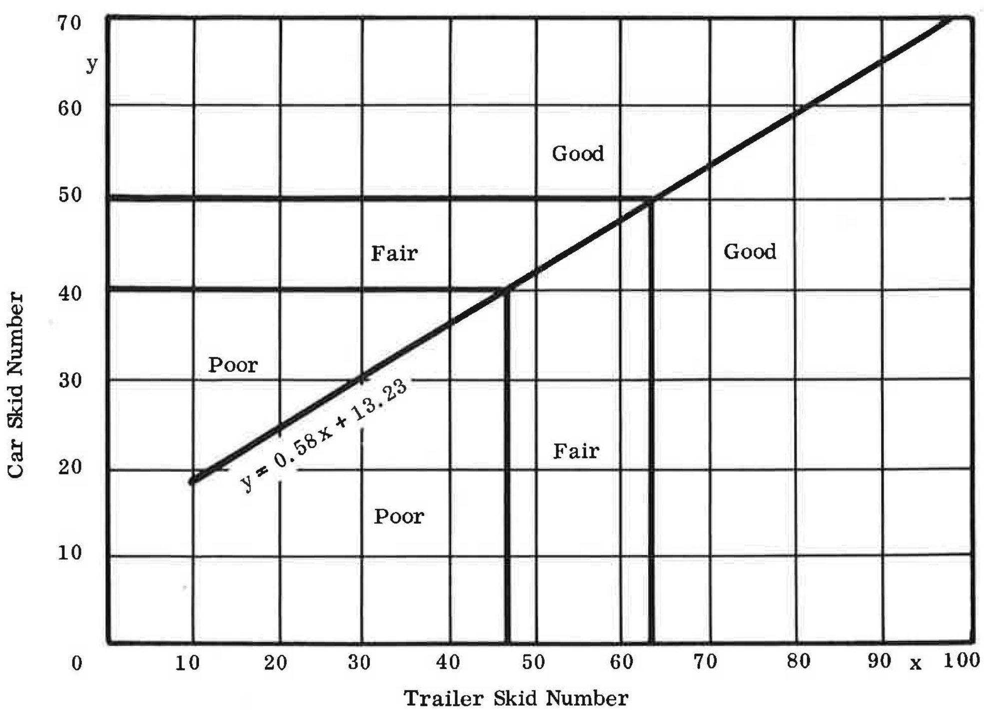


Figure 1. Car-trailer correlation and skid resistance classification ranges.

TABLE 2

CONTROL TESTS—PREDICTED CAR SKID NUMBER

| Site | Days After Treatment | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | Initial | 1 | 2 | 3 | 4 | 6 | 7 | 8 | 9 | 24 | 29 |
| 1 | 55 | 55 | — | — | — | 57 | 56 | 54 | — | — | 63 |
| 2 | 55 | 55 | — | — | — | 57 | 56 | 54 | — | — | 63 |
| 3 | 63 | — | — | 62 | 61 | — | — | — | — | — | — |
| 4 | 62 | — | — | 61 | — | — | — | — | — | — | — |
| 5 | 54 | 55 | 55 | — | — | — | 57 | 54 | 54 | — | 63 |
| 6 | 57 | 56 | 54 | — | — | — | — | — | — | 63 | — |

# TEST RESULTS

## Control Tests

To ensure that changes in skid resistance as determined by the skid trailer were real and not due to changes in the testing equipment, control tests were run on untreated control sections during the times the treatments were made. Five or more skid tests were run for each control sample. The results of these tests are given in Table 2.

The high readings obtained 24 and 29 days after the treatment were probably due to a heavy rain that occurred just prior to these tests and helped to clean the road surface. Prior to this rain, there had been a dry period of several weeks.

Based on the consistency of the control tests and normal testing variability experienced, it was decided that a change of 3 predicted car skid numbers could be considered significant in a minimum sample size of 5 tests. This rule could be used with at least 95 percent confidence.

Statistical significance tests were not used because it was decided that trends and significant changes could be determined without the use of strict statistical methods.

## Dry Pavement Test Results

A limited number of skid tests on dry pavement were performed at Site 4 with the stopping distance car. Tests were run after the first and second applications with the results shown in Table 3.

Not enough data were collected to allow comparisons between the first and second treatment with regard to skid resistance, but it was evident that a significant decrease did occur after each treatment and that recovery was fairly rapid. Although recovery was not complete 6 hours after the treatment, it was 92 percent complete after only 2 to 3 hours for both treatments. These findings agree with those of Kubie, Gast, and Cowan (2).

The maximum and minimum values are also given in Table 3. Notice that the variability increased for a period after a treatment and then began to decrease with elapsed time. This same pattern was observed in the trailer data and was a result of the variation in the application and absorption conditions.

TABLE 3

STOPPING DISTANCE TEST RESULTS ON DRY PAVEMENT AT SITE 4

| Treatment No. | Time After Application (hr) | $\overline{X}$ | N | Minimum-Maximum | Range |
|---|---|---|---|---|---|
| 1 | Initial | 65 | 5 | 63-67 | 4 |
| | 0-1 | 43 | 41 | 25-61 | 36 |
| | 2-3 | 58 | 5 | 51-61 | 10 |
| 2 | 1-2 | 51 | 43 | 35-63 | 28 |
| | 2-3 | 60 | 10 | 55-70 | 15 |
| | 5-6 | 61 | 7 | 58-63 | 5 |

## Wet Pavement Test Results

Table 4 gives the trailer data and predicted car skid values for Sites 1 and 2. The skid resistance dropped at both sites, and no recovery was evident 8 to 10 hours after the treatment. However, for both conditions the decrease was not large enough to cause much concern because the skid resistance remained at a safe level. Only twice, both times in the center lanes, did the skid resistance drop from the good to fair classification. Recovery was considered complete in 7 days.

TABLE 4

TRAILER SKID VALUES AND PREDICTED CAR SKID VALUES AT SITES 1 AND 2

| Time After Treatment | Site 1 $\bar{X}$ | Site 1 N | Site 1 Minimum-Maximum | Site 1 Predicted Car | Site 2 $\bar{X}$ | Site 2 N | Site 2 Minimum-Maximum | Site 2 Predicted Car |
|---|---|---|---|---|---|---|---|---|
| Initial | 74 | 18 | 69-77 | 56 | 72 | 19 | 68-75 | 55 |
| 2-3 hr | 68 | 11 | 64-77 | 53 | 63 | 14 | 52-66 | 50 |
| 3-4 hr | 66 | 5 | 63-69 | 51 | | | | |
| 4-5 hr | 68 | 5 | 64-74 | 53 | 57 | 5 | 55-63 | 46 |
| 7-8 hr | 66 | 5 | 63-69 | 51 | 63 | 5 | 58-71 | 50 |
| 8-9 hr | | | | | 60 | 5 | 55-68 | 48 |
| 9-10 hr | 67 | 4 | 64-68 | 52 | | | | |
| 7 days | 73 | 10 | 69-77 | 55 | 68 | 10 | 63-70 | 53 |

Data for Sites 3 and 4 are given in Table 5. There was no decrease at Site 3 after the first application and only a slight decrease 2 or 3 hours after the second application. The predicted car values remained in the fair range (40-50) for all the tests.

In the traffic lanes, Site 4, the situation was much worse even though the pretreatment skid values were the same as those in the passing lanes, Site 3. The predicted car value dropped to the poor range from 2 to at least 5 hours after the treatment; but after 1 day, recovery appeared to be almost complete. The rain that occurred shortly after the treatment may have speeded the recovery by washing some oil from the surface. No tests were run in the interval from 5 hours to 1 day after the treatment. After the second treatment, the predicted car value immediately dropped to the poor range and remained at that level through 1 day. Even 4 days after the treatment, the predicted car value was substantially below the initial value and at the border line between fair and poor.

The greater decrease in skid resistance after the second treatment in both cases clearly illustrated that the quantity of oil applied was a significant factor in the skid resistance. However, as the difference in behavior between the traffic and passing lanes indicated, the amount of oil applied was not the only factor that influenced the skid resistance after the treatment.

There were 3 factors, all given in Table 1, that probably caused the skid resistance to decrease more in the traffic lanes than in the passing lanes. First, the surface condition was better in the passing lanes, particularly with regard to the amount of road film present. Second, the air and surface temperatures were generally higher during the treatments to the traffic lanes, especially the second treatment. Third, the amount applied in the traffic lanes was slightly higher than that applied in the passing lanes for both the first and second treatments. In the opinion of the author, the first reason given is the most important with regard to the different decreases in skid resistance.

As indicated earlier, there was a 3-day delay between the first and second treatments for three-fourths of Site 4, and during this time traffic was allowed on the treated pavement; whereas there was only a 2-hour delay for the remaining one-fourth of the area, and no traffic was allowed on the pavement during this time. An analysis was made to determine what effect the difference in elapsed time might have had on skid resistance after the second treatment. The 2-hour delay results are also given in Table 5. In comparing these results with those of the 3-day delay, it can be seen that there was very little difference in the decrease in skid resistance after either treatment. This was true even though there was a substantial difference between the skid resistance values just prior to the second treatment. For the area that had a 3-day delay between treatments, the skid resistance had returned to somewhere between the pretreatment predicted car level of 50 and the 1-day level of 47 prior to the second treatment, while the area with the 2-hour delay had a predicted car skid number of only 43 prior to the second treatment.

Based on these data, it was concluded that for up to 3 days the delay time between treatments had little effect on the skid resistance after the second treatment.

TABLE 5

TRAILER SKID VALUES AND PREDICTED CAR SKID VALUES AT SITES 3, 4, 5, AND 6

| Site | Time After Treatment | Application 1 | | | | Application 2 | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | $\overline{X}$ | N | Minimum-Maximum | Predicted Car | $\overline{X}$ | N | Minimum-Maximum | Predicted Car |
| 3 | Initial | 62 | 48 | 57-64 | 49 | 62 | | | 49 |
| | 0-1 hr | | | | | 58 | 73 | 40-68 | 47 |
| | 1-2 hr | 62 | 22 | 60-63 | 49 | 58 | 63 | 45-66 | 47 |
| | 2-3 hr | 63 | 9 | 57-69 | 50 | 52 | 28 | 43-60 | 43 |
| | 3-4 hr | 62 | 6 | 57-63 | 59 | | | | |
| | 4-5 hr | | | | | | | | |
| | 7-9 hr | | | | | | | | |
| | 11-12 hr | | | | | | | | |
| | 3 days | | | | | 58 | 32 | 48-63 | 47 |
| 4 (3-day delay) | Initial | 63 | 32 | 54-71 | 50 | 59 | 63 | | 47-50 |
| | 0-1 hr | 52 | 47 | 34-60 | 43 | 48 | 32 | 40-54 | 41 |
| | 1-2 hr | 50 | 30 | 37-68 | 42 | 44 | 52 | 28-57 | 39 |
| | 2-3 hr | 44 | 12 | 39-52 | 39 | 39 | 26 | 22-51 | 36 |
| | 3-4 hr | 46 | 8 | 39-51 | 40 | | | | |
| | 4-5 hr | 40 | 12 | 25-51 | 36 | 40 | 16 | 34-48 | 36 |
| | 7-9 hr | | | | | 46 | 46 | 29-57 | 40 |
| | 10-12 hr | | | | | 42 | 47 | 32-52 | 38 |
| | 1 day | 59 | 94 | 51-68 | 47 | 42 | 208 | 25-57 | 38 |
| | 4 days | | | | | 50 | 62 | 34-60 | 42 |
| 4 (2-hr delay) | Initial | 66 | 8 | 57-71 | 51 | 52 | | | 43 |
| | 0-1 hr | 50 | 20 | 40-60 | 42 | | | | |
| | 1-2 hr | 52 | 18 | 40-68 | 43 | 41 | 28 | 28-54 | 37 |
| | 7-8 hr | | | | | 41 | 14 | 29-52 | 37 |
| | 1 day | | | | | 41 | 49 | 28-54 | 37 |
| | 4 days | | | | | 48 | 14 | 34-54 | 41 |
| 5 | Initial | 70 | 95 | 63-78 | 54 | 65-70 | | 41-69 | 51-54 |
| | 0-1 hr | 61 | 72 | 52-71 | 49 | 58 | 60 | 47-69 | 47 |
| | 1-2 hr | 65 | 68 | 58-71 | 51 | 60 | 40 | 44-74 | 48 |
| | 2-3 hr | 65 | 52 | 55-71 | 51 | 62 | 40 | 41-74 | 49 |
| | 3-4 hr | 65 | 28 | 44-71 | 51 | 61 | 35 | 47-74 | 49 |
| | 4-5 hr | 62 | 31 | 44-71 | 49 | 62 | 40 | 39-69 | 49 |
| | 5-6 hr | 60 | 12 | 52-66 | 48 | 54 | 20 | 44-69 | 44 |
| | 6-7 hr | 62 | 14 | 36-69 | 49 | 59 | 20 | 47-69 | 47 |
| | 7-8 hr | 63 | 6 | 60-66 | 50 | 57 | 15 | | 46 |
| | 8-9 hr | 58 | 4 | 52-66 | 47 | | | | |
| | 9-10 hr | 58 | 4 | 39-66 | 47 | | | 39-75 | |
| | 1 day | 65 | 85 | 50-75 | 51 | 63 | 190 | 41-74 | 50 |
| | 2 days | | | | | 63 | 95 | 59-80 | 50 |
| | 8 days | | | | | 71 | 95 | | 54 |
| 6 | Initial | 62 | 38 | 52-74 | 49 | | | | |
| | 0-1 hr | 52 | 15 | 47-60 | 43 | | | | |
| | 1-2 hr | 60 | 10 | 58-71 | 48 | | | | |
| | 2-3 hr | 48 | 25 | 28-63 | 41 | | | | |
| | 3-4 hr | 51 | 25 | 28-66 | 43 | | | | |
| | 4-5 hr | 47 | 25 | 28-71 | 40 | | | | |
| | 5-6 hr | 49 | 40 | 25-66 | 42 | | | | |
| | 6-7 hr | 49 | 25 | 33-66 | 42 | | | | |
| | 7-8 hr | 53 | 20 | 36-63 | 44 | | | | |
| | 8-9 hr | 46 | 15 | 33-63 | 40 | | | | |
| | 1 day | 47 | 50 | 33-64 | 40 | | | | |
| | 2 days | 48 | 50 | 37-64 | 41 | | | | |
| | 24 days | 66 | 35 | 58-70 | 51 | | | | |

The data collected at Site 5 (Table 5) show that there was a decrease after both treatments with slightly more of a decrease after the second treatment. The predicted car skid values, however, remained near the border line between good and fair at all times. Recovery was not entirely complete even 2 days after the second treatment.

At Site 6 the decrease was very significant; the predicted car value dropped to the border line between fair and poor (40) and remained at that level for at least 2 days. The large decrease was expected because of the poor surface condition, and, undoubtedly, would have been greater had not the intended application amount been reduced from 0.040 to 0.020 gal/sq yd (the actual application was 0.024 gal/sq yd). This was true despite the fact that the application conditions were considered very good with regard to air and surface temperatures.

TABLE 6

SUMMARY OF PREDICTED CAR SKID NUMBER EXPRESSED AS PERCENTAGE OF INITIAL VALUE AT SITES 1 THROUGH 6

| Time After Treatment | Good Conditions | | | | | | Fair Conditions | | | | | Poor Conditions | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Site 1 | Site 2 | Site 3 | Site 5 | $\overline{X}$ | R | Site 3[a] | Site 4 | Site 5 | $\overline{X}$ | R | Site 4[a] | Site 6 | $\overline{X}$ | R |
| Initial | 100 | 100 | 100 | 100 | 100 | 93-107 | 100 | 100 | 100 | 100 | 88-108 | 100 | 100 | 100 | 88-114 |
| 0-4 hr | 93 | 91 | 100 | 94 | 94 | 72-103 | 93 | 82 | 90 | 88 | 66-108 | 77 | 90 | 84 | 52-110 |
| 4-8 hr | 93 | 88 | | 91 | 91 | 63-100 | | 72 | 86 | 79 | 56-103 | 76 | 86 | 81 | 57-110 |
| 8-12 hr | 93 | 87 | | 87 | 89 | 67-95 | | | | | | 76 | 82 | 79 | 64-102 |
| 1 day | | | | 95 | 95 | 78-105 | | 94 | 93 | 94 | 67-106 | 76 | 82 | 79 | 54-102 |
| 2 days | | | | | | | | | 93 | 93 | 69-104 | | 84 | 84 | 70-102 |
| 3 days | | | | | | | 96 | | | 96 | 84-102 | | 84 | | |
| 4 days | | | | | | | | | | | | 84 | | | |
| 7 days | 98 | 96 | | | 97 | 91-103 | | | | | | | | 84 | 66-96 |
| 8 days | | | | | | | | | | 100 | 87-109 | | | | |

[a]Second application.

## CONCLUSIONS

The data given in Table 6 are expressed as percentage of initial predicted skid number and grouped according to the surface condition rating. Figure 2 shows these same data in graphic form.

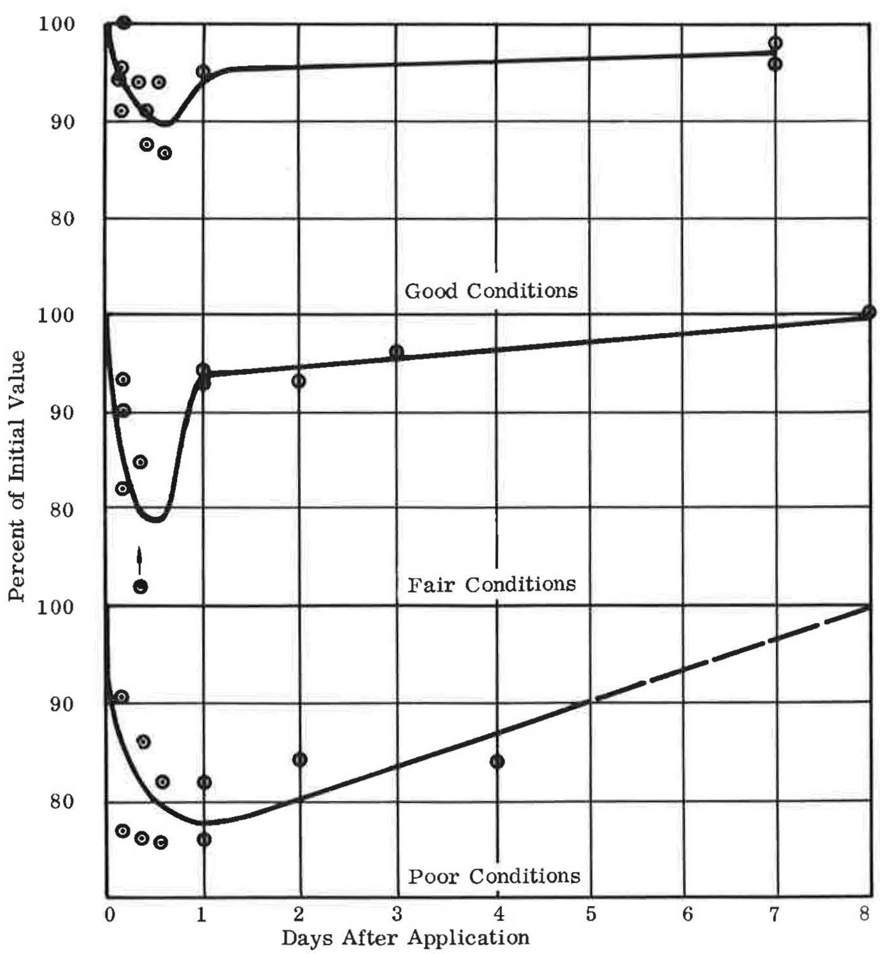


Figure 2. Recovery of skid resistance after linseed oil treatment.

Although the data are sparse in places, definite trends can be seen. With good conditions the average skid readings dropped to only about 90 percent of the initial values and had recovered to 95 percent in 1 day. With fair conditions the skid readings dropped to between 75 and 80 percent of the initial values during the first day but, as with the good conditions, recovered to about 95 percent after 1 day. With poor conditions the skid values dropped to about 80 percent and, unlike the other conditions, remained at that level for 2 days. Even 4 days after the treatment the skid values were less than 90 percent of the initial value. It is interesting to note that, although the decrease and early recovery are different for the 3 conditions, the time required for full recovery, 8 to 9 days, appears to be the same for all.

Of course, the percentages of the initial skid value are averages. The actual ranges of skid values that correspond approximately to the 95 percent confidence limits are given in Table 6. The range increases greatly after application, which is the result of unequal applications, nonuniform application conditions, and nonuniform drying. The variability does not return entirely to normal until 7 to 8 days after the treatment.

The amount of linseed oil applied appears to have a definite influence on the loss and recovery of skid resistance when the surface condition is poor prior to treatment. When the surface condition is fair or good, the amount of oil applied (within the normal limits) does not appear to affect the skid resistance. Figure 3 shows the effect of application amount after a 1-day curing period with either a fair to good surface condition or a poor surface condition.

Based on the information shown in Figure 2 and given in Table 6, the probable skid conditions after a linseed oil treatment are given in Table 7. The numbers with the footnote reference indicate an average skid value of 40 or below, and the blocked-in areas indicate conditions where 15 to 20 percent of the area tested was below a value of 40. As indicated previously, a skid value of 40 is the border line between fair and poor. Therefore, the importance of considering the pretreatment surface rating and surface condition is evident in that skid resistance falls to a lower level and remains at that low level longer with a poor surface rating. Of course, the lower the initial value is, the lower the skid resistance falls after a treatment. Unfortunately, the same conditions that result in a poor rating, i.e., smooth texture and heavy road film, also result in a relatively low initial skid resistance.

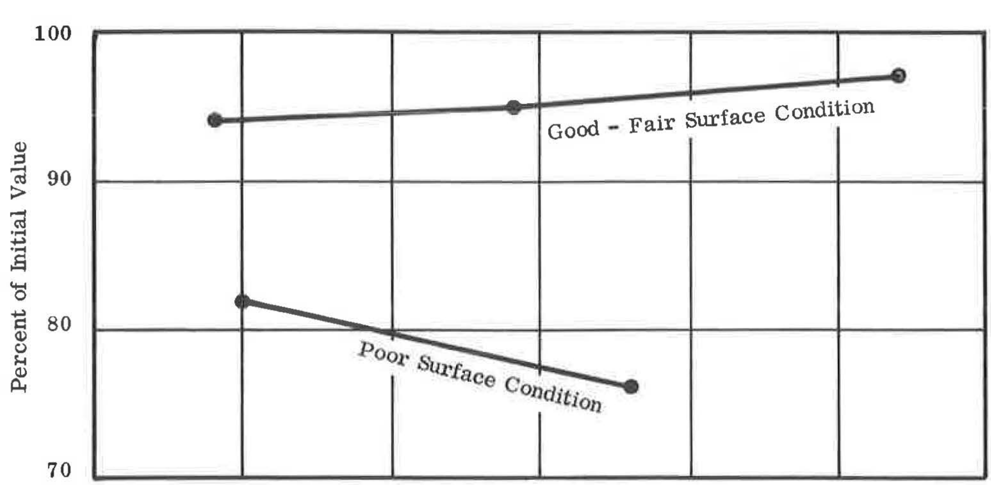


Figure 3. Effect of application amount on skid resistance (1-day cure).

Recovery of dry pavement skid resistance is almost complete 4 to 6 hours after the treatment. For up to 3 days the delay time between the first and second treatment had little effect on the skid resistance after the second treatment.

## RECOMMENDATIONS

Based on the results of the tests run, the following recommendations are made with regard to skid resistance of linseed oil treated pavements.

1. Prior to any linseed oil treatment, a close evaluation should be made of the surface condition and pavement skid resistance and the planned application amount decreased if necessary. Also, this evaluation should help determine what precautionary measures should be taken, such as when to let traffic back on the treated pavement or when to use SLIPPERY WHEN WET signs, if the pavement gets wet after the treatment. It is not necessary to have precautionary measures beyond 7 days. Figures 2 and 3, as well as Table 7, are offered as aids in making any decisions necessary.

TABLE 7

PROBABLE RECOVERY OF SKID RESISTANCE ON LINSEED OIL TREATED PAVEMENTS

| Time After Treatment | Surface Rating | Skid Numbers | | | |
|---|---|---|---|---|---|
| Initial | Good | 55 | 50 | 45 | 40 |
| 0-4 hr | | 52 | 47 | 42 | 38[a] |
| 4-8 hr | | 50 | 45 | 41 | 36[a] |
| 8-12 hr | | 49 | 44 | 40[a] | 35[a] |
| 1 day | | 52 | 47 | 42 | 38[a] |
| 2 days | | 53 | 47 | 42 | 38[a] |
| 3 days | | 53 | 47 | 42 | 38[a] |
| 4 days | | 53 | 48 | 43 | 38[a] |
| 7 days | | 54 | 49 | 44 | 39[a] |
| 8 days | | 55 | 50 | 45 | 40[a] |
| Initial | Fair | 55 | 50 | 45 | 40 |
| 0-4 hr | | 47 | 43 | 39[a] | 34[a] |
| 4-8 hr | | 44 | 40[a] | 36[a] | 32[a] |
| 8-12 hr | | 42 | 38[a] | 35[a] | 31[a] |
| 1 day | | 51 | 46 | 42 | 37[a] |
| 2 days | | 52 | 47 | 43 | 38[a] |
| 3 days | | 53 | 48 | 43 | 38[a] |
| 4 days | | 53 | 48 | 44 | 39[a] |
| 7 days | | 54 | 49 | 44 | 40[a] |
| 8 days | | 55 | 50 | 45 | 40[a] |
| Initial | Poor | 55 | 50 | 45 | 40 |
| 0-4 hr | | 45 | 41 | 37[a] | 33[a] |
| 4-8 hr | | 44 | 40[a] | 36[a] | 32[a] |
| 8-12 hr | | 43 | 39[a] | 35[a] | 31[a] |
| 1 day | | 43 | 39[a] | 35[a] | 31[a] |
| 2 days | | 44 | 40[a] | 36[a] | 32[a] |
| 3 days | | 46 | 41[a] | 37[a] | 33[a] |
| 4 days | | 48 | 43 | 39[a] | 35[a] |
| 7 days | | 54 | 49 | 44 | 39[a] |
| 8 days | | 55 | 50 | 45 | 40[a] |

Note: Blocked-in areas represent 15 to 20 percent of individual test results below a value of 40.

[a]Average skid value of 40 or below.

2. If there are wet pavement conditions (rain), the treated area should not be opened to traffic for 24 hours after treatment unless skid tests indicate the pavement has sufficient skid resistance (predicted car skid number of 40 or above). Judgment will have to be used to determine if the pavement should be opened to traffic after the 24-hour period. Whenever possible, it would be desirable to run skid tests before making any decisions.

3. If there are dry pavement conditions, the treated area can be opened to traffic as soon as 4 to 6 hours after the treatment.

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## *Discussion*

C. E. MORRIS, National Flaxseed Processors Association, Chicago—Reference is made in this paper to possible skid hazards resulting from the application of linseed oil to concrete highway surfaces, particularly if the pavement is wet with water relatively soon after the oil is applied.

Although we do not as yet have quantitative data, a number of qualitative observations have shown that a light application of fine silica sand made while the oil is still wet is highly effective in providing adequate skid resistance during the period when an untimely rain might possibly introduce a hazardous condition. This might well eliminate the need for the SLIPPERY WHEN WET signs to which the author referred.

On the Ohio Turnpike, the application of ¼ lb/sq yd of sharp silica sand meeting the following specifications has been found to be very satisfactory: The silica sand shall weigh approximately 2,700 lb/cu yd and shall be sharp, angular, or subangular silica sand with a silica content of not less than 95 percent and a clay content of not more than 1 percent. The moisture content of the sand shall not exceed 5 percent. The silica sand shall be produced from the deposit known as the Sharon Conglomerate and shall meet the following gradation requirements:

| Sieve | Percent Retained | Cumulative |
|---|---|---|
| 20 mesh | 00.10 | 00.10 |
| 30 mesh | 01.50 | 01.60 |
| 40 mesh | 08.40 | 10.00 |
| 50 mesh | 14.00 | 24.00 |
| 70 mesh | 22.50 | 46.50 |
| 100 mesh | 33.50 | 80.00 |
| 140 mesh | 15.00 | 95.00 |
| 200 mesh | 04.40 | 99.40 |
| 270 mesh | 00.30 | 99.70 |
| Pan | 00.30 | 100.00 |

STEPHEN N. RUNKLE, Closure—I appreciate Mr. Morris's interest in my paper. In some cases we did apply sand to the treated pavement, but not while the linseed oil mixture was still wet. I do not know the gradation of the sand we used, but it certainly was

TABLE 8

SKID TEST RESULTS AFTER APPLICATION OF SAND TO LINSEED

| Site | Lane | Time After Treatment | Trailer Skid Number | Predicted Car Skid Number | Comment |
|---|---|---|---|---|---|
| 5[a] | Southbound passing | Initial | 63 | 50 | |
| | | 0-3 hr | 59 | 47 | |
| | | 3-6 hr | 57 | 46 | |
| | | 6-8 hr | 58 | 47 | Sanded just prior to tests |
| | | 24 hr | 63 | 50 | Sand gone |
| 5[a] | Northbound passing | Initial | 67 | 52 | |
| | | 0-3 hr | 61 | 49 | |
| | | 3-6 hr | 66 | 51 | Sanded just prior to tests |
| | | 24 hr | 67 | 52 | Sand gone |
| 6 | Southbound center | Initial | 62 | 49 | |
| | | 0-2 hr | 56 | 45 | |
| | | 2-4 hr | 43 | 39 | |
| | | 4-6 hr | 52 | 43 | Sanded just prior to tests |
| | | 6-8 hr | 49 | 42 | Most sand gone |
| | | 24 hr | 47 | 40 | All sand gone |
| | | 48 hr | 47 | 40 | |
| 6 | Northbound center | Initial | 59 | 47 | |
| | | 2-4 hr | 54 | 44 | |
| | | 4-6 hr | 44 | 39 | |
| | | 6-8 hr | 52 | 43 | Sanded just prior to tests |
| | | 8-10 hr | 48 | 41 | Most sand gone |
| | | 24 hr | 47 | 40 | All sand gone |
| | | 48 hr | 49 | 42 | |

[a]Second application of oil.

coarser than the type of sand Mr. Morris suggests using. The sand we used most likely did have a silica content greater than 95 percent.

Some improvement in skid resistance resulted after the sand was applied as is illustrated in Table 8. Sand was applied to Site 5, which was 20 lane-miles, from 3 to 8 hours after the second application of linseed oil; and a slight increase in the predicted car skid number of 1 to 2 skid numbers resulted. Site 6 had only one application of linseed oil of about 0.024 gal/sq yd. Sand was applied at Site 6 from 4 to 8 hours after the linseed oil application and increased the predicted car skid number by 4 skid numbers. However, the improvement was temporary as the skid numbers began to decrease again shortly after the sanding because the sand was blown from the surface by traffic.

If the method suggested by Mr. Morris offers more permanent improvement in skid resistance than we experienced, it perhaps would greatly eliminate the problem of low skid resistance after linseed oil treatments. I would be interested in learning of any skid test results obtained after using the method he describes.

# Concrete Repairs With Epoxy and Polyester Resins

R. K. GHOSH, Central Road Research Institute, New Delhi

This paper reports results from laboratory investigation and field trials on concrete repairs using epoxy and polyester resins with particular reference to outdoor works such as repair of concrete pavement.

•A STUDY was conducted at the Central Road Research Institute, New Delhi, with heat-convertible synthetic resins such as epoxy and polyester resins to determine their efficacy in concrete repairs with particular reference to concrete pavement. Although both of these resins, particularly the epoxy resin, have been in use for concrete repairs for sometime past (1, 2, 3), more information is needed to establish satisfactorily their different engineering properties in relation to environmental and job conditions.

## LABORATORY INVESTIGATIONS

### Materials

In the epoxy resin formulation, the materials used were diphenyl-propane epichlorohydrin, polysulfide polymer, and tertiary amine. The tertiary amine, a curing agent of catalytic type, was used to obtain an epoxy resin system with a relatively long pot life.

The materials used in the polyester resin formulation were polyester resin (solution of unsaturated polyester and monomer styrene), catalyst (methyl ethyl ketone peroxide), and accelerator (cobalt naphthanate). The purpose of the accelerator was to hasten up the reaction. The quantities of curing agent and accelerator were selected from consideration of gel time, curing schedule, and satisfactory pot life for outdoor works under tropical climatic conditions of India. Decisions on the optimum and economical proportions of the different resin constituents were based on some preliminary tests.

Jumna sand of fineness modulus 0.95 was used in the preparation of resin mortars.

### Mix Proportions

In the case of polyester resin formulation, the gelation of which was sensitive to temperature and humidity, the quantity of accelerator was varied from 0.5 to 1.5 ml per 100 ml of polyester resin, keeping the proportion of catalyst the same, i.e., 2.5 ml. The mixing, compaction, and finishing of 1:3 resin sand mortar were done manually at an average temperature of 29 C. The results of compressive strength and void content determined on 1½-in. cubes after 2 days of curing at 29 C in a thermostatically controlled oven are given in Table 1. It was observed that an increase in the amount of accelerator from 0.5 to 1.5 ml per 100 ml of resin reduced the pot life of the formulation at 32 C from 60 to 28 min, while increasing the 2 days' compressive strength from 9,037 to 10,963 psi. The void content increased from 1.5 to 1.9 percent when the amount of accelerator was increased from 0.5 to 1.5 ml per 100 ml of resin, presumably due to entrapping of more air as a result of quick-setting.

Paper sponsored by Committee on Adhesives, Bonding Agents and Their Uses.

TABLE 1

EFFECT OF QUANTITY OF CURING AGENT AND ACCELERATOR ON STRENGTH AND VOID PERCENTAGE OF RESIN SAND MORTARS

| Binder (Resin) | Accelerator | Curing Agent (phr) | Pot Life (min) | Resin Sand Mortar (1:3) Cured for 48 Hr at 29 C (avg) | |
|---|---|---|---|---|---|
| | | | | Void (percent) | Compressive Strength (psi) |
| Polyester[a] (100 ml of polyester and 2.5 ml of catalyst) | 0.5 | | 60 | 1.5 | 9,037 |
| | 1.0 | | 35 | 1.5 | 10,518 |
| | 1.5 | | 28 | 1.9 | 10,963 |
| Epoxy[b] (100 phr of diphenyl-propane epichlorohydrin and 53 phr of poly-sulfide polymer) | | 4 | 50 | 1.05 | 5,185 |
| | | 5 | 45 | 1.75 | 6,281 |
| | | 7 | 35 | 2.65 | 6,444 |
| | | 9 | 25 | 2.68 | 6,667 |
| | | 11 | 19 | 3.00 | 7,000 |

[a]Resin components about a year old.

[b]Resin components freshly obtained.

Because the decrease in pot life reduced the usable period of operation and also because the atmospheric temperature in a large part of India during summer months may be 45 C and still higher on a concrete surface exposed to the sun, the amount of accelerator chosen for subsequent investigation was 0.5 ml per 100 ml of resin. It was felt that the warm environmental condition would itself act an accelerator and complete the exothermic reaction at a faster rate.

Similar tests were also carried out with epoxy resin formulation by varying the quantity of the curing agent from 4 to 11 parts per hundred parts of resin (phr) and by keeping the same quantities of diphenyl-propane epichlorohydrin (100 phr) and polysulfide polymer (53 phr) (Table 1). It was observed that the results were similar to those obtained with the polyester resin. An increase in the amount of curing agent from 4 to 9 phr reduced the pot life at 32 C from 50 to 25 min and increased the void content from 1.05 to 2.68 percent, while increasing the compressive strength at 29 C average curing temperature from 5,185 to 6,667 psi. Because the difference in strength was not high, from the point of view of pot life and void content, the proportions chosen for the different constituents were diphenyl-propane epichlorohydrin, 100 phr; polysulfide polymer, 53 phr; and curing agent, 4 phr.

## Properties

With the selected formulations, studies were conducted on resin sand mortars (1:3, 1:4, and 1:5 by weight) to determine compressive (at 2,000 psi per min loading rate), tensile (as per IS 269, 1958), and flexural (at 100 psi per min loading rate) strengths under curing at 32 C and at elevated temperatures of 100 and 200 C. The specimens consisted of 1½-in. cubes, 1- by 1- by 6-in. bars, and standard briquettes for compressive, flexural, and tensile strength tests respectively. The curing period was varied from 3 hours to 14 days. The results given in Table 2 show that 1:3 polyester resin sand mortar could yield compressive and tensile strengths of 4,000 and 600 psi respectively after 6 hours of curing at 32 C. In the case of epoxy resin mortar, a period of 24 hours was found to be necessary to obtain the same compressive strength. The flexural strength values for richer mixes (1:3 or 1:4) of both the resin mortars were quite close (5,000 to 7,000 psi) to each other. There was, however, a sudden drop when the mix was lean (1:5) such that the polyester resin mortar gave a flexural strength of about 60 psi only after 7 days of curing. The rate of strength gain increased with elevation of the curing temperature up to about 100 C. Higher temperatures, particularly for a curing period of 12 hours or more, resulted in decreased strength.

The gluing properties of 1:3 resin sand mortars with cement concrete were determined through direct shear test at the bonded interface (4), flexural strength, and tensile strength on composite glued cement concrete cubes (4 in.), cement concrete beams (4 by 4 by 20 in.), and standard cement mortar briquettes respectively. Tests

TABLE 2

STRENGTH RESULTS OF RESIN SAND MORTARS

| Mix (by wt) | Curing Period | | Polyester Resin | | | Epoxy Resin | | |
|---|---|---|---|---|---|---|---|---|
| | Time | Deg C | Compressive Strength (psi) | Tensile Strength (psi) | Flexural Strength (psi) | Compressive Strength (psi) | Tensile Strength (psi) | Flexural Strength (psi) |
| 1:3 | 3 hr | 32 | 3,111 | — | — | Curing was not complete | — | — |
| | 6 hr | 32 | 4,000 | 600 | — | 400 | 200 | — |
| | 18 hr | 32 | 6,666 | 1,500 | — | 2,666 | 800 | — |
| | 24 hr | 32 | 9,556 | 1,500 | — | 4,000 | 1,000 | — |
| | 48 hr | 32 | 12,000 | 1,700 | — | 5,111 | 1,400 | — |
| | 96 hr | 32 | 12,899 | 1,850 | — | 6,000 | 1,600 | — |
| | 7 days | 32 | 13,111 | — | 6,300 | 6,889 | — | 6,750 |
| | 14 days | 32 | 13,333 | — | | 7,111 | — | |
| 1:4 | 7 days | 32 | 10,222 | 1,850 | 5,400 | 6,667 | 1,500 | 5,625 |
| 1:5 | 7 days | 32 | 1,773 | 222 | 62.5 | 4,889 | 1,000 | 3,600 |
| 1:3 | 12 hr | 50 | 7,667 | — | — | 4,111 | — | — |
| | | 100 | 10,222 | — | — | 4,667 | — | — |
| | | 200 | 5,555 | — | — | 3,778 | — | — |
| 1:4 | 12 hr | 100 | 7,444 | — | — | 4,000 | — | — |
| | | 200 | 4,000 | — | — | 3,555 | — | — |
| 1:5 | 12 hr | 100 | 2,111 | — | — | 3,333 | — | — |
| | | 200 | 1,270 | — | — | 2,778 | — | — |

were first conducted on monolith specimens cured 28 days. The broken parts were glued together by applying a thin primer coat of resin formulation and then a $^1/_{16}$-in. thick resin sand mortar and retested. The curing was done for 2 days at 32 C. The results given in Table 3 show that the bond between hardened cement concrete and resin sand mortar was satisfactory. Compared to monolith specimens, the glued samples showed a reduction of the order of 30 to 40 percent, 20 to 30 percent, and 5 to 10 percent

TABLE 3

GLUING AND BONDING PROPERTIES OF RESIN SAND MORTARS WITH CEMENT CONCRETE AND MORTAR

| Specimen | Bonding Media | Direct Shear Strength of Cement Concrete Cubes (psi) | | Flexural Strength of Cement Concrete Beams (psi) | | Tensile Strength of Standard Cement Mortar Briquettes (psi) | |
|---|---|---|---|---|---|---|---|
| Monolith, 28 days old | — | 591 | 510 | 260 | 280 | 380 | 380 |
| Glued, 2 days' curing at 32 C | Polyester resin sand (1:3) | 358 | | 205 | | 360 | |
| Glued, 2 days' curing at 32 C | Epoxy resin sand (1:3) | | 350 | | 200 | | 350 |
| Monolith, 28 days old | — | 317 | 317 | | | | |
| Overlaid with new concrete, additional 28 days' curing in water | Polyester formulation | 120[a] | | | | | |
| Overlaid with new concrete, additional 28 days' curing in water | Epoxy formulation | | 300[b] | | | | |
| Overlaid with new concrete, additional 28 days' curing in water | Cement sand slurry (1:1) | 246[a] | | | | | |

[a]Failure at interface. [b]Failure in concrete overlay.

in direct shear, flexural, and tensile strengths respectively. In flexural and tensile test specimens, the failure was not at the glued surface, indicating that the reduction in strength was most probably due to formation of fatigue cracks near the plane of rupture during the first test.

In addition direct shear test was conducted to determine the efficacy of both the resin formulations as bonding media between old and new cement concrete for works such as patch repairs and overlay. The application consisted of a thin coat of resin formulation over 28-day-old surface-dry concrete specimens. The new concrete was placed immediately afterward, followed by compaction and finishing, The test results after 28 days of normal curing of new concrete in water are given in Table 3. It is seen that, although epoxy resin formulation can be used as a satisfactory bonding media for concrete overlay work, with polyester resin there was considerable reduction (50 to 60 percent) in bond strength.

In durability tests 1:3 resin sand mortar samples were subjected to (a) alternate heating and cooling (one cycle consisted of heating at 50 C in an oven for 6 hours and cooling in water at 27 C ± 2 C for 18 hours); (b) freezing and thawing (one cycle consisted of 18 hours of freezing at -3 C in a cooling chamber followed by thawing in water at 27 C ±2 C for 6 hours); and (c) 7 days' continued immersion in water at 27 C ±5 C. While in the first 2 cases flexural strength was obtained on 1- by 1- by 6-in. mortar bars, the effect of continued immersion was determined on 1½-in. cubes through compressive strength test (Table 4). The results show that both of the mortars were adversely affected by the durability tests. This was more so with polyester resin mortar. After 90 cycles of heating and cooling, the reduction in flexural strength was 56 and 45 percent for polyester and epoxy resin mortars respectively. Alternate freezing and thawing test showed a strength reduction of 40 and 33 percent for polyester and epoxy resin mortar respectively after 90 cycles. The strength reduction due to continued immersion in water for 7 days was 30 to 40 percent for both polyester and epoxy resin mortars.

## FIELD TRIALS

Some full-scale field trials were conducted in 1964 with 1:3 polyester resin sand mortar to repair fine and wide cracks, shallow and deep low spots, and broken edges of concrete slabs in one of the airfields. It was observed that, although the performance of such repairs was quite satisfactory up to a year or two, the cracks reappeared

TABLE 4

RESULTS OF DURABILITY TESTS OF CURED SAMPLES

| Item | 91 Days in Air at 27 C | 24 Hr at 27 C | | | 7 Days in Air at 32 C | 7 Days at 32 C[d] |
|---|---|---|---|---|---|---|
| | | 1[a] | 2[b] | 3[c] | | |
| Flexural strength, psi | | | | | | |
| Polyester 1:3 | 5,626 | 3,825 | 2,475 | 3,452 | | |
| Epoxy 1:3 | 6,150 | 3,825 | 3,375 | 4,127 | | |
| Compressive strength, psi | | | | | | |
| Polyester 1:3 | | | | | 13,111 | 9,111 |
| Polyester 1:4 | | | | | 10,222 | 6,667 |
| Polyester 1:5 | | | | | 1,773 | 889 |
| Epoxy 1:3 | | | | | 6,889 | 4,444 |
| Epoxy 1:4 | | | | | 6,667 | 4,222 |
| Epoxy 1:5 | | | | | 4,888 | 2,222 |
| Water absorption, percent | | | | | | |
| Polyester 1:3 | | | | | | 0.30 |
| Polyester 1:4 | | | | | | 0.67 |
| Polyester 1:5 | | | | | | 8.70 |
| Epoxy 1:3 | | | | | | 0.15 |
| Epoxy 1:4 | | | | | | 0.32 |
| Epoxy 1:5 | | | | | | 4.0 |

[a]After 90 cycles of alternate heating at 50 C in air and cooling at 27 C in air.
[b]After 90 cycles of alternate heating at 50 C in air and cooling at 27 C in water.
[c]After 90 cycles of alternate freezing at -3 C and thawing at 27 C in water.
[d]After 7 days of immersion in water at 27 C.

Figure 1. Preparation of central groove in the freshly laid resin sand mortar along a crack under repair on National Highway 2.

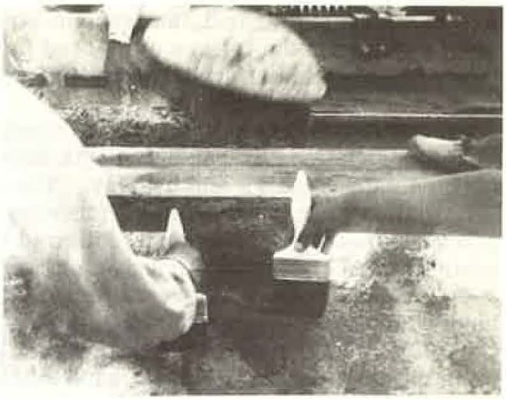

Figure 2. Application of epoxy resin formulation to treated surface of existing concrete slabs immediately before placing overlay concrete on National Highway 2.

subsequently. This was probably due to fatigue of bond between the resin mortar and cement concrete and inadequate contact area between them. It was felt that, in the repair of the cracks, trapezoidal notch with larger width at the bottom should be preferred to V-notch, and forming a regular joint in the mortar might prove effective in preventing subsequent spalling of the repaired portion. The latter was to provide a guided, weakened plane. This was tried in 1966 when some cracks were repaired in cement concrete slabs on National Highway 2 near Delhi by using both the resins (Fig. 1). The grooves were later sealed with a joint sealing compound to prevent ingress of water. The performance of these repairs over a period of about 1½ years has been very satisfactory.

About 1,000 sq ft of an old 5-in. cement concrete pavement on National Highway 2 near Delhi was overlaid with a 2-in. cement concrete surfacing by using epoxy resin formulation as bonding media (Fig. 2). The treatment to the existing concrete surface followed the usual procedure recommended for rigid overlay work. The green concrete was placed immediately after the resin coat was applied to the dry surface of the old cement concrete. The section has been giving satisfactory performance for the past 1½ years.

## CONCLUSION

For outdoor works in tropical countries such as India, the amount of curing agent required to be admixed with epoxy resin was found to be 4 to 5 phr instead of the 9 phr used in colder climates and for indoor works. In the case of polyester resin, the quantity of accelerator necessary was 0.5 ml per 100 ml of the resin. Higher amounts of curing agent or accelerator not only decreased the pot life of the formulation considerably thus making its use uneconomical, but also resulted in increased void content and only inconsiderable gain in strength. For high early strength, polyester resin mortar was found to be preferable. Mortar mixes leaner than 1:4 were found to be unsuitable for repair work. Curing at temperatures up to 100 C was beneficial, but higher temperatures affected the strengths adversely. With dry surfaces of concrete, both resin mortars gave good bond. These mortars were, however, susceptible to alternate heating and cooling, alternate freezing and thawing, and continued contact with moisture, although the effect on polyester resin mortar was more severe. In rigid overlay construction involving laying of green concrete over the old concrete surface, only epoxy resin formulation could come in question as bonding media.

In repair of cracks and low spots, both types of resin mortars could be used. A trapezoidal notch was preferred to a V-notch. In concrete pavement repairs, a regular joint could be made by making a groove in the resin mortar before it was dry.

## ACKNOWLEDGMENT

The assistance rendered in the laboratory by C. S. Pant and in the field trials by K. L. Sethi of the Central Road Research Institute is hereby acknowledged. The paper is published with the permission of the director of the Central Road Research Institute.

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# Pavement-Salvaging Experience on the Pennsylvania Turnpike

R. H. KLUCHER, Pennsylvania Turnpike Commission

This paper outlines the experiences of the Pennsylvania Turnpike Commission in salvaging concrete pavement with bituminous overlays and in salvaging bituminous pavement with either a bituminous overlay or a surface treatment. Design features, specification requirements, construction operations, and problem areas are outlined briefly. This is not a research paper and is not so intended. It is an informative paper only.

•THE PENNSYLVANIA TURNPIKE heads into its thirtieth year in 1970, bigger and better in every respect than the original toll road that opened to the public on October 1, 1940.

The Turnpike was authorized by the General Assembly of the Commonwealth of Pennsylvania on May 21, 1937. This first of the modern turnpikes had a modest beginning and an uncertain future. The 4-lane highway started in Irwin in Westmoreland County, crossed the Appalachian Mountains, and ended in the little town of Middlesex in Cumberland County, a distance of 160 miles.

Many doubters appeared on the scene when the turnpike concept was first mentioned. Few people had enough foresight to anticipate the welcome a superhighway would receive from the average motorist. The facility was referred to by many names—some complimentary and some uncomplimentary.

Two weeks after the road was opened to traffic, an average of 26,000 vehicles used the scenic highway daily. Today, more than 140,000 vehicles use the facility daily.

Since the opening of the original section, 4 extensions have been built to the original 160 miles to make its present length 470 miles. The Turnpike now extends from the Ohio-Pennsylvania line eastward to the New Jersey-Pennsylvania line and northward from the Philadelphia area to the city of Scranton.

The rapid growth of the Turnpike, from 160 miles in 1940 to 470 miles in 1957, is all the more remarkable because all construction work is paid for out of tolls collected from the users. Taxes are not utilized for Turnpike construction, maintenance, or operations.

## DESIGN FEATURES

Design and construction on the original section of the Turnpike took place in the middle and late 1930's, and traffic began using the roadway in October 1940. When the Turnpike was opened to traffic it was considered to be one of the finest highways ever constructed. In fact, many features incorporated into this facility were so modern that they have been incorporated into the Interstate System, but on a much larger scale than was ever anticipated at that early date. Think about these modern ideas and the place they play in today's superhighway design: limited access, fenced-in facility; access at interchanges only; no at-grade crossings of highways and railroads; opposite traveling lanes of traffic divided by a 10-ft median, thought by many at that time to be excessive; service plazas, roadside rest areas, and roadside picnic tables; maximum grades of 3 percent; high speed limit of 65 mph; and patron service and roadway patrols 24 hours every day.

Paper sponsored by Committee on Salvaging Old Pavements by Resurfacing and presented at the 49th Annual Meeting.

The fact that all of these features—except the service plaza and patron service concepts—have been incorporated into the Interstate System points out the foresight of the designers of this pioneering road construction program.

As I indicated earlier, the original section of the Pennsylvania Turnpike was opened to traffic during the latter part of 1940. The design features for this section consisted of the following:

1. Two 12-ft reinforced concrete lanes 9 in. in depth on each side of a 10-ft grass median;
2. A straight slope $1\frac{1}{4}$ in. per 12-ft lane and $1\frac{3}{4}$ in. per 12-ft lane on the median and shoulder lanes respectively;
3. No special subgrade;
4. Shoulders 10-ft wide in both cut and fill sections, but not paved or stabilized, with slope on the fill shoulder $\frac{1}{2}$ in./ft, and slope on the cut shoulders $\frac{3}{4}$ in./ft for the first 7 ft and $1\frac{1}{2}$ in./ft for the remaining 3 ft; and
5. A right-of-way of 200 ft.

Although many of these design features were adequate at that early date, the large increase in passenger car and heavy truck traffic after World War II had a tremendous effect on the roadway surface. So great was the effect that in the early 1950's the original pavement began to show extreme signs of deterioration. The Commission decided to undertake an extensive salvage program soon thereafter. No attempt was made to increase the design strength of the pavement except the increase that resulted through the additional depth due to the overlay.

## SEQUENCE OF ORIGINAL SALVAGE OPERATIONS

This salvage program consisted of the following progressive sequential operations to effect the necessary repairs over the original 160-mile section:

1. Concrete slabs removed where deterioration or adverse drainage conditions necessitated such action and subgrade and drains placed before replacing the concrete;
2. Hot asphalt underseal introduced to fill the voids beneath the slabs and to provide a more stable base for concrete pavement but not to change pavement elevation (Pennsylvania Class U-1 asphalt cement at a temperature of 400 F ±25 F introduced under pressure was used to effect the underseal);
3. Mud jacking used to provide a smoother riding surface in a few isolated areas where uneven pavement settlement was evident (in these areas a cement slurry pumped beneath the slab was used to raise the pavement to the proper elevation);
4. Six-inch perforated or porous pipe placed to drain the roadways where underdrain had not been placed previously or where existing underdrain was not functioning; and
5. Concrete pavement resurfacd with bituminous material and median and shoulders stabilized with an asphalt and aggregate mixture compacted in place.

Salvaging work was undertaken during the summer of 1954 and was completed during the summer of 1962. The work consisted of the resurfacing of the pavement including interchange ramps, approach lanes to service stations, bridges, and other Turnpike facilities with a 2-course bituminous hot mix designated as an ID-2 mix in the Pennsylvania Department of Highways Specification Form 408. The binder course was placed to a 2-in. compacted thickness, and the top course was placed to a 1-in. compacted thickness.

## SPECIFICATION REQUIREMENTS

The Pennsylvania Department of Highways specification was supplemented to meet the following Commission requirements:

1. Class A-1 asphalt cement with a penetration range of 70-80 and a specific gravity at 77 F with a minimum of 1.010;
2. Slag sand and slag coarse aggregate in the wearing course;
3. Either stone or slag coarse aggregate in the binder course; and

4. Binder and wearing courses with the following stability and density requirements: stability (Marshall method) of binder course 800 minimum and of wearing course 1,500 minimum, flow value (Marshall method) 16 maximum, density of laboratory-compacted mixture in percentage of calculated voidless mixture of same materials 94-96, and compacted field density in percentage of laboratory compacted density 95 minimum.

The work proceeded as required, and a smooth riding surface was obtained.

## SURFACE-TREATMENT EXPERIENCES

The resurfacing work just discussed was undertaken at a rather late stage in the deterioration of the original concrete pavement. This factor along with continual heavy traffic growth caused accelerated wear and tear on the salvaged pavement, which required repair work on some sections in the early 1960's.

Consideration was given to the type of repair to be utilized. Plant mix overlays and surface treatments were the 2 methods considered. The decision to utilize a surface treatment was made after much discussion and deliberation by Commission personnel, inasmuch as some members of the staff felt that a surface treatment would not hold up under the traffic volume encountered on the Turnpike and that the surface might tend to become fatty.

In May 1961 eleven experimental test patches were placed in the Allegheny Mountain area. Each patch was 1 by 9 ft and extended across the driving lane so that both wheels of a westbound vehicle would cross the patch. After several weeks of observation, Test 9, 0.25 gal of F-2 emulsion covered with fine slag aggregate, proved the most satisfactory. In order to substantiate this result, a 1,500-ft patch covering the entire 24-ft lane was placed at milepost 129, eastbound. Visual inspection of this patch indicated that good results could be expected from a surface treatment.

After thoroughly discussing the problem with representatives of The Asphalt Institute, with asphalt suppliers, and with its consulting engineer, and after reviewing various test patches utilizing different types of aggregate, the Commission decided to proceed with a surface treatment of slag aggregate and an F-2 asphalt emulsion on 17 miles from milepost 123 to milepost 140.

Slag aggregate was found to give the best results on the test patches simply because it did not polish or become slippery under traffic. The conclusion has been further verified by checking the surface treatments placed during 1961 and thereafter.

Placement of the surface seal was done in the usual manner with bituminous pressure distributors and stone spreaders. All areas at transverse and longitudinal joints that showed reflection cracks greater than ½ in. width, spalling, or raveling were replaced prior to application. Similar repair work was also done in some areas in the surface where excessive alligatoring was observed.

Slag was delivered to assigned locations prior to the start of the job. Here experience taught us a lesson. We found that we could utilize the haul trucks for additional rubber-tire kneading if we located the stockpiles near the starting point of the work.

Traffic was diverted to one side of the median, and 0.25 gal/sq yd of 150 to 160 deg F-2 asphalt emulsion was placed by 2 distributors each spreading a 12-ft wide pattern. Two 12-ft wide Flaherty spreaders placed 15 lb of slag per square yard. Rolling was done by three 10-ton steel tandem rollers—one tandem at each edge and one 3-wheel roller between them. Three passes were made, and the steel rollers continued. Back-rolling was done by two 10-ton pneumatic rollers.

Gradation tests were made for every 100 tons delivered, and a very close inspection resulted in obtaining the best possible material.

I think I should point out a few specific details concerning this operation at this time. Seventeen miles of the eastbound roadway from milepost 123 to milepost 140 was surfaced-treated on September 12, 13, 14, and 15, 1961. Six miles of the westbound roadway from milepost 123 to milepost 129 was surface-treated on September 18. The ranges and weather conditions on the respective dates were as follows:

| Date | Temperature | Weather |
|---|---|---|
| September 12 | 74 to 94 | Clear |
| September 13 | 72 to 96 | Clear |
| September 14 | 68 to 90 | Clear |
| September 15 | 53 to 70 | Cloudy |
| September 18 | 46 to 74 | Clear during the day, rain in the evening |

Traffic was permitted to use the roadway 4 hours after the completion of the rolling operation. No detrimental effect was noted due to the early use by traffic on the sections treated on September 12, 13, and 14. The same cannot be said about the sections completed on September 15 and 18. The day after the portion placed on the fifteenth was opened to traffic, the roadway surface showed some signs of distress and excessive streaking. The Commission staff felt that this condition could be traced to the lower temperatures experienced on that day. A different type of problem was experienced on the 6-mile section placed on the eighteenth. A rain storm developed soon after this section was opened to traffic. The roadway surface—on a 3 percent upgrade with heavy truck traffic concentration—showed some signs of serious distress the following day. A considerable amount of slag chip-off and some asphalt carry-over was noted in the wheel tracks.

## SURFACE-TREATMENT REPAIRS AT PROBLEM AREA

Repair work on this 6-mile section was carried out on September 22 and 23. Repairs were made on one 12-ft lane at a time under varying methods and strict controls during placing operations.

Asphalt was placed at a rate of 0.2 gal/sq yd on the entire section, and slag—dried and heated to a temperature of 300 deg—was then placed and rolled. Traffic was restricted for a 36-hour period after rolling operations were completed. The other 12-ft lane was repaired with unheated slag and traffic was restricted for an 18-hour period. The temperature range during repair work was from 71 to 92 F.

Some slight bleeding was noted, but this was corrected by a dust application. The surface seal—on both the repaired and the unrepaired sections—held up equally well.

## REVISED SURFACE-TREATMENT PROCEDURES

Additional surface treatments have been placed at other locations since that first project in 1961. The same application rate and equipment were utilized as those mentioned earlier. Several minor changes were made, however, for experimental purposes.

After finish-rolling for several hours with rubber-tired rollers, traffic was permitted on the completed surface under 2 separate and distinct methods. In one case, traffic was permitted to use the surface on completion of rolling and 1 hour prior to darkness. In the other case, traffic was not permitted to use the surface until the following day. No appreciable difference was noticeable on the lanes regardless of the method of traffic control.

## SURFACE-TREATMENT COSTS

The cost per two 12-ft wide lanes per mile for the various sections ranges between \$1,400 and \$2,100. The average per mile costs for calendar years 1962, 1963, 1964, 1965, 1967, and 1969 were \$1,706, \$1,422, \$1,681, \$1,605, \$1,623, and \$2,057 respectively. Variations in costs seem to be dependent on the length of the project and the traffic volumes encountered in the given area rather than on the material and labor cost index for the given year. These costs compare favorably with the costs used by the Pennsylvania Department of Highways for its surface seal projects. Department estimates are based on a range from 10 to 12 cents/sq yd. Based on the 12 cents/sq yd figure, the cost per mile for a roadway comparable to ours would approximate \$1,686. Considering the high volume of traffic on the Turnpike and the necessary traffic controls that are required for adequate patron safety, I feel that our costs are more than realistic. This is especially true when compared with Pennsylvania Department of Highways costs because most highway surface seal jobs are done on low-volume rural roadways where extensive traffic controls are not required.

One of the unknown factors to us was the actual cost of the various operations involved in surface-treatment work. At least we had never broken these costs down in a refined manner. We decided to do just that on our latest job. On a sq-yd basis our costs were as follows:

| Item | Cost |
|---|---|
| Traffic control | \$0.01 |
| Labor and equipment | 0.035 |
| Material | 0.07 |
| Total | \$0.115 |

Traffic control—although a minor item normally—accounts for 8.7 percent of the total cost. This is higher than we thought it would be.

Aggregate gradation in surface-treatment operations were as follows:

| Screen | Percent Passing |
|---|---|
| $^3/_8$ in. | 100 |
| $^1/_4$ in. | 60-80 |
| No. 4 | 25-45 |
| No. 8 | 0-10 |

## PRESENT PAVEMENT-SALVAGE PROCEDURES

In 1965 it was evident that 32 miles of the originally resurfaced roadway was in need of extensive repairs to preserve the riding quality and to prevent deterioration of the bituminous surfacing placed earlier. This section extended from Irwin to the Laurel Highlands of the Alleghenies. Extensive lengths on 3 percent grades and increasingly heavy volume of truck traffic were responsible for the deterioration.

The roadway was repaired by removing distressed areas and patching with a bituminous wearing course material. Most of the distressed areas were over the existing concrete pavement joints and resulted from pumping action of the pavement. Additional drainage and concrete pavement replacement was necessary to repair this condition. We also discovered that in many instances the 6-in. underdrain placed earlier to prevent such pumping action was no longer functioning and required replacement. The extent of these repairs further indicated the adverse effects of the increased traffic and truck loadings.

After considerable investigation by and discussion among all interested parties, we decided to improve the entire roadway, including shoulders and drainage, during resurfacing operations. The selection of the methods to be used was complicated by the necessity to maintain traffic during operations with minimum disturbance to the patrons. Complete repaving of the roadway and removing and replacing the existing 3-in. bituminous surface were 2 repair methods considered. Because it was not economically feasible to rebuild the entire roadway and because all agreed that the base was satisfactory, we decided to utilize the same resurfacing technique over the bituminous surface as was used previously over the concrete surface. Specifications were upgraded, however, to reflect our experience as well as to utilize the latest thinking regarding materials, equipment, and construction methods.

The first operation consisted of placing 6-in. perforated underdrain within the median to provide additional subgrade drainage. All unsatisfactory existing underdrain along the pavement edge was replaced. Numerous sections of this underdrain were found to be clogged with fine silt. Additional outlets were located to facilitate drainage and to prevent a recurrence of the drainage problem.

The median and shoulders were then reconstructed with aggregate to provide an adequate base for a bituminous paved shoulder capable of supporting traffic. After the pavement was repaired and cleaned of all patches and joint material, a leveling course was placed. The leveling course was either the standard bituminous wearing course or the binder course material used for bituminous surface course. The mix was dependent on the depth of application.

The roadway was then resurfaced in the usual bituminous paving sequence. After the shoulders and median were resurfaced with a bituminous course-binder material, they were sealed with 2 applications of asphalt cement aggregate.

Reconstruction of drainage structures to the revised roadway elevation and construction of additional drainage facilities were necessary as a result of this extensive repair work.

The Pennsylvania Department of Highways specifications were used as a general specification but were modified to meet Commission requirements. These modifications changed the previous Commission specifications by (a) requiring Class AC-2000 asphalt cement, which was similar to the Class A-1 previously used except that the designation reflects the viscousness of the asphalt; (b) increasing the stability of the binder course to 1,500 minimum; (c) decreasing the percentage voids allowable per total mix to 3-6 for binder course and 2-5 for wearing course; (d) increasing the percentage aggregate voids filled to 65-75 for binder course and 82-90 for wearing course; (e) providing for a more positive means of control to ensure a satisfactory mix (the compacted field density was changed from 95 percent of the laboratory compacted density to 95 percent of corresponding daily compacted specimen density); (f) requiring 4 hot bins for binder course material separation in lieu of the 3 previously specified to provide more consistency in the mix; (g) predrying slag aggregate and storing in covered supply areas so that the hot gases could escape and yet the aggregate would not be subject to an increase in moisture content; (h) regulating the temperature of the asphalt cement in the mix to yield a kinematic viscosity within the range of 280 and 150 centistokes; (i) requiring that paving machines be controlled electronically to maintain the desired slope and grade; and (j) operating tandem pavers within 150 ft of each other to prevent the formation of cold longitudinal joints.

With the exception of items e, f, g, and j, which were changed as a result of our experience, these changes were made to incorporate the latest asphalt technology into our overlay projects.

Although these repair methods may seem extensive, they enable the Turnpike Commission to provide a satisfactory roadway capable of handling the heavy traffic volumes and loadings common today on all major highways. Further, we feel that, in our situation where the comfort and protection of the user is considered of prime importance and where alternate Turnpike facilities are not available, the cost, although high, is fully justified especially when the extended roadway life is considered.

We have repaired 33 miles of the originally salvaged pavement to date, and we are currently salvaging 10 additional miles of concrete pavement with a bituminous overlay. The average cost per 12-ft lane-mile exclusive of the cost of bridge repair and traffic control for the last 43 miles repaired in this manner is $60,000.

## BRIDGE DECK PAVEMENT SALVAGE

Salvage of bridge deck pavement is probably the most serious of all bridge maintenance problems, and the Pennsylvania Turnpike Commission has experienced its share of problems in this area. The causes for deck deterioration are many and varied and have been discussed in many articles on the subject. Regardless of the cause, repairs must be made immediately if complete deck failure is to be prevented.

We have utilized several methods to correct bridge deck spalling failures. The treatment is dependent on the condition of the deck at the time repairs are made. Where possible, we use our own forces to make the necessary repairs. On major failures, however, we contract for the required corrective measures.

We have placed linseed oil treatment in several critical areas in an attempt to prevent and curtail deterioration caused by freezing and thawing or the use of de-icing chemicals. The results to date indicate that some beneficial effects are obtained. Generally, our maintenance forces correct spalled conditions by using the following procedure:

1. Saw a vertical edge around the limits of the crack;
2. Use a light chipping hammer to clean out all deteriorated concrete until sound concrete is exposed;
3. Remove all dust and chips with air, water, or brooms;
4. Apply a premixed mortar paste to the entire surface including the vertical face;
5. Place a metallic aggregate concrete in the hole while the paste is still wet or tacky and finish concrete in the normal manner; add stone in those areas where the holes extend 1 or 2 in. in depth; and
6. Cure with wet burlap for at least a 72-hour period.

The nonshrinking premixed mortar we are now using consists of 1 part iron aggregate, 2 parts cement, and 3 parts sand aggregate delivered to us in 100-lb bags. The nonshrink metallic aggregate is also delivered in 100-lb bags. Stone is added to the mix in those areas where the holes extend 1 or 2 in. in depth. The premixed materials are furnished by the concrete service company and the mortar is designated as C-S-C premixed shrink-proofer mortar. Embeco and Perma Cement have also been used in lieu of the C-S-C mortar and aggregate.

These patches prove to be excellent repairs when they are properly placed and cured. Maintenance repair longevity, however, like new construction is dependent on good workmanship.

As indicated earlier our major repair work is performed under contract, and we have had many bridges repaired by contract. Preparatory for, and incidental to, the resurfacing of bridge decks, the contractor is required to remove and dispose of all bituminous patch material and all loose and unsound concrete. Final cleaning, which is accomplished by air-blasting, is carried on immediately ahead of the tack coat operations. Immediately following the final cleaning of the bridge deck the contractor is required to apply a tack coat of Class F-3, Type 2, asphaltic emulsion. The rate of application is determined by the engineer on the basis of furnishing an asphaltic residue on the surface from 0.04 to 0.07 gal/sq yd.

Following the tack-coat operation, the surface of the bridge deck is brought to proper section by the placement of a scratch coat of bituminous surface course JA-1 material. The material is placed by means of a finishing machine over the full width of deck in a manner to fill all irregularities and to bring the surface just slightly above the normal surface of the deck's concrete wearing surface. Compaction of the material is made by a pneumatic-tired roller, and rolling is continued until all areas are thoroughly compacted. The bituminous binder course ID-2 utilizes an asphalt cement having a penetration range of 70 to 80 and a minimum specific gravity at 77 F of 1.010. Slag coarse aggregate is used for the binder course. The Marshall method is used to determine the plant formula and the mixture must meet the following Marshall stability test requirements:

| Requirement | Minimum |
|---|---|
| Stability | 1,500 |
| Flow value | 8-16 |
| Percentage voids, total mix | 4-6 |
| Percentage aggregate voids filled | 65-72 |

The bituminous surface course utilizes an asphaltic material with a 70 to 85 penetration. The mineral aggregate is usually a mixture of slag sand and snuff sand meeting the specifications of the Pennsylvania Department of Highways.

Results to date from using the methods described have been satisfactory.

# Evaluating and Resurfacing Old Pavements in Virginia

PAUL F. CECCHINI, Virginia Department of Highways

This paper reports on the methods of evaluating and resurfacing old highway pavements in Virginia. With traffic increasing at the rate of about 5 percent annually, and with 50,000 miles of roads in the state highway system, this maintenance function must have high priority if adequate safety and service are to be provided. The Virginia Department of Highways evaluates the road system by historical performance in areas such as safety, cost to maintain, relation of service to other roads, available funds, and the most economical methods of restoring roads to original condition. Types of resurfacing are based on results of experience gained by research and materials use and by availability of materials. Because the road system is so extensive and varies from heavy-volume Interstate routes to light-volume secondary roads, it is necessary that the approach to evaluating and resurfacing remain flexible to take advantage of all possible economies. The department believes it has developed a satisfactory method of evaluating and restoring old surfaces by using historical data, by using the judgment of the resident, district, and maintenance engineers in each case, and by having specialists in materials and research available for detailed investigations if necessary.

•OF FIRST PRIORITY in our resurfacing program is resurfacing roads that have been determined to be slick and have a predicted skid-resistant coefficient reading of less than 0.40. Although we do not have at the present time a complete log of all road surface coefficients, this program is under way by our Research Council. All skid-resistant coefficient readings are correlated to a predicted car reading at 40 mph.

Of second priority is the resurfacing of roads that are at accident-prone locations where the road surface may be in poor condition, have improper superelevation, not be sufficiently wide, lack deceleration lane, or have excessive dips.

After allocations for these priorities are made from the funds available, the following procedure is used.

The resident engineer reviews the need for resurfacing of the Interstate, primary, and secondary mileage under his jurisdiction and furnishes the district engineer his recommendations for the Interstate and primary highway systems. This recommendation is based on the visual condition of the surface and performance of the road in question. Information as to the road base and surface are available to him. The type and amount of the last treatment and surface maintenance costs for the road are also available for consideration. Recent instructions state that chip-seal treatments will generally not be placed on any Interstate or primary road (Class 1 or arterial) unless the previous treatment was a chip seal. After the recommendations are made to the district engineer, one of the maintenance division engineers reviews the requests with the district engineer and the resident engineer. The district materials engineer and others are called in when special conditions exist or a more thorough investigation of the road failure is determined necessary before a decision is made to resurface.

Paper sponsored by Committee on Salvaging Old Pavements by Resurfacing and presented at the 49th Annual Meeting.

Because we are endeavoring to improve our system of evaluation and determinations for resurfacing, we are looking at some of the studies currently under way in other states. Most current studies, with which we are familiar, use a combination of means to arrive at a figure that relates to need. In all cases so far, there is a provision for a judgment factor. In an interim report concerning the AASHO road test findings applied to flexible pavements in Virginia, N. K. Vaswani of the Virginia Highway Research Council indicated a correlation could be established by using the deflections of the road to determine the thickness of the overlay.

For the present, we feel that, by having the historical data on the road and by using the judgment of 3 engineers for general conditions, plus having our specialists in both the Materials Division and at the Research Council available for any detailed investigation, we have arrived at a very satisfactory method for evaluating and restoring old surfaces.

Unless something exists out of the ordinary, the retreatment program on the secondary system is left to the resident engineer with some efforts being made to review in the office his recommended course of action.

## TYPES OF TREATMENT

Realizing the need for new methods and materials in the resurfacing of highways, we have been developing different approaches to the control of bituminous plant-mixed material. Because bituminous paved roads in Virginia have traffic volumes ranging from a high in excess of 90,000 vehicles per day on some sections of the Interstate System to a low of 50 vpd on a secondary road, we have to allow for the maximum amount of flexibility in order to provide the necessary services within funds available.

Our first prerequisite for an overlay, be it chip seal, slurry seal, or plant mix, is that, if the traffic count is above 1,000 vpd, the surface material will be manufactured with a nonpolishing aggregate. This criterion has been modified slightly now since our Research Council, Materials Division, and Maintenance Division have been and are trying mixes using various percentages of a combination of polishing and nonpolishing aggregates, which so far are proving satisfactory. This has had a tendency to decrease the cost of overlays in areas of the state where nonpolishing aggregate material is not locally available. The dividing line of 1,000 vpd is not firm but is generally the maximum range for use of polish-susceptible aggregates in wearing courses.

Plant-mix overlays are not restricted to any particular road system; however, the decision to upgrade a secondary road surface and service is a decision to be made by the district engineer with concurrence by the Secondary Roads Division and with the recommendation of the Maintenance Division being considered.

On high-type primary and Interstate routes, the basic cause for resurfacing has been the failure of the surface to perform as a satisfactory wearing course and sealer of the base. When this occurs, we normally recommend and apply at 100 lb/sq yd our toughest and best performing mix, designated S-5 (Appendix C). This reestablishes the structural integrity of the base by sealing the surface while providing a suitable wearing surface.

On roads carrying traffic volumes of up to 5,000 vpd, we permit the use of plant mix designated S-4 (Appendix C). To correct surface failures, we apply this mix at approximately 100 lb/sq yd.

With the exception of the Interstate System, when a combination of surface failure and base distortion takes place, the rate of application is increased to the point of reestablishing a reasonable road section with a mix designated I-2 (Appendix C). On the Interstate System, we increase the rate of application of our S-5 mix.

With the I-2 mix for the past 6 years, we have been using a combination of polish-resistant coarse aggregate and polish-susceptible fine aggregate to obtain a skid-resistant surface in areas of the state that do not have locally available polish-resistant aggregates. Previous experienee has defined polish-resistant and polish-susceptible aggregates; however, in the very near future we expect to define the aggregates by chemical means, which would be more exact and allow for the use of some materials in selected instances that are considered unacceptable now.

We are using a combination of sand and No. 10 polish-resistant stone screenings for a tough plant-mix seal coat on roads that are essentially sound but need a new surface. This is being used on curb and gutter sections to a good advantage as well as on some high-type primary roads. We have successfully applied this mix at approximately 60 lb/sq yd on roads with a reasonably good cross section (Appendix D). In the area of the state where polish-resistant stone screenings are not available, we have designed a mix that is a combination of 50 percent sand and 50 percent stone screenings to be used on roads carrying traffic volumes up to 2,000 vpd (Appendix D). Based on experience and information available from various research projects, some by our own Council, it was decided to use this procedure to verify the skid-resistant quality of the mix during its life, which at the present time is expected to be from 6 to 10 years.

Local pit materials in combination with asphalt have been used and have produced a good plant-mixed seal on some of the low-type primary roads and some secondary roads.

Because a large percentage of all roads in Virginia are surface treated with either chip seals or plant mix, this is our area of greatest surface rehabilitation. You may recall I mentioned that our first priority for resurfacing is to cover sections of road that have been determined to be slippery. This is generally done using 30 lb/sq yd of plant-mixed treatment consisting of either silica sand or slag sand and asphalt, our designation S-1 or S-2 (Appendix C). These 2 mixes are used only to deslick an existing surface that is otherwise in good condition. The use of this mix is decreasing each year since we have been requiring a skid-resistant aggregate in all surface courses on roads with traffic volumes in excess of 1,000 vpd for the past 10 years.

In order not to become involved with requiring nonpolishing materials in a low-cost mix, it was decided that we could design a more economical plant-mixed material that would generally be utilized on roads carrying less than 750 vpd. This, however, does not rule out the possibility of using this material on roads carrying higher traffic volumes by the use of an additional polish-resistant surface course.

One major factor leading to the production of a low-cost plant mix would be utilizing a material that was readily available at most of the aggregate sources in the state. It was noted that, because of the extensive seal-treatment program in the state, most plant mix would be utilized on roads needing additional strengthening. The type of treatment normally used on roads of this nature consists of mixed-in-place or penetration-surface treatments. After conducting tests on random stone samples in the laboratory, it became evident that a product equal to these surface treatments could be obtained by using a ¾-in. crusher-run material.

The next step was to advise everyone that, by relaxing certain controls, there was a calculated risk of getting what may appear to be a substandard material. However, these odds had to be weighed against the savings that could be effected by virtue of the fewer controls.

The Materials Division was requested to determine what we could expect from various aggregate producers should we proceed in this direction. Samples of No. 26 dense-graded aggregate were obtained from 17 aggregate suppliers and represented materials available in all of the 8 districts. To establish the qualities a crusher-run mix would have, our Bituminous Section proceeded conducting Marshall design experiments in the laboratory with generally good results (Appendix E).

The results from this original testing showed to our satisfaction that we could expect a high percentage of good mixes that should provide good service. It was decided to draft a specification that would allow for the wide variation in material and still be reasonably certain of obtaining a satisfactory product.

Since 1958 the department has an established policy of providing for the resealing with chip seals of secondary surface-treated roads every 5 years. Since it went into effect, we have not had any major spring breakups. This is partly attributable to this policy. In addition, regular surface maintenance costs have continued downward. As a side benefit, we are able to use our forces for other maintenance activities and services. Chip-seal treatments on the secondary system amount to about 95 percent of all treatments of this type in Virginia. For the fiscal year 1969-70, a total of 17 million gal of bituminous material and 650 thousand tons of cover material were awarded to contract for chip-sealing approximately 4,500 miles of road.

The decision on which roads in the secondary system are to receive a chip seal rests entirely with the resident engineer. His determinations are weighed against the general policy, road conditions, and funds available. Essentially maintenance is considered first in the preparation of secondary county budgets; therefore, there is seldom any problem in providing for the necessary treatments.

Seal treatments to be applied to the primary system are generally restricted to low-class primary routes but are occasionally applied to high-type primary roads when there is a need for a seal treatment to maintain the road for a short period of time until more funds are available to place a plant-mix surface or in some instances until the road is reconstructed. It is in this area that the district engineer with the concurrence of the maintenance engineer has to make the final decision. This would be an instance when, in addition to the road condition itself, the priority of funds available and the overall highway program will dictate the treatment.

Slurry seals have been used in Virginia for approximately 4 years. Each year we have expanded their use, particularly in urban areas on subdivision streets, which are classed as secondary roads. This past year slurry seals were placed on some high-speed primary roads in the 2,000-vpd category and are proving to be satisfactory. The cost of this type of seal has been consistently about 20 cents/sq yd. The decision to use a slurry seal instead of a chip seal on secondary roads develops from a recommendation of the resident engineer with concurrence of the district engineer and the secondary roads engineer. This type of treatment has been of tremendous help, particularly in our program of resealing in subdivisions where some roads with higher traffic volumes have a plant-mix surface, and public reaction to chip seals has caused us to change to slurry seal. This and other benefits as described by available literature have to be weighed against the cost, which is somewhat higher. During this current year, approximately 1.3 million sq yd of slurry-seal treatment were applied. With the advent of a rapid-curing system, it is anticipated that more of this type of material will be used, provided the cost remains competitive. It is the recommendation of the district engineer with the concurrence of the maintenance engineer that determines whether a slurry-seal treatment will be used on the primary system of highways.

We have found in Virginia that, to do a good job of overlaying portland cement concrete pavement, we must clean and seal all joints in the pavement prior to placing an overlay. This has a tendency to reduce the amount of joint blowups. We have decided after considerable experimenting that all present methods suggested, such as sanding and paper covering of joints prior to overlaying, do not reduce the chance of reflective cracks showing through. We have been accomplishing more mudjacking and concrete patching repair work to existing concrete pavement in order to maintain the pavement as a concrete road. Recently, we have overlaid concrete pavement when pavement widening was being done in conjunction with the overlay work. On most of the roads where this is done, the concrete pavement is anywhere from 20 to 30 years old and badly cracked and slightly distorted. We have not considered seriously the complete breaking up of the pavement before overlaying because in most cases the roads involved are commuter roads with a very light volume of heavy truck traffic; however, any short section that needs more attention is corrected by removal or additional build-up, depending on which is the more economical and the safest to perform under traffic conditions at the time.

Because of the urbanization taking place in Virginia and the ever-increasing traffic volumes and loads on the roads, we are confronted with roads that need more attention than just spot-patching and covering with a thin overlay. In some of the earlier subdivisions, it is necessary to undertake corrective action that will not raise the elevation of the road surface because curbs and gutters are in place. In these instances, it was decided to scarify and remove any excess material from the roadway and then add either hydrated lime or portland cement in amounts determined by tests to be adequate, remix, compact, and seal. The seal serves as a curing membrane. Then this is covered with approximately 125 lb/sq yd of bituminous plant mix. This same procedure is used on other types of roads where it is determined that rehabilitating the base will justify the expenditure of funds and be the most economical method of repair.

## SUMMARY

The Virginia Department of Highways essentially evaluates the road system by historical performance including safety, cost to maintain, relation of service to other roads, money available, and most economical method to restore the road to original condition. The final decision is the result of engineering judgment from at least 3 levels in the department's organization. The type of resurfacing is based on results of experience gained by research, material usage, and material availability. Because the road system in Virginia varies from the Interstate System to the secondary system (farm-to-market roads), it is necessary that the approach to evaluating and resurfacing remain flexible to take advantage of all possible economies to provide for a safe, usable facility.

## ACKNOWLEDGMENTS

The author acknowledges the comments, suggestions, and data offered by other highway engineers in developing this paper, particularly W. S. G. Britton, J. V. Clarke, J. H. Dillard, R. V. Fielding, C. S. Hughes, and R. S. Thomas.

# *Appendix A*

## FINANCING

To completely understand the reasoning of our approach to updating road surfaces, you first have to understand our method of financing, which is a little different for each of our 4 systems of roads: Interstate, primary, urban, and secondary.

The Maintenance Division prepares preliminary estimates for anticipated maintenance needs based on personal observations of all of the field engineers and those in the Maintenance Division for the Interstate and primary systems of highways and presents them to the director of operations with the request that funds in the amount requested be made available for the next fiscal year. When the Highway Commission decides on a final figure, an allocation is made for maintenance on these 2 systems of highways as a lump-sum allocation.

The urban system (within incorporated limits of cities and some towns) receives funds on a per-mile basis as set forth by the Legislature for the maintenance of roads and streets within the corporate jurisdiction.

Our secondary system funds are allocated by lump sum for the maintenance and construction of all secondary roads over which the department has control. The amount is determined by formula as arranged by the acts of the Legislature.

More specifically, the funding of maintenance replacement activities, one of which is the restoration of existing road surfaces by resurfacing, is provided as follows:

1. Interstate System—All Interstate maintenance funds are pooled. From this total, funds for fixed expenditures are provided for in each district, then anticipated maintenance replacement funds are set aside based on estimates for the amount of work needed. The remainder of the maintenance funds are allocated to the district on the basis of vehicular traveled-miles and lane-miles of road.
2. Primary system—Maintenance funds received are for all practical purposes distributed to the districts on a factor basis in a lump-sum amount for further distribution to the counties by the district.
3. Urban system—Funding is handled by the local governments.
4. Secondary system—The distribution to each county is by formula and on a lump-sum basis. The resident engineer has instructions in preparing his budget to provide first for the necessary ordinary maintenance funds, then for maintenance replacement funds, and then for construction funds. Once funds are allocated to a county, they remain allocated. All surplus or deficits are carried forward to be either redistributed

or financed from the following year's funds. (The secondary system of highways includes farm-to-market roads, subdivision streets, and streets in towns of less than 3,500 population that are not otherwise included in the primary system.) At the time the state was obtaining control over these roads, 2 counties chose not to come under state control. In these 2 counties, the secondary system is still constructed and maintained by the county.

## *Appendix B*

### ORGANIZATION

A brief discussion of the Virginia Department of Highways' organization might help explain some of the reasons behind our operational policies.

The Highway Commission is composed of 8 members appointed by the governor representing each of the 8 districts of the state.

The commissioner is appointed by the governor and serves on a full-time basis as head of the department and chairman of the commission. The chief engineer, who is also the deputy commissioner, is the head administrative officer of the department. Directly under him are 4 directors, each supervising the functions of their assigned divisions. The abbreviated organizational chart shown in Figure 1 may help explain the functional levels of the department.

The district is the highest operational unit of the department and generally oversees the residency's operations on construction and maintenance.

The residency unit, headed by the resident engineer, is responsible for the proper execution of department policies and procedures for the areas under its jurisdiction

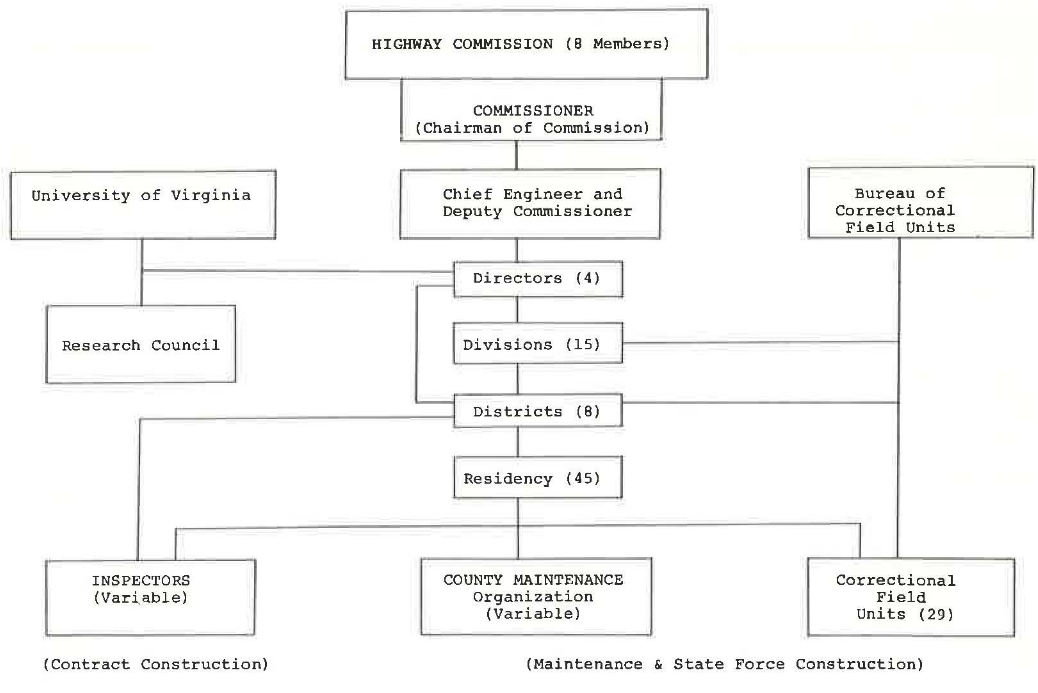


Figure 1. Organization of the Virginia Department of Highways.

as they relate to maintenance and construction. This may vary from 1 county to as many as 4 counties. The resident engineer usually has at least 1 assistant, a residency maintenance supervisor, and a project engineer to directly oversee both maintenance and construction activities.

# *Appendix C*

## STANDARD SPECIFICATIONS

### Type S-1 Bituminous Concrete (Deslicking Mix)

Type S-1 bituminous concrete shall consist of sand and asphalt cement AP-3, unless otherwise specified. A heat stable additive shall be added to the asphalt cement prior to introduction into the mix. The mix shall be proportioned in accordance with Table 1.

The sand shall have a minimum sand equivalent value of 70, as determined by the AASHO Designation T 176, or have a proven performance record of no service failures, and shall have a minimum silica ($SiO_2$) content of 95 percent.

A heat stable additive shall be added as specified by the engineer. This material shall meet the following requirements:

1. General requirements—The material shall contain no ingredient harmful to the bituminous material or to the operator and shall not appreciably alter the specified characteristics of the bituminous material when added in the recommended proportions. It shall be capable of thorough dispersion in the bituminous material in storage indefinitely at temperatures normally encountered without detrimentally affecting the bituminous material, or losing its effectiveness as an asphalt antistripping compound and without any discernible settlement or stratification.
2. Acceptance test—The material shall be subjected to the following test to determine acceptance: One hundred grams of asphalt cement AP-3, treated with the heat stable additive at the manufacturer's recommended percentage, not to exceed 1.0 percent, shall be placed in a clean container and heated to 275 F. The container shall be sealed securely and placed in an oven that will hold this temperature for 96 hours. The sample shall then be removed from the oven and stirred thoroughly. Upon removal from the oven, 28.5 grams of asphalt so treated shall be mixed with 271.5 grams of the fine aggregate to make a total mix of 300 grams. After complete coating, the mixture shall be placed in boiling water and boiling continued for 10 minutes. The water shall then be drained from the mixture and the mixture removed and placed on a paper towel. After 12 hours the sand grains shall maintain a glossy black appearance with no signs of stripping. Combinations of sand and asphalt plus a heat additive failing to meet these requirements will not be acceptable.

### Type S-2 Bituminous Concrete (Deslicking Mix)

Type S-2 bituminous concrete shall consist of slag, heat stable asphalt additive, combined with asphalt cement AP-3, unless otherwise specified, and be in accordance with Table 1. Type S-2 bituminous concrete shall conform to the requirements of Section 212.10 except as otherwise specified.

### Type S-3 Bituminous Concrete

Type S-3 bituminous concrete shall consist of natural sand, granite, slag, gravel, or gravel or granite screenings, or a combination thereof, combined with asphalt cement AP-1, unless otherwise specified, and be in accordance with Table 1.

TABLE 1
BITUMINOUS CONCRETE MIXTURES

| Type | Percentage by Weight Passing Square Mesh Sieves* | | | | | | | | | | | Per Cent Bitumen | Mix Temperature (At Plant) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 2 | 1½ | 1 | 3/4 | 1/2 | 3/8 | No. 4 | No. 8 | No. 30 | No. 50 | No. 100 | No. 200 | | |
| S-1 | - | - | - | - | - | - | 100 | 95-100 | 50-95 | 25-65 | 8-25 | 0-8 | 8.5-10.5 | 225-300°F |
| S-2 | - | - | - | - | - | 100 | 95-100 | 60-85 | 20-40 | 10-30 | 8-25 | 2-10 | 9.5-12.0 | 225-300°F |
| S-3 | - | - | - | - | - | 100 | 90-100 | 70-95 | 25-55 | 15-35 | 6-22 | 2-12 | 6.5-10.5 | 200-240°F |
| S-4 | - | - | - | - | 100 | 90-100 | 75-90 | 60-80 | 25-45 | 10-30 | 4-14 | 2-10 | 5.5-9.5 | 225-300°F |
| MS-4 | - | - | - | - | 100 | - | - | 60-80 | 25-45 | 10-30 | - | 2-10 | 5.5-9.5 | 225-300°F |
| S-5 | - | - | - | - | 100 | 80-100 | 50-70 | 35-55 | 15-30 | 7-22 | 3-15 | 2-10 | 5.0-8.5 | 225-300°F |
| MS-5 | - | - | - | - | 100 | - | - | 35-55 | 15-30 | - | - | 2-10 | 5.0-8.5 | 225-300°F |
| I-1 | - | - | 100 | 90-100 | - | 85-100 | 75-100 | 60-95 | 25-60 | 12-35 | 3-17 | 2-12 | 5.0-7.5 | 225-300°F |
| I-2 | - | - | 100 | 95-100 | - | 60-80 | 40-60 | 25-45 | - | 5-14 | 2-9 | - | 4.5-8.0 | 225-300°F |
| MI-2 | - | - | 100 | - | - | - | 40-60 | - | - | 5-14 | 2-9 | - | 4.5-8.0 | 225-300°F |
| B-1 | - | - | 100 | 90-100 | - | - | 70-100 | 55-95 | 25-65 | 12-40 | 1-20 | 0-10 | 3.0-6.5 | 225-300°F |
| B-2 | - | 100 | - | 50-75 | - | - | 20-35 | 15-25 | - | - | - | 0-5 | 4.0-6.0 | 200-240°F |
| B-3 | - | 100 | - | 72-87 | - | - | 35-50 | 28-38 | - | - | - | 2-6 | 4.0-7.0 | 225-300°F |
| B-4 | 100 | 90-100 | - | 70-100 | - | - | 35-80 | 25-70 | 15-45 | - | - | 3-15 | 2.5-4.0 | 225-300°F |
| P-1 | - | - | - | - | - | 100 | 85-100 | 65-95 | 25-55 | 12-38 | 4-26 | 2-12 | 6.5-9.5 | 125-175°F |
| P-2 | - | - | - | - | 100 | 80-100 | 50-70 | 35-55 | 15-30 | 6-20 | 3-14 | 2-10 | 6.5-8.5 | 125-175°F |
| P-3 | - | - | - | 100 | - | 60-80 | 35-55 | 20-35 | - | - | - | 0-5 | 5.5-7.5 | 125-175°F |

* In inches, except where otherwise indicated. Numbered sieves are those of the United States Standard Sieve Series.

## Type S-4 Bituminous Concrete

Type S-4 bituminous concrete shall consist of natural sand, granite, slag, gravel, or gravel or granite screenings, or a combination thereof, combined with asphalt cement AP-3, unless otherwise specified, and be in accordance with Table 1.

The combination of aggregate and asphalt shall have a minimum Marshall stability of 1,000 lb at 140 F. If this value cannot be obtained, the addition of mineral filler conforming to Section 201 in an amount not to exceed 5 percent of the completed mixture will be permitted in order to obtain this minimum stability. If the mixture still lacks stability, another source of aggregate will be necessary.

## Type S-5 Bituminous Concrete

Type S-5 bituminous concrete shall consist of crushed stone, crushed slag, or crushed gravel and sand, or slag or stone screenings, or a combination thereof, combined with asphalt cement AP-3, unless otherwise specified, and be in accordance with Table 1.

The combination of aggregate and asphalt shall have a minimum Marshall stability of 1,450 lb at 140 F and a flow of between 0.05 and 0.20 in. If this value cannot be obtained, the addition of mineral filler conforming to Section 201 in an amount not to exceed 5 percent of the completed mixture will be permitted in order to obtain this minimum stability. If the mixture still lacks stability, another source of aggregate will be necessary.

Whenever the amount of aggregate passing the No. 200 sieve exceeds 5 percent, a minimum of 15 percent sand (minimum grade B) may be required to be added to the mix.

## Type I-1 Bituminous Concrete (Local Material)

Type I-1 bituminous concrete shall consist of local pit material combined with asphalt cement AP-3, unless otherwise specified, and be in accordance with Table 1.

The combination of local pit material and asphalt shall have a minimum Marshall stability of 500 lb at 140 F. If this value cannot be obtained with the local pit material, the addition of gravel, slag, or stone screenings will be permitted provided the gradation of the final mix is within the limitations provided in Table 1. If the stability value still cannot be obtained, mineral filler will be permitted in order to obtain this minimum stability. If the mixture still lacks stability, another source of local pit material will be necessary.

Type I-2 Bituminous Concrete

Type I-2 bituminous concrete shall consist of crushed stone, crushed slag, or crushed gravel, coarse aggregate and sand, or stone or gravel screenings, or a combination thereof, combined with asphalt cement AP-3, unless otherwise specified, and be in accordance with Table 1.

Type B-1 Bituminous Concrete (Local Material)

Type B-1 bituminous concrete shall consist of local pit material combined with asphalt cement AP-3, unless otherwise specified, and be in accordance with Table 1.

The combination of local pit material and asphalt shall have a minimum Marshall stability of 400 lb at 140 F. If this value cannot be obtained with the local pit material, the addition of gravel, slag, or stone screenings will be permitted provided the gradation of the final mix is within the limitations provided in Table 1. If the stability value still cannot be obtained, mineral filler in an amount not to exceed 5 percent of the completed mixture will be permitted in order to obtain this minimum stability. If the mixture still lacks stability, another source of local pit material will be necessary.

Type B-2 Bituminous Concrete

Type B-2 bituminous concrete shall consist of crushed stone, crushed slag, or crushed gravel, coarse aggregate and sand, slag, or stone or gravel screenings, or a combination thereof, combined with asphalt cement AP-1, unless otherwise specified, and be in accordance with Table 1.

Type B-3 Bituminous Concrete

Type B-3 bituminous concrete shall consist of crushed stone, crushed slag, or crushed gravel, coarse aggregate and sand or slag, or stone or gravel screenings, or a combination thereof, combined with asphalt cement AP-3, unless otherwise specified, and be in accordance with Table 1.

Type B-4 Bituminous Concrete Subbase Course

Type B-4 bituminous concrete subbase course shall consist of natural sand, granite, slag, gravel, or gravel or granite screenings, or a combination thereof, combined with asphalt cement AP-3, unless otherwise specified, and be in accordance with Table 1. The aggregate shall conform to the requirements of Section 209.03.

Type P Bituminous Concrete

Type P bituminous concrete mixes shall consist of crushed stone, crushed slag, or crushed gravel, coarse aggregate and sand, or slag or stone screenings, or a combination thereof, combined with asphalt MC-800, unless otherwise specified, and be in accordance with Table 1.

All other requirements of Section 212 shall be adhered to except the following: (a) after the aggregate has been properly dried, it shall be allowed to cool until the temperature is within a range of 125 to 175 F; and (b) at the time of mixing, the temperature of the asphalt shall be between 125 and 175 F.

# *Appendix D*

## SPECIAL PLANT-MIX SPECIFICATIONS

Maintenance Mixture MI-3

The maintenance mixture shall conform to the requirements of Section 212 of the 1966 edition of the Road and Bridge Specifications as amended in the following:

Aggregate—(a) Aggregate shall conform to the requirements of Section 206, Crusher Run Aggregate, except that the first paragraph of Section 206.01 is amended to read:

> Crusher run aggregate shall consist of crushed stone, slag, or crushed gravel. It shall be the complete product of a crusher, essentially free of overburden and only oversize removed to conform to gradings as specified.

(b) All aggregate shall meet the gradation requirement of dense-graded aggregate size No. 26 unless otherwise designated. (c) Section 212.02(d) and (e) are deleted from this contract. (d) Section 212.03 is amended so that the job-mix formula is to indicate grading band for No. 26 aggregate. Gradation tolerances for aggregate will not apply.

Bitumen—(a) The optimum bitumen content for this mixture shall be determined by using Marshall design procedures and expressed as percent, by weight, of the total mixture. (b) The type of bitumen shall be AP-3 unless otherwise designated. (c) Section 212.03 is amended so that the asphalt content tolerances for this mixture shall be ±0.7 percent.

Mixture—(a) Proportioning the combination of aggregate and asphalt shall have a minimum Marshall stability of 700 lb at 140 F. (b) Marshall density results of the mix shall be between 90 and 98 percent of theoretical density unless otherwise waived. (c) Temperature of the mixture at the plant shall be between 225 and 300 F.

Special Mixture MS-7

The special mixture designated in the proposal shall conform to the requirements of Section 212.13 (Type S-4) of the 1966 Specifications as amended in the following:

Fine Aggregate—The use of fine aggregate from local deposits will be permitted in the special mixture provided such source of material is approved by the department.

Bitumen Content—The bitumen content for the special mixture shall be 7.0 to 9.0 percent, by weight, of the total mixture.

Mineral Filler—The quantity of mineral filler used in the special mixture shall be 4.0 to 8.0 percent, by weight, of the total mixture.

Mixture—Proportioning the combination of aggregate, asphalt, and mineral filler shall have a minimum Marshall stability of 250 lb at 140 F when used on secondary routes and 300 lb at 140 F when used on primary routes.

Screening Mixture MS-8

The screening mixture shall conform to the requirements of Section 212 of the 1966 Specifications as amended in the following:

Aggregate—(a) Aggregate for this mixture shall consist of approximately 50 percent No. 10 screenings and approximately 50 percent of a nonpolishing sand. (b) Stone screening shall not exceed 55 percent of the aggregate in the mixture. (c) Gradation tolerances for aggregate will not apply.

Bitumen—(a) The optimum bitumen content for this mixture shall be determined by using Marshall design procedures and expressed as percent, by weight, of the total mixture. (b) The type of bitumen shall be AP-3 unless otherwise designated.

Mixture—(a) Proportioning the combination of aggregate and asphalt shall have a minimum Marshall stability of 700 lb at 140 F. (b) Marshall density results of the mix shall be from 85 to 98 percent of theoretical density unless otherwise waived. (c) Temperature of the mixture at the plant shall be between 225 and 300 F.

Screening Mixture MS-9

The screening mixture shall conform to the requirements of Section 212 of the 1966 Specifications as amended in the following:

Aggregate—(a) Aggregate for this mixture shall consist of a combination of polish-resistant No. 10 stone screenings, polish-resistant sand, and a mineral filler, if required. (b) Gradation tolerances for aggregate will not apply.

Bitumen—(a) The optimum bitumen content for this mixture shall be determined by using Marshall design procedures and expressed as percent, by weight, of the total mixture. (b) The type of bitumen shall be AP-3. (c) Heat stable additive shall be added if required by the engineer.

Mixture—(a) Marshall density results of the mix shall be from 90 to 98 percent of theoretical density. (b) Proportioning of the aggregate and bitumen shall result in the mixture having a minimum Marshall stability of 700 lb at 140 F. (c) Temperature of the mixture at the plant shall be between 225 and 300 F.

Expanded Shale Mixture MS-10

The expanded shale (known at local sources as clinchlite, solite, and weblite) and screening mixture shall conform to the requirements of Section 212 of the 1966 Specifications as amended in the following:

Aggregate—(a) Aggregate for this mixture shall consist of a combination of No. 10 limestone screenings not to exceed 75 percent by weight of the total aggregate. The remainder of the aggregate shall consist of expanded shale meeting the gradation requirements for No. 68 open-graded coarse aggregates. (b) No more than 8 percent by weight of the total aggregate retained on the No. 4 sieve shall be limestone screenings.

Bitumen—(a) The optimum bitumen content for this mixture shall be determined by using Marshall design procedures and expressed as percent, by weight, of the total mixture. (b) The type of bitumen shall be AP-3. (c) Heat stable additive shall be added if required by the engineer.

Mixture—(a) Marshall density results of the mix shall be from 90 to 98 percent of theoretical density. (b) Proportioning of the aggregate and bitumen shall result in the mixture having a minimum Marshall stability of 700 lb at 140 F. (c) Temperature of the mixture at the plant shall be between 225 and 300 F.

Modified Mix

This mixture shall conform to the requirements of Section 212 of the 1966 Specifications as amended in the following:

Aggregate—(a) Maximum top size of aggregate used shall not exceed two-thirds of the thickness of the layer being applied. (b) No more than 7 percent by weight of the total aggregate shall pass the No. 200 sieve. (c) The sand equivalent of the material shall not be below 25.

Bitumen—(a) The optimum bitumen content for this mixture shall be determined by using Marshall design procedures and expressed as percent, by weight, of the total mixture. (b) The type of bitumen shall be AP-3.

Mixture—(a) Marshall density results of the mix shall be from 90 to 98 percent of theoretical density. (b) Proportioning of the aggregate and bitumen shall result in the mixture having a minimum Marshall stability of 700 lb at 140 F. (c) Temperature of the mixture at the plant shall be between 225 and 300 F.

# *Appendix E*

## NO. 26 DENSE-GRADED AGGREGATE SAMPLES

Samples of No. 26 dense-graded aggregate were obtained from 17 suppliers and represented materials available in all of the 8 districts. The gradation and test results of these samples are given in Tables 2 and 3.

TABLE 2

GRADATION OF 17 SAMPLES OF NO. 26 AGGREGATE

| | SAMPLE NUMBER | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Screen | 1 | 2 | 3 | 4 | 5 | 6 | 7 | * 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | Range Hi - Low |
| 3/4 | 100 | 100 | 100 | 100 | 100 | 100 | 100 | 100 | 100 | 100 | 100 | 100 | 100 | 100 | 100 | 100 | 100 | 100-100 |
| 1/2 | 82 | 82 | 83 | 91 | 86 | 88 | 67 | 34 | 85 | 88 | 70 | 80 | 92 | 61 | 81 | 82 | 95 | 61-95 |
| 3/8 | 69 | 68 | 71 | 80 | 76 | 76 | 49 | 18 | 73 | 73 | 53 | 63 | 63 | 46 | 62 | 62 | 83 | 46-83 |
| #4 | 46 | 47 | 48 | 65 | 52 | 55 | 37 | 5 | 57 | 49 | 42 | 43 | 30 | 27 | 31 | 34 | 55 | 27-65 |
| #8 | 30 | 32 | 33 | 27 | 35 | 35 | 24 | 3 | 44 | 35 | 29 | 34 | 20 | 17 | 19 | 24 | 36 | 17-36 |
| #30 | 12 | 13 | 12 | 6 | 14 | 11 | 13 | 2 | 26 | 16 | 14 | 24 | 9 | 7 | 10 | 9 | 13 | 6-26 |
| #50 | 8 | 9 | 7 | 4 | 11 | 7 | 11 | 2 | 20 | 11 | 9 | 17 | 7 | 4 | 8 | 5 | 8 | 4-20 |
| #100 | 5 | 6 | 5 | 2 | 8 | 4 | 8 | 2 | 12 | 7 | 5 | 8 | 4 | 3 | 6 | 3 | 4 | 2-12 |
| #200 | 3 | 4 | 3 | 2 | 6 | 3 | 5 | 1 | 6 | 4 | 2 | 3 | 3 | 2 | 4 | 2 | 3 | 1-6 |

* Rejected - DID NOT MEET GRADATION REQUIREMENTS

TABLE 3

RESULTS OF TESTS ON 17 SAMPLES OF NO. 26 AGGREGATE

| SAMPLE NO. | ASPHALT | MARSHALL | FLOW | DENSITY | : | SAMPLE NO. | ASPHALT | MARSHALL | FLOW | DENSITY |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 4.0 | 2463 | 11 | 96.3 | : | 9 | 4.0 | 1705 | 9 | 92.1 |
| | 5.0 | 1874 | 10 | 97.0 | : | | 5.0 | 2251 | 10 | 95.7 |
| | 6.0 | 1877 | 11 | 99.2 | : | | 6.0 | 2718 | 11 | 97.8 |
| 2 | 4.0 | 1953 | 9 | 94.3 | : | 10 | 4.0 | 2015 | 9 | 91.1 |
| | 5.0 | 2067 | 11 | 99.1 | : | | 5.0 | 2505 | 10 | 94.6 |
| | 6.0 | 1890 | 11 | 99.5 | : | | 6.0 | 2260 | 11 | 97.4 |
| 3 | 4.0 | 2099 | 11 | 94.1 | : | 11 | 4.0 | 1483 | 9 | 92.7 |
| | 5.0 | 1973 | 11 | 95.2 | : | | 5.0 | 1875 | 11 | 95.1 |
| | 6.0 | 1704 | 10 | 96.9 | : | | 6.0 | 2046 | 11 | 98.3 |
| *4 | 4.0 | 987 | 12 | 92.8 | : | 12 | 4.0 | 2255 | 10 | 95.2 |
| | 5.0 | 978 | 12 | 92.1 | : | | 4.5 | 2935 | 10 | 97.6 |
| | 6.0 | 574 | 13 | 91.8 | : | | 5.0 | 2535 | 10 | 98.8 |
| | | | | | : | | 5.5 | 2350 | 10 | 100.0 |
| **5 | 6.0 | 2081 | 11 | 97.0 | : | 13 | 4.0 | 1667 | 11 | 95.7 |
| | 7.0 | 1326 | 24 | 97.1 | : | | 5.0 | 1587 | 11 | 94.7 |
| | 7.5 | 1174 | 35 | 97.9 | : | | 6.0 | 1257 | 10 | 95.0 |
| 6 | 4.0 | 1640 | 10 | 92.0 | : | 14 | 4.0 | 1418 | 17 | 92.5 |
| | 5.0 | 1676 | 11 | 92.2 | : | | 5.0 | 2060 | 16 | 91.5 |
| | 6.0 | 1666 | 10 | 95.1 | : | | 6.0 | 1567 | 17 | 92.5 |
| 7 | 4.0 | 2220 | 10 | 96.5 | : | 15 | 4.0 | 1865 | 9 | 94.5 |
| | 5.0 | 2461 | 11 | 98.5 | : | | 5.0 | 2291 | 10 | 96.9 |
| | 6.0 | 2583 | 15 | 100.0 | : | | 6.0 | 1896 | 9 | 97.6 |
| 8 | Rejected, did not meet #26 dense graded aggregate gradation requirements. | | | | : | 16 | 4.0 | 1579 | 10 | 93.5 |
| | | | | | : | | 5.0 | 1603 | 11 | 95.0 |
| | | | | | : | | 6.0 | 1690 | 10 | 96.7 |
| * Material Not Cohesive | | | | | : | 17 | 4.0 | 1436 | 12 | 92.1 |
| ** Poor Coating of Aggregate at 4 & 5% Asphalt - about 80% coated at 6.0% Asphalt | | | | | : | | 5.0 | 1719 | 12 | 93.3 |
| | | | | | : | | 6.0 | 1513 | 12 | 95.4 |

# The New Jersey Turnpike Approach to Salvaging Old Pavements

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The New Jersey Turnpike was opened in January 1952. The road currently consists of 32 miles of 4-lane roadway and 98 miles of 6-lane roadway. In 1970 another widening will be completed that will expand the northernmost 30 miles to 12 lanes (4 separate 3-lane roadways). Resurfacing of existing pavements began in 1961 and has continued to date with approximately 70 percent of the old pavements having been salvaged through resurfacing. Original pavement was 4½ in. of asphaltic concrete over 7½ in. of penetrated macadam. This paper covers the Authority's approach to the inspection of old pavements, as well as the methods employed to design and construct resurfaced pavement sections. The specific problems engendered because of high traffic densities (more than 78 million vehicles in 1968) as well as costs associated with the Authority's approach are discussed.

• THE ORIGINAL TURNPIKE was 118 miles long, extending from the Delaware Memorial Bridge in the south near Wilmington, Delaware, in a generally northeasterly direction to State Route 46 in the north.

The first construction contracts were awarded in December 1949. The project was completed in January 1952. The original turnpike was 4 lanes wide for a distance of 92 miles, and the rest was 6 lanes wide. In 1956 a major widening was completed that had the effect of widening to 6 lanes from Woodbridge south, a distance of 60 miles. In 1956 the Newark Bay-Hudson County extension was completed. This extension connects the Newark area with the Holland Tunnel and downtown New York; it is 8 miles long and includes a major bridge over Newark Bay. Another major extension was also opened in 1956 from the Bordentown area to Florence, New Jersey, and the Pennsylvania Turnpike. The Delaware River Bridge at Florence is jointly owned by the 2 turnpike authorities. The Pennsylvania extension is also 6 lanes wide and is 6 miles long.

At the present time we are undertaking the greatest expansion project yet: widening the northern 30 miles from 6 lanes to 12 lanes. This will be a dual-dual configuration with 4 separate roadways. A portion of this, however, is an entirely new alignment on the west side of the Hackensack River, through virgin tidal meadows.

Our existing pavements are all of flexible design and of asphaltic concrete construction. The standard section is 4½ in. of asphaltic concrete over 7½ in. of penetrated macadam over 6½ in. of compacted subbase. The road is constructed through areas of the Jersey meadows, requiring extensive muck excavation, and through areas of clay deposits. Post-construction settlements in the meadow areas have always been a problem, but during the original construction the economic factors involved dictated that time had to take precedence over consolidation in some areas. I should point out, however, that maintenance costs in these areas have not been as great as were originally anticipated. No replacements of original pavements were required.

The first resurfacing contracts were let during 1961. Since that time 152 roadway miles have been resurfaced (a roadway mile being 1 mile in either direction, either 2

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or 3 lanes wide). Eighty-five percent of the northern half of the turnpike has been resurfaced. All areas where the original construction was through meadowland have been resurfaced.

Our revenue vehicle count for 1969 exceeded 79 million vehicles.

Our approach to resurfacing may generally be divided into 3 parts: inspection of existing roadway, resurfacing design, and construction.

## INSPECTION OF EXISTING ROADWAY

From these inspections, the determinations as to what areas will be resurfaced are made. There are 3 basic inspections involved:

1. An annual inspection of roadway is made concurrently by the Maintenance and Engineering Departments during the fall. At this time the condition of all roadway is noted and compared with previous inspections. Detailed, highly technical inspections have not been found to be necessary. Strip maps of the turnpike, made for the purpose, are utilized. Extensive notes are made directly on the maps. Notes are compared with observations from previous years. Supplemental photographs are often used. One obvious result is that the rate of deterioration can be established and conclusions drawn as to priorities to be assigned to various areas. We attempt to establish requirements for the next 5-year period. There is a great deal of flexibility, however, with the 5-year evaluation, because we are only locked into one year at a time as far as budget considerations are involved. What must be done this year and what may be acceptable for another year are questions that require a great deal of soul-searching, as anyone connected with pavement maintenance well knows.
2. In addition to this annual inspection, there is continual reporting throughout the year by maintenance district foremen. Any problems with roadways or bridge decks are reported as they occur. There are 7 maintenance districts for the turnpike, and the district foremen are each responsible for the maintenance of their particular section of roadway. Other agencies, as well, contribute to the overall knowledge of road conditions. The State Police, Traffic Engineering Department, and general consultant personnel are extremely familiar with and constantly traveling the turnpike.
3. Reports are prepared by the general consultants, engaged by the Turnpike Authority, for all bridges. Although these reports are aimed primarily at bridges from the structural aspect, they do give valuable information regarding the necessity for overlaying decks. It has been our policy to overlay concrete bridge decks with a special asphaltic bridge deck mix containing asbestos fibers. Although this particular problem is not a part of this presentation, I mention it because it is usually accomplished as a part of a resurfacing contract.

With all the evidence now in, it is necessary to make the determination of where best to spend available money.

Resurfacing, for the past 2 years, has been limited to areas south of the current widening project. The intricate staging of the widening contracts and the necessity to maintain traffic on all existing roadways dictated that resurfacing not be undertaken in the construction areas. We intend, however, beginning in 1970, to concentrate resurfacing activities in this northern area. With the widening completed, it is our hope to resurface old roadways with minimum interference to traffic.

Financing is also a factor that determines the amount of resurfacing that can be accomplished.

Assuming now that we have made the determination as to which areas will be resurfaced, we have 1 or 2 more decisions to make.

The first determination is to decide how much design and construction supervision can be handled directly by the engineering staff and how much will be handled by consultants.

In recent years we have attempted, with our own forces, to design and supervise a spring contract, a summer contract, and a fall contract. A complication to this schedule, however, is the fact that ever-increasing traffic counts have dictated that resurfacing during the summer months on the main line be accomplished during nighttime hours.

The second determination to be made is how to divide the area into contracts. We are limited, because of traffic considerations, to work areas not exceeding 2 miles in length. In addition there must be 2 miles between work areas. Lengthening the former or shortening the latter will generally create unacceptable backups. We have always found it more advantageous to pave contiguous northbound and southbound areas under the same contract. There are definite savings to be had if the contractor can schedule his work such that, after completing a lane in an area, say, southbound, he merely swings over to the other side and paves northbound. If it can be so scheduled, he can pave 4 miles of 3-lane roadway without ever making an unnecessary equipment move. As you know, moving equipment without production is very expensive.

The third determination to be made is which areas can be paved by our own maintenance forces with our own equipment. The Turnpike Authority has had, for the past 6 years, all the equipment necessary and the qualified personnel to carry on full-scale paving operations. The men are full-time maintenance employees. Primarily, the areas resurfaced by maintenance personnel are those areas that do not conveniently fit into contracts for one reason or another.

For this very reason it is difficult to compare costs between contract work and Authority work. At the present time we are sure that costs of work by our own forces are no greater than and probably less than costs of work by the contractor. There is one decided plus, however. Contractors now know that we are fully capable of resurfacing and that artificially high prices do not have to be tolerated. Average costs for resurfacing from 1963 to 1969 are shown in Figures 1 and 2.

## RESURFACING DESIGN

The design of a resurfacing project is extremely important. We could, in almost all cases, simply set the screed on a paving machine for $1\frac{1}{2}$ in., or whatever the overlay should be, and pave. Our profile and cross slopes are generally good enough to make this a distinct possibility and, in fact, we have experimented along these lines with our own paving forces. I feel, however, that I cannot recommend this method even on a turnpike such as ours with a generally excellent profile. Imperfections are magnified, problem areas, when they do exist, are left to the paving foreman to straighten out, and drainage at inlets and curbs is often adversely affected. Actually, the cost of a design is minimal. The preparation of a design such as the one the Turnpike Authority has developed ensures good, uniform riding qualities; a profile, although not necessarily the same as the original, that conforms to original standards; and uniform cross slopes. Moreover, our design ensures the most economical job, because it provides for adjustment of the design profile to take advantage of the existing profile.

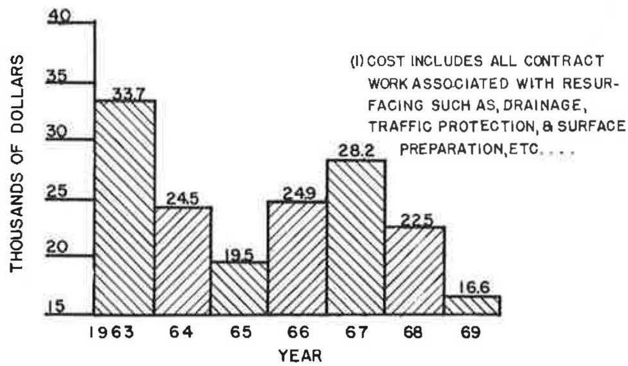


Figure 1. Average cost per lane-mile of resurfacing.

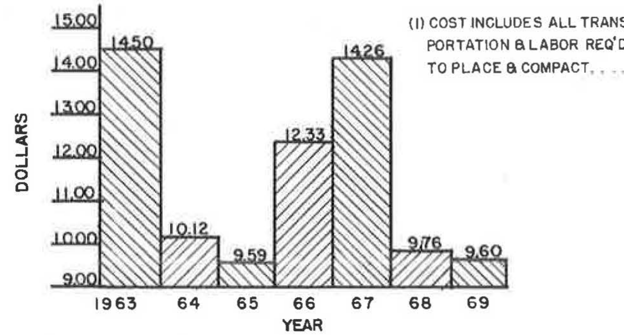


Figure 2. Average annual cost per ton of asphalt.

## Field Survey and Sectioning

The first step is to cross section the existing roadway in the area to be resurfaced. Our usual policy is to do this work during the winter, utilizing the same engineering department personnel who will supervise and inspect construction during the spring and summer. The cross sections are taken every 25 ft or quarter station, and 12 control points are used for each section. Figure 3 shows our standard control section. The numbering system, as shown in Figure 3, is used throughout the design procedure and appears finally on the completed grade sheets. A completed standard grade sheet is shown in Figure 4. These sheets are made a part of the final contract documents, along with the plans and specifications. All inlets are also located both vertically and horizontally, and provision is made for them to be raised as required. This information is again shown on the grade sheets.

We accomplish sectioning by first closing the left or fast lane. Stations are then measured in and painted on the pavement. Bench marks are then established on median guardrail posts at 200-ft intervals. We have found that sight distances of over 200 ft are impractical with a level set up in such close proximity to high-speed traffic, a great percentage of which is trailer trucks. With the left lane closed we can shoot the left shoulder, solid painted line between left lane and left shoulder, and dashed painted line between left and center lanes. The painted white lines are accurately spaced, and no transverse measurements are required. All bridge clearances are checked and noted as the sectioning progresses.

The next step is to close the right lane. Stationing need not be remeasured, because it is possible to range in the painted control points in the left lane quite accurately. The rest of the control points are then painted, and elevations are taken. It is never necessary to close the center lane. We go to great lengths to avoid center-lane closings because they are exceedingly hazardous. A 3-man field party can section a mile of roadway in 3 days.

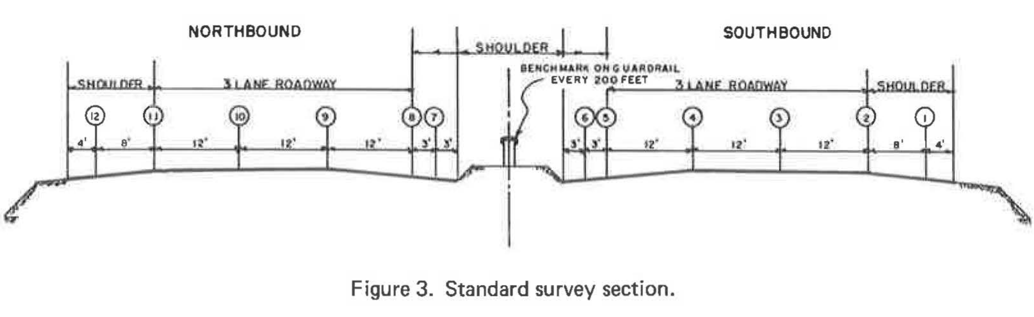


Figure 3. Standard survey section.

NEW JERSEY TURNPIKE AUTHORITY

RESURFACING: MILE 71.7 TO MILE 73.1 : CONTRACT R-287 : S.B.L. SHEET 10 OF 70

| | STA. | 1 | 2 | 3 | 4 | 5 | 6 |
|---|---|---|---|---|---|---|---|
| PROP. | 3816 | 58.19 | 58.67 | 58.89 | 59.04 | 58.92 | 58.74 |
| EXIST. | +00 | 58.19 | 58.50 | 58.69 | 58.82 | 58.79 | 58.71 |
| FILL-ft. | | — | .17 | .20 | .22 | .13 | .03 |
| FILL-in. | | — | 2 | 2 3/8 | 2 5/8 | 1 1/2 | 3/8 |
| | | B-0 | B-1/2 | B-7/8 | B-1 1/8 | B-0 | B-0 |
| | | T-0 | T-1 1/2 | T-1 1/2 | T-1 1/2 | T-1 1/2 | T-3/8 |
| PROP. | | 58.32 | 58.80 | 59.02 | 59.17 | 59.05 | 58.87 |
| EXIST. | +25 | 58.25 | 58.62 | 58.83 | 58.95 | 58.93 | 58.82 |
| FILL-ft. | | .07 | .18 | .19 | .22 | .12 | .05 |
| FILL-in. | | 7/8 | 2 1/8 | 2 1/4 | 2 5/8 | 1 1/2 | 5/8 |
| | | B-0 | B-5/8 | B-3/4 | B-1 1/8 | B-0 | B-0 |
| | | T-7/8 | T-1 1/2 | T-1 1/2 | T-1 1/2 | T-1 1/2 | T-5/8 |
| PROP. | | 58.45 | 58.93 | 59.15 | 59.30 | 59.18 | 59.00 |
| EXIST. | +50 | 58.35 | 58.74 | 58.95 | 59.07 | 59.04 | 58.96 |
| FILL-ft. | | .10 | .19 | .20 | .23 | .14 | .04 |
| FILL-in. | | 1 1/4 | 2 1/4 | 2 3/8 | 2 3/4 | 1 5/8 | 1/2 |
| | | B-0 | B-3/4 | B-7/8 | B-1 1/4 | B-1/8 | B-0 |
| | | T-1 1/4 | T-1 1/2 | T-1 1/2 | T-1 1/2 | T-1 1/2 | T-1/8 |
| PROP. | | 58.58 | 59.06 | 59.28 | 59.43 | 59.31 | 59.13 |
| EXIST. | +75 | 58.60 | 58.90 | 59.11 | 59.18 | 59.16 | 59.07 |
| FILL-ft. | | — | .16 | .17 | .25 | .15 | .06 |
| FILL-in. | | — | 1 7/8 | 2 | 3 | 1 3/4 | 3/4 |
| | | B-0 | B-3/8 | B-1/2 | B-1 1/2 | B-1/4 | B-0 |
| | | T-0 | T-1 1/2 | T-1 1/2 | T-1 1/2 | T-1 1/2 | T-3/4 |

Figure 4. Standard grade sheet.

Office Work

The high point between left and center lanes is the basis of the design. They are the only profiles that must be plotted. After plotting these existing profiles, design profiles may be plotted and adjusted. Our control between existing and proposed profile is $1\frac{1}{2}$-in. minimum overlay.

We are fortunate in New Jersey to have excellent trap rock aggregates. We use $\frac{5}{8}$-in. maximum size aggregate in surface courses and a $\frac{7}{8}$-in. aggregate in binder courses with an asphalt content of between 5 and 7 percent for the former, and $4\frac{1}{2}$ and 6 percent for the latter. The Marshall method of design is specified with a minimum stability of 1,200 lb and a flow between 0.12 and 0.18 in. We obtain stabilities of 1,500 and 1,600 lb consistently.

After a good economical profile has been designed, it is only necessary to check cross slopes in critical areas to ensure minimum cover. With a little experience, this can be done by inspection very rapidly.

For one of our projects a few years ago, a consultant engaged to design the project wrote a computer program using our design method. With this system, the grade sheets (showing all the information, and in the same form as our usual grade sheets) were printed out by the computer (Fig. 5).

## CONSTRUCTION

The protection of traffic is a very important factor in the construction phase. It is impossible to discuss in detail our traffic protection procedures in this paper. More than 10 percent of the money spent on a resurfacing contract is accounted for by traffic protection items.

A second important area, with respect to construction, is drainage. In almost all areas we now install transverse bleeder drains spaced at 100-ft intervals wherever practical. These drains are 2 ft wide, begin at the edge of the right lane, extend through the shoulder and berm area, and run out into the side slope. They are simply french drains, backfilled with washed, compacted $\frac{3}{8}$-in. gravel. In areas where it is impractical to "daylight" these drains in the side slopes they are constructed from the edge of pavement to longitudinal underdrains that are constructed in the right shoulder area. These intercepting underdrains are combination drains using the same washed gravel and perforated, corrugated metal, bitumastic-coated pipe. These pipes outfall either in the ditch areas or into existing drainage structures. The most important point to make with respect to the bleeder drains is that they must extend into the existing macadam base.

Similar drainage is installed in the left shoulder area where required. Ordinarily, however, existing inlets and drainage in this area are sufficient. We do make it a policy, though, to drain all sag points. We have found that in some instances our existing macadam base is acting like a large drain itself, with no outlet.

All drainage work must necessarily be performed before paving commences.

## PAVING

The first step in the paving operation is crack sealing. We require all cracks over $\frac{1}{8}$ in. to be sealed with a neoprene-modified sealer. A tack coat is then applied with a pressure-type distributor. We use an RS1 emulsified asphalt applied at the rate of 0.10 gal/sq yd. Power brooming is required on shoulder areas, but traveled lanes do not require any cleaning.

Pavers are required to be provided with automatic screed control. We have had excellent results using piano-wire guides for screed control when the contractor has conscientiously maintained the wire in place, after initial setting. Standard equipment and method of compaction are as follows: The first rolling is accomplished with a 3-wheel "breakdown" roller with a minimum weight of 10 tons. The 3-wheel roller is followed by a rubber-tired roller developing 90 psi ground pressure per tire. The pneumatic-tired roller is, in turn, followed by a 3-axle "bump" roller weighing not less than 12 tons. The main point, however, is that the breakdown roller work as closely as possible to the paver; and that the rubber-tired roller work as closely as possible to the

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| | | | | | | | |
|---|---|---|---|---|---|---|---|
| 4201+25. | JOINT NO | 7 | 8 | 9 | 10 | 11 | 12 |
| | PROP EL | 96.14 | 96.40 | 96.55 | 96.37 | 96.19 | 95.79 |
| | EXIST EL | 96.14 | 96.27 | 96.40 | 96.12 | 96.04 | 95.78 |
| | DIFF (FT) | .00 | .13 | .15 | .25 | .15 | .01 |
| | DIFF (IN) | 0/8 | 1 4/8 | 1 6/8 | 3 0/8 | 1 6/8 | 1/8 |
| 4201+50. | JOINT NO | 7 | 8 | 9 | 10 | 11 | 12 |
| | PROP EL | 96.36 | 96.64 | 96.79 | 96.61 | 96.42 | 96.04 |
| | EXIST EL | 96.36 | 96.50 | 96.63 | 96.40 | 96.25 | 96.04 |
| | DIFF (FT) | .00 | .14 | .16 | .21 | .17 | .00 |
| | DIFF (IN) | 0/8 | 1 5/8 | 1 7/8 | 2 4/8 | 2 0/8 | 0/8 |
| 4201+75. | JOINT NO | 7 | 8 | 9 | 10 | 11 | 12 |
| | PROP EL | 96.58 | 96.88 | 97.03 | 96.85 | 96.66 | 96.26 |
| | EXIST EL | 96.58 | 96.75 | 96.90 | 96.66 | 96.50 | 96.20 |
| | DIFF (FT) | .00 | .13 | .13 | .19 | .16 | .06 |
| | DIFF (IN) | 0/8 | 1 4/8 | 1 4/8 | 2 2/8 | 1 7/8 | 6/8 |
| 4202+00. | JOINT NO | 7 | 8 | 9 | 10 | 11 | 12 |
| | PROP EL | 96.82 | 97.12 | 97.27 | 97.09 | 96.90 | 96.48 |
| | EXIST EL | 96.78 | 96.99 | 97.12 | 96.87 | 96.76 | 96.39 |
| | DIFF (FT) | .04 | .13 | .15 | .22 | .14 | .09 |
| | DIFF (IN) | 4/8 | 1 4/8 | 1 6/8 | 2 5/8 | 1 5/8 | 1 1/8 |
| 4202+25. | JOINT NO | 7 | 8 | 9 | 10 | 11 | 12 |
| | PROP EL | 97.06 | 97.36 | 97.51 | 97.33 | 97.14 | 96.70 |
| | EXIST EL | 96.95 | 97.20 | 97.34 | 97.13 | 97.00 | 96.65 |
| | DIFF (FT) | .11 | .16 | .17 | .20 | .14 | .05 |
| | DIFF (IN) | 1 3/8 | 1 7/8 | 2 0/8 | 2 3/8 | 1 5/8 | 5/8 |
| 4202+50. | JOINT NO | 7 | 8 | 9 | 10 | 11 | 12 |
| | PROP EL | 97.30 | 97.60 | 97.75 | 97.57 | 97.38 | 96.92 |
| | EXIST EL | 97.20 | 97.39 | 97.52 | 97.37 | 97.23 | 96.91 |
| | DIFF (FT) | .10 | .21 | .23 | .20 | .15 | .01 |
| | DIFF (IN) | 1 2/8 | 2 4/8 | 2 6/8 | 2 3/8 | 1 6/8 | 1/8 |
| 4202+75. | JOINT NO | 7 | 8 | 9 | 10 | 11 | 12 |
| | PROP EL | 97.54 | 97.84 | 97.99 | 97.81 | 97.62 | 97.14 |
| | EXIST EL | 97.41 | 97.59 | 97.77 | 97.63 | 97.45 | 97.10 |
| | DIFF (FT) | .13 | .25 | .22 | .18 | .17 | .04 |
| | DIFF (IN) | 1 4/8 | 3 0/8 | 2 5/8 | 2 1/8 | 2 0/8 | 4/8 |
| 4203+00. | JOINT NO | 7 | 8 | 9 | 10 | 11 | 12 |
| | PROP EL | 97.78 | 98.08 | 98.23 | 98.05 | 97.86 | 97.38 |
| | EXIST EL | 97.64 | 97.81 | 98.00 | 97.84 | 97.67 | 97.35 |
| | DIFF (FT) | .14 | .27 | .23 | .21 | .19 | .03 |
| | DIFF (IN) | 1 5/8 | 3 2/8 | 2 6/8 | 2 4/8 | 2 2/8 | 3/8 |

Figure 5. Computer-printed grade sheet.

breakdown roller. The rubber-tired roller is the most important link in this chain, in our opinion; and it is only effective when working the hot asphalt. The final requirement is that 95 percent of laboratory density be reached. We have never had any real problem in this regard.

One final point relates to our method of beginning and ending resurfaced areas. We make a feathered transition from the existing section to the full-depth overlaid section. We have had excellent results with feathered edges. Primary considerations for the construction of these feathered areas are that the asphalt be hot and that care be exercised by the workmen in raking out the large aggregate in the portion of the feather that must necessarily vary from $\frac{5}{8}$ to 0 in. A great deal of close inspection is required, but the ridability, as well as savings gained by this method over cutting and removing pavement, are in our opinion well worth it.

The average cost for the resurfacing I have been describing was just over $1.50/sq yd for the year 1969. This includes drainage, traffic protection, and all other associated contract work. Average thickness is between 2 and $2\frac{1}{2}$ ft.

Resurfacing on the New Jersey Turnpike has, of course, certain unique problems that others may not be faced with. The main points I have tried to make with regard to our work are as follows:

1. Attention to the design of resurfacing is well worth the cost of the design.
2. In order to receive full value for a resurfacing dollar, it is extremely important that particular attention be paid to drainage. I know how very basic this is; yet, somehow its importance is often submerged in the attempt to stretch resurfacing dollars.
3. The New Jersey Turnpike Authority, through its policy of acting before permanent and irreparable damage has been done to the paving structure, has found resurfacing, as discussed here, to be an entirely acceptable and economical means of salvaging its pavements.

# Heavy Pneumatic Rolling Prior to Overlaying: A 10-Year Project Report

J. W. LYON, Louisiana Department of Highways

This report covers a 10-year field study to determine the practicability of using a pneumatic-tired, 50-ton roller to break and seat old concrete pavements before overlaying with hot-mix asphaltic concrete. The method of this study is a comparative analysis of the behavior of various roadway sections under traffic employing different breaking and seating techniques of a 50-ton pneumatic roller and an impact hammer on a selected construction project with a wet subgrade. Results indicate that a 50-ton roller should be used in conjunction with an impact hammer, using 3 or 4 roller coverages for slab-breaking and seating on a wet subgrade to reduce deflection cracking. Specifications were developed but subsequent results, because of the general dryness of the subgrades on these projects, were not as successful as anticipated. Ten-year characteristics of the study project verified the initial study findings. The 50-ton roller used in conjunction with an impact hammer to break and seat old concrete pavements on wet or yielding subgrades can reduce reflection cracking, but its use on firm subgrades has not been effective.

•FOR MANY YEARS the Louisiana Department of Highways has renewed old concrete pavements by using hot-mix overlays. Too often slab movement has continued. A study was undertaken in 1959 to reduce slab movement by breaking and seating concrete pavement slabs with a 50-ton roller prior to overlaying.

This study was made on a construction project completed in the early winter of 1959. The project site selected was an old, badly pumping, 18-ft concrete pavement in the bayou country of southern Louisiana. The concrete slab is 6 in. thick at the center, thickened to 8 in. at the edges, and widened 3 ft on each side with concrete. The hot-mix overlay was $3\frac{1}{2}$ in. of binder and wearing courses with a quantity of binder course for leveling (equivalent to 1 in. of thickness).

The subgrade immediately under the old concrete slabs are high silt soils containing more than 50 percent silt, with moderate plasticity indexes from 5 to 15 and with group indexes from 8 to 12. These soils would generally be classified as ML and CL by the United Soil Classification System. The general soil conditions on the study project are relatively uniform (and of high silt content) being an old flood distributary of the Mississippi River.

Initial findings were reported in 1963 (1).

## METHODOLOGY

The study project was divided into 3 general sections for test and evaluation purposes. The first general test section was treated with an impact hammer only and was intended as the study's parameter. A second general test section was treated with a roller only. The remaining general test section was treated with both an impact hammer and a roller.

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The following roller and hammer configurations were tried and evaluated.

1. Impact hammer—complete slab breakage (slab crushed in place); nominal breakage (midpoint or third-point cracking); and general breakage (more intense treatment near joints and cracks).
2. Roller—1, 2, 3, and 4 coverages.
3. Roller and impact hammer—single location coverage followed by impact hammering and a single seating coverage of the roller; single coverage followed by hammering; 2 roller coverages followed by hammering, 1 coverage with additional hammering (as needed), and 1 final seating coverage; 2 roller coverages, hammering, and 2 additional roller coverages; and a location coverage, hammering, a seating coverage, additional hammering (as needed), and a final seating coverage of the roller.

In addition to the general treatments tried, several other variables were studied. Among these were roller speed, with and without an open side trench, and treatment before and after placement of the base widening. Parts of the old concrete pavement were deliberately rolled excessively to determine the maximum limits of effective rolling.

The comparative systems used in evaluating the performance of the various breaking and seating techniques are as follows:

1. Relative cracking reflected through the hot mix,
2. Surface measured deflections (determined by Benkelman beam and Dynaflect), and
3. Road surface roughness (determined by a BPR roughness indicator).

In addition, visual observations played an important role in evaluating the section performance.

## RESULTS

The amount of reflection cracking evident after 10 years of service generally verify the first and second year results that were reported previously (1). These previous results indicate that the 50-ton roller should be used in conjunction with an impact hammer with 3 coverages of the roller appearing to be optimum coverage.

Deflections after 10 years of traffic (8,600,000 vehicles and 405,000, 18-kip equivalent loads) are relatively consistent for all treatments as shown in Figure 1. The minimum deflection average was 0.020 in. and the maximum average was 0.303 in. equivalent rebound Benkelman beam deflections under 18-kip axle load.

Based on the deflections obtained after 10 years of traffic, all study sections are performing equally well and satisfactory. There is no slab pumping noticed in any section, regardless of the breaking and seating treatment. Overlaying this study project is imminent because of its surface deterioration, not because of structural deficiency.

General roughness measurements are very satisfactory after 10 years of traffic (Table 1). Eighty percent of all readings are in the range of 60 to 70 in./mile, obtained in ½ mile increments using the BPR roughness indicator.

The section treated with the roller only has the most consistent smoothness. The section treated with the hammer only and the section treated with the roller and hammer together have more variable roughnesses (from lows of approximately 45 in./mile to highs of approximately 85 in./mile). All sections have smoother surfaces now than immediately after overlaying.

The consistent smoothness of the section on which only the roller was used might be related to the seating operation or to surface characteristics of the hot-mix overlay.

## DISCUSSION OF RESULTS

This test project indicates the superiority of a seating system using 4 roller coverages with an impact hammer to reduce reflection cracking. All sections of the test project have performed satisfactorily. Deflections obtained after 10 years are uniform and within acceptable levels, regardless of the breaking and seating technique used. Previous conclusions (1) are as follows:

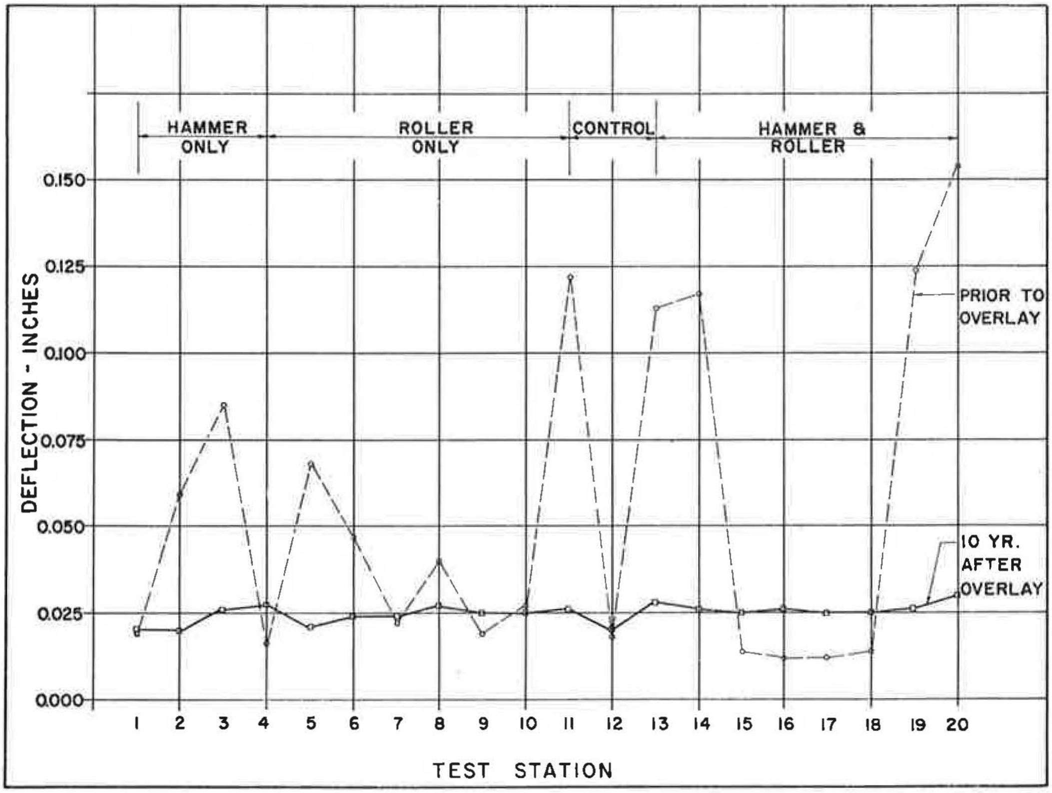


Figure 1. Deflection values of the original concrete pavement before overlaying and 10 years after overlaying for the various treatments.

TABLE 1

ROUGHNESS MEASUREMENTS

| Test Station | Treatment | No. of Roller Coverages | Roughness (in./mile) Original | 1 Year | 10 Years |
|---|---|---|---|---|---|
| 1 | Hammer only | | 158 | 76 | 66 |
| 2 | Hammer only | | 194 | 108 | 64 |
| 3 | Hammer only | | 186 | 92 | 61 |
| 4 | Hammer only | | 148 | 86 | 48 |
| 5 | Roller only | 4 | 148 | 88 | 68 |
| 6 | Roller only | 2 | 152 | 84 | 70 |
| 7 | Roller only | 2 | 152 | 84 | 70 |
| 8 | Roller only | 4 | 166 | 92 | 68 |
| 9 | Control | | 142 | 96 | 66 |
| 10 | Control | | 142 | 94 | 66 |
| 11 | Roller only | 2 | 140 | 98 | 66 |
| 12 | Control | | 162 | 82 | 62 |
| 13 | Hammer and roller | 4 | 202 | 90 | 46 |
| 14 | Hammer and roller | 3 | 284 | 86 | 52 |
| 15 | Hammer and roller | 2 | 128 | 92 | 48 |
| 16 | Hammer and roller | 2 | 136 | 94 | 48 |
| 17 | Hammer and roller | 2 | 134 | 90 | 62 |
| 18 | Hammer and roller | 2 | 128 | 82 | 66 |
| 19 | Hammer and roller | 3 | 206 | 86 | 72 |
| 20 | Hammer and roller | 4 | 206 | 86 | 74 |

> This study, after two years of service, points out the superiority of a breaking and seating procedure for old pavements on wet subgrades, consisting of a combination of three roller coverages and impact hammering.... The recommended field procedure would be to locate moving slabs with one coverage of the 50-ton roller; to break these slabs with the impact hammer; to apply a seating coverage with the 50-ton roller, also locating additional or continual rocking slabs with this coverage; to accomplish what additional breaking may be required with the impact hammer; and to follow this with the final seating coverage of the 50-ton roller.

Based on these conclusions, specifications were developed. Subsequent use of the 50-ton roller on other overlay projects was not considered successful. Field use indicated that the 50-ton roller was not effectively locating areas that needed hammering, nor was the roller breaking or seating slabs.

Generally these projects were overlaid during dry periods of the year when the weather is conducive to hot-mix operations. Subgrades were relatively dry, nonyielding, and well able to support the 50-ton roller. The rolling and seating techniques previously developed did not fit the conditions being encountered in the field, and 50-ton rolling was not particularly productive. The success of the 50-ton roller on the study project is attributed to the general wetness of the subgrade, allowing the roller to fully exploit its weight by moving, breaking, and seating the old concrete into the subgrade. Lack of success on subsequent projects was certainly influenced by generally drier subgrade conditions.

This study, based on the test projects and subsequent projects, indicates that subgrade moistures must be near, or slightly above, their optimum moisture contents (AASHO T99) in order to derive any appreciable benefits from the use of the 50-ton roller in Louisiana.

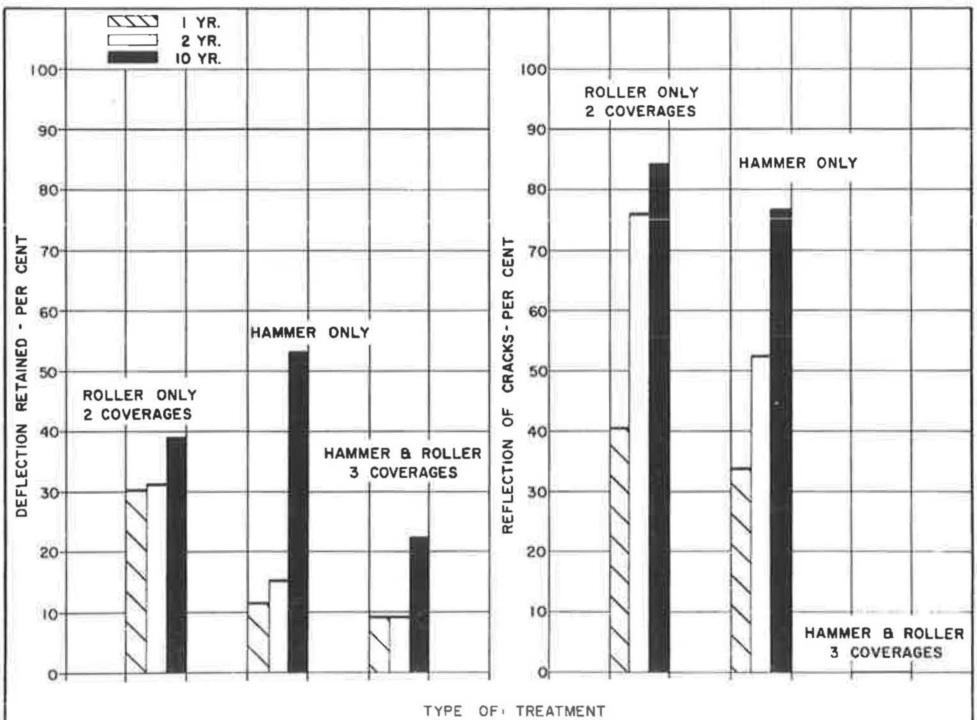


Figure 2. Changes in reflection cracking and deflections for 1, 2, and 10 years of traffic versus original conditions for various optimum treatments.

TABLE 2

MEASUREMENT OF TRANSVERSE REFLECTION CRACKS

| Test Station | Treatment | No. of Roller Coverages | Length of Transverse Cracks (ft/100 linear ft) | | | | |
|---|---|---|---|---|---|---|---|
| | | | Before Overlay | Immediately After Overlay | 6 Months After Overlay | 1 Year After Overlay | 10 Years After Overlay |
| 1 | Hammer only | | 64 | 35 | 30 | 30 | 56 |
| 2 | Hammer only | | 67 | 0 | 0 | 0 | 27 |
| 3 | Hammer only | | 18 | 6 | 11 | 8 | 17 |
| 4 | Hammer only | | 28 | 21 | 28 | 34 | 36 |
| 5 | Roller only | 4 | 78 | 6 | 6 | 6 | 23 |
| 6 | Roller only | 2 | 58 | 32 | 5 | 14 | 28 |
| 7 | Roller only | 2 | 14 | 13 | 11 | 11 | 27 |
| 8 | Roller only | 4 | 17 | 35 | 50 | 49 | 43 |
| 9 | Control | | 29 | 33 | 50 | 39 | 74 |
| 10 | Control | | 21 | 14 | 11 | 9 | 27 |
| 11 | Roller only | 2 | 152 | 20 | 30 | 30 | 47 |
| 12 | Control | | 66 | 12 | 9 | 9 | 22 |
| 13 | Hammer and roller | 4 | 86 | 0 | 0 | 0 | 0 |
| 14 | Hammer and roller | 3 | 109 | 0 | 0 | 0 | 0 |
| 15 | Hammer and roller | 2 | 36 | 26 | 26 | 19 | 19 |
| 16 | Hammer and roller | 2 | 33 | 27 | 33 | 33 | 46 |
| 17 | Hammer and roller | 2 | 33 | 27 | 33 | 31 | 57 |
| 18 | Hammer and roller | 2 | 34 | 19 | 19 | 18 | 18 |
| 19 | Hammer and roller | 3 | 96 | 0 | 0 | 0 | 0 |
| 20 | Hammer and roller | 4 | 74 | 0 | 0 | 0 | 0 |

## CONCLUSIONS

1. Where subgrade moisture contents are high (near optimum moisture but not more than 5 percent above optimum), the 50-ton roller used with an impact hammer may greatly reduce reflection cracking. The test project, after 10 years of traffic, basically verified previous study conclusions that the best results for breaking and seating old pavements on wet subgrades to reduce reflection cracking can be obtained by using the 50-ton roller with an impact hammer. Previous study conclusions from the indicated 3-roller coverages were optimum. Ten-year results indicate 4 roller coverages are optimum for reduction of reflection cracking. This is shown in Figure 2 and given in Table 2.

2. The 50-ton roller is not generally recommended for breaking and seating old concrete pavements prior to overlaying. Its performance on the study project was good, but subsequent use was not as successful as hoped. Its use as outlined in this report should be restricted to yielding subgrades.

## REFERENCE

1. Lyon, J. W., Jr. Slab Breaking and Seating on Wet Subgrades With Pneumatic Roller. Highway Research Record 11, 1963, pp. 89-98.

# The Effect of Pavement Breaker-Rolling on the Crack Reflectance of Bituminous Overlays

G. R. KORFHAGE, Minnesota Department of Highways

The purpose of this study was to determine whether breaking a concrete pavement prior to being overlaid would result in any reduction in the amount of crack reflectance. This report describes the design, construction, and performance of a typical widening and resurfacing project, on a portion of which a 59-ton roller was used to crack the old concrete slab prior to construction. This process was found to significantly reduce some types of cracking and is recommended for future projects of this nature.

•MANY OF THE 18- to 20-ft concrete pavements constructed a number of years ago have required rehabilitation due to deterioration of the concrete and inadequate width to satisfactorily serve present-day traffic. This has usually been accomplished by placing a widening strip on one or both sides of the pavement to increase the width to 24 ft, placing a thin bituminous leveling course directly over the old concrete, and surfacing the entire width with 3 in. of bituminous material. This method has proved to be quite satisfactory in that it provides a smoother, wider pavement. However, it has been found that usually within a short period of time the transverse joints and centerline joint of the old pavement and the joint between the old pavement and the widening strip reflect through the bituminous overlay, and ultimately slab movements cause recurrence of general roughness. Maintenance costs go up, and the serviceability of the surface is reduced.

In the past, one solution has been to provide lifts of granular material over rough, old pavements before placing the bituminous surface. The added thickness retarded the reflectance of cracks and roughness; and, when thick enough, lifts actually eliminated most of the effects of the old pavement. However, this type of construction was costly because of the large quantities of granular material needed and because of the additional grade widening usually required. It also might be considered extravagant, in that the full potential of the old pavement as a base course was not utilized and because it consumed such large volumes of good base aggregate—an undesirable feature in any case, but especially so in areas of gravel scarcity.

The use of a heavy roller to crack the old concrete pavement and to seat it on the subgrade in an attempt to reduce crack reflection was tried experimentally in Minnesota in 1959. The project was located on T. H. 212 between Bird Island and Stewart (CS 6511 and CS 6512). The 1931 concrete pavement, like a number of the older pavements in Minnesota, had warped panels, cracks, and faulted joints to the extent that the riding qualities had become somewhat objectionable, especially for trucks. The project was a typical widening and resurfacing project, in which the bituminous mixtures were placed directly on and adjacent to the old concrete pavement. The subgrade soil was predominantly clay loam.

The experiment was limited to a section approximately 1½ miles in length located about a mile west of Stewart. This section was rolled with a 59-ton roller to break the old pavement. The rolling produced 1,868 new transverse cracks in the 2 lanes. Including joints and old cracks, there were nearly 3,000 transverse openings. In other words, after rolling there was a transverse crack or joint about every 5 ft in each lane.

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Paper sponsored by Committee on Salvaging Old Pavements by Resurfacing and presented at the 49 Annual Meeting.

The hot-mixed bituminous mixtures for the surfacing over the old slab on most of this project consisted of a 1½-in. leveling course, a 1½-in. binder course and a 1½-in. wearing course. However, on the section where the old slab was cracked by the roller, 3 different thicknesses of leveling course or bituminous base were used, making the total bituminous thickness 5, 6, and 9 in. The 2-ft widening strip placed on each side of the old slab had 6 in. of bituminous stabilized base, a total of 9 in. of bituminous material, throughout the entire project.

In addition to the 3 test sections in the rolled section, which correspond to the variation in the thickness of the overlay, 3 control sections were selected in the unrolled portion for comparison purposes.

The average daily traffic on the test sections in 1962 was 1,930 vehicles including 320 (17 percent) trucks.

Evaluation of the various test sections was based on periodic condition surveys, roughometer measurements, and rut depth determinations.

This final report covers observations and measurements made during construction and the 5½-year period that followed. Preliminary results were reported by Velz (1, 2).

## THE OLD PAVEMENT

The old concrete pavement constructed in 1931 was 20 ft wide, 9 in. thick at the edges, and tapered to 7 in. thick 4 ft from the edges, a typical 9-7-9 cross section. Panels were generally 40 ft 4 in. long, with every other joint being an expansion joint. The 1-in. expansion joints had ¾-in. dowels with steel sockets on one end. The dummy-type contraction joints had ¾-in. dowels that were greased on one end. Slab reinforcement consisted of two ⅝-in. bars along each edge, one ⅝-in. bar along each side of the centerline, one ⅝-in. bar along each side of the joints, and ½-in. by 4-ft tie bars across the centerline. The pavement design details are shown in Figure 1.

In several small portions of the project, the panels were only 20 ft in length. Some of these shorter panels occurred in both the rolled and comparison sections.

Just prior to reconstruction in April 1959, the Minnesota (BPR-type) roughometer showed the old pavement to have an average roughness of 138 in./mile, with 163 in. being the roughest miles recorded and 121 in. being the smoothest. This roughness,

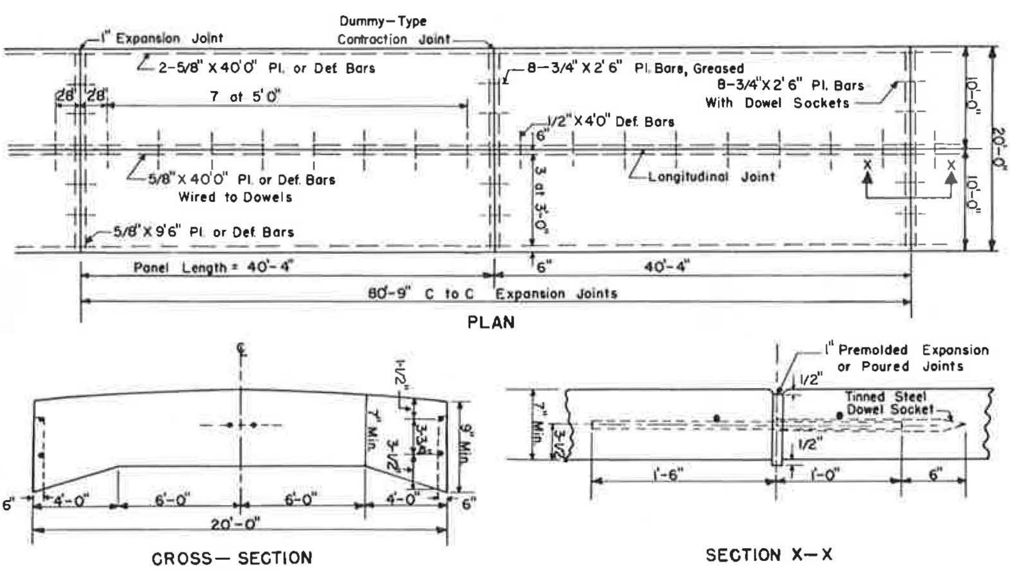


Figure 1. Old pavement design details.

combined with the warped panels, caused very unsatisfactory riding qualities on a considerable length of the project.

## PAVEMENT BREAKER-ROLLING

The pavement breaker-rolling was performed July 23 and 24, 1959, on the $1\frac{1}{2}$-mile experimental section near the east end of this project, Station 950 to Station 1025. The Special Provisions required that each 10-ft lane be covered by 10 passes of a 59-ton roller having 4 wheels on 1 transverse axle and tire air pressure of 90 psi. These provisions were followed, except that 20 passes of the roller were made in the westbound lane between Station 949 and Station 964+13, and with the further exception that the rolling was extended beyond Station 1025 to an entrance at Station 1029 to facilitate turning the roller around.

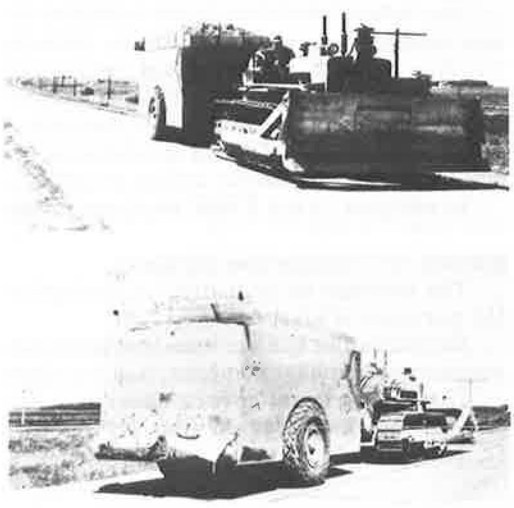

Figure 2. Roller-tractor combination.

### Tire Contact Pressure

The roller was a Bros Compactor loaded to 118,000 lb (59 tons) and fitted with four 18.00 × 25, 24-ply, diamond-tread tires inflated to 90-psi air pressure. The rolling width, measured from outside to outside of tire contact, was 8 ft 8 in. An International TD-24 tractor was used to pull the roller at a speed of 2 to 3 mph and did an excellent job of controlling it at all times. The roller-tractor combination is shown in Figure 2.

To measure the tire contact areas, a length of 30-in. wide paper was placed across a clean portion of pavement slab, and the roller was pulled forward until the tires were on the paper. Then, by using pressurized cans, paint was sprayed completely around the contact periphery of each tire. When the roller was moved ahead, the 4 contact areas were outlined on the paper as shown in Figure 3. Later, these areas were measured as follows:

| Tire | Gross Contact Area (sq in.) |
|---|---|
| Left outside | 354.3 |
| Left inside | 344.7 |
| Right inside | 384.8 (noticeably larger) |
| Right outside | 329.3 (noticeably smaller) |
| Total | 1,413.1 |

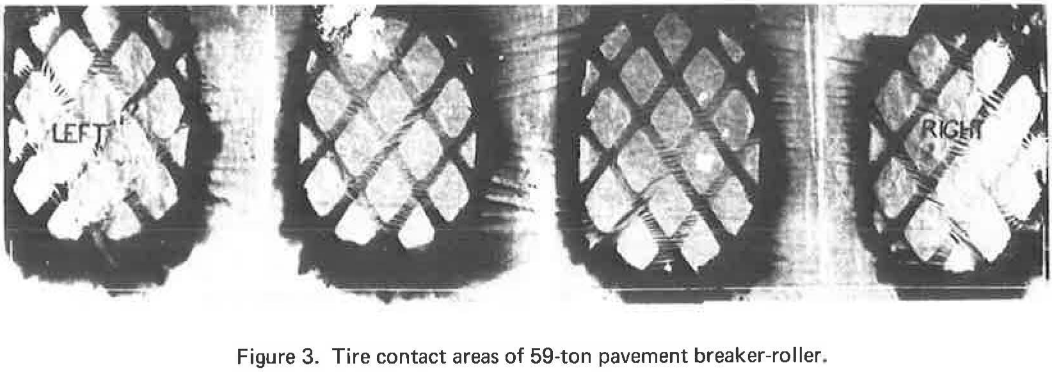


Figure 3. Tire contact areas of 59-ton pavement breaker-roller.

TABLE 1

VERTICAL MOVEMENTS MEASURED AT EXPANSION AND CONTRACTION JOINTS

| Station | Joint Type | At Centerline (ft) | | At Left Edge (ft) | |
|---|---|---|---|---|---|
| | | Panel 1 | Panel 2 | Panel 1 | Panel 2 |
| 959+00 | Expansion | 0.015 | 0.030 | 0.025 | 0.030 |
| 959+41 | Contraction | 0.010 | 0.030 | 0.025 | 0.025 |
| 959+82 | Expansion | 0.030 | 0.015 | 0.040 | 0.020 |
| 960+22 | Contraction | 0.020 | 0.025 | 0.025 | 0.010 |

By using this total area, the average contact pressure was computed as 83.5 psi.

## Vertical Slab Movements

From visual observations, it was noted that there was considerable variation in slab movement at joints from panel to panel and place to place. It seemed that when the roller passed the slab ends moved more at expansion joints than at contraction joints. This seemed logical considering that some aggregate interlock was still effective in resisting vertical movement at the contraction joints. However, measurements taken at 4 joints (2 expansion and 2 contraction) indicated that the vertical movements were not significantly larger at the expansion joints (Table 1).

## Cracking

Visible cracking of the pavement slab did not occur until after several passes of the roller. When they first occurred, cracks were extremely fine and difficult to observe. As rolling progressed, the top edges spalled slightly, and the cracks became more visible. It was also observed that traffic caused a similar spalling of the cracks. Much of this spalling was very minute, being just enough to show whitish dots along the path of the crack, although a few spalls ultimately were an inch or more in diameter. Practically all cracks, new and old, were transverse cracks with the exception of some diagonal cracks in frost-heave areas.

Prior to rolling, there were 368 cracks in the left (westbound) lane and 339 cracks in the right (eastbound) lane in the experimental sections. Rolling caused 933 and 935 new cracks in the respective lanes. When 420 half-width joints were added to the cracks, there was a total of 2,995 openings in the 2 lanes. Converting these figures to cracks or openings per station, the following comparisons can be made:

| Cracks or Openings | Left Lane | Right Lane | Both Lanes |
|---|---|---|---|
| Old cracks per station | 4.7 | 4.3 | 9.0 |
| New cracks per station | 11.9 | 11.9 | 23.8 |
| Total | 16.6 | 16.2 | 32.8 |
| Openings per station (including joints) | 19.3 | 18.9 | 38.2 |
| Average spacing between openings (ft) | 5.2 | 5.3 | — |

Between Station 949+00 and Station 964+13, 10 roller passes were made in the right lane and 20 passes in the left lane. The difference in cracking in the 2 lanes was as follows:

| Cracks or Openings | Left Lane | Right Lane |
|---|---|---|
| Old cracks | 77 | 67 |
| New cracks | 207 | 184 |
| Total | 284 | 251 |
| Openings (including joints) | 322 | 289 |
| Old cracks per station | 5.1 | 4.4 |
| New cracks per station | 13.7 | 12.2 |
| Total | 18.8 | 16.6 |
| Openings per station (including joints) | 21.3 | 19.1 |

The 20 passes in the left lane caused only 23 more new cracks than the 10 passes in the right lane. This small increase in cracking indicates that, for this project, 10 passes of the roller were sufficient to develop the optimum amount of cracking.

On this project, the standard joint spacing was approximately 40 ft. However, there was a 580-ft section in the rolled areas that had a joint spacing of 20 ft. A comparison of cracking can be made between the sections using these 2 joint spacings as follows:

| Cracks or Openings | Cracks in Both Lanes | |
|---|---|---|
| | 20-Ft Panels | 40-Ft Panels |
| Old cracks per station | 10.9 | 8.9 |
| New cracks per station | 22.2 | 24.0 |
| Total | 33.1 | 32.9 |
| Openings per station (including joints) | 42.8 | 37.9 |

Rolling produced slightly more cracks per station in the section with 40-ft panels than in the section with 20-ft panels. However, there were slightly more old cracks in the section with 20-ft panels. Combining the new and old cracks, the section with 20-ft panels had slightly more total cracks per station after rolling. When the joints are added, the section with 20-ft panels averaged 4.9 more openings per station than the section with 40-ft panels.

### Permanent Slab Deflections and Roughness

Roughometer readings before and after rolling showed a slight decrease in average roughness in the westbound lane and no change in the eastbound lane. The westbound lane averaged 160 in./mile in April prior to rolling and 154 in./mile in July after rolling. The eastbound lane averaged 154 in./mile at both times.

Profiles were taken 8 ft right and left of the centerline on the old concrete pavement at 3 locations before and after the pavement breaker-rolling.

Stations 950 to 955—There were only minor changes in the profile of the old pavement. At most places, rolling depressed the old slab 0.01 to 0.06 ft. No measureable change in elevation was noted in the remainder of this section, except for a few places where the slab was 0.01 to 0.04 ft higher after rolling.

Stations 980 to 990—Throughout most of this section, the profile after rolling was within 0.02 ft of the original profile. Generally, the rolling depressed the slab, but not at all places. The old pavement was depressed more than 0.02 ft only in isolated areas.

Stations 1010 to 1025—Rolling caused a lowering of the pavement of 0.02 to 0.05 ft throughout most of this section. However, there were areas of little or no change and areas where the slab was slightly higher after rolling. At one point, at a crack, the slab was depressed 0.13 ft. This was the maximum permanent deflection caused by rolling as measured by the profiles.

## PROJECT DESIGN

The standard resurfacing section for the project consisted of a 1½-in. average leveling course, a 1½-in. binder course, and a 1½-in. wearing course over the old concrete pavement. This section was varied in the rolled experimental portion of the project to provide for a 2-in. average leveling course, a 3-in. minimum bituminous base, and a 6-in. minimum bituminous base as alternates to the 1½-in. average leveling course on the standard section. The design thickness for the various sections are given in Table 2. A layout of the project indicating the locations of the various experimental sections and 3 control sections in the unrolled area selected for comparison purposes is shown in Figure 4.

The 2-ft widening section on each side of the old concrete pavement was uniform throughout the project and consisted of 11 in. of Class 4 subbase and 6 in. of Class 4A base stabilized with asphalt emulsion (SS-1). Typical cross sections of the various sections are shown in Figure 5.

TABLE 2

DESIGN THICKNESSES

| Station | Thickness (in.) | | | |
|---|---|---|---|---|
| | Wearing Course | Binder Course | Leveling Course | Bituminous Base |
| Unrolled sections | 1½ | 1½ | 1½[a] | — |
| Stations 950 to 975 | 1½ | 1½ | — | 3[b] |
| Stations 975 to 1000 | 1½ | 1½ | — | 6[b] |
| Stations 1000 to 1025 | 1½ | 1½ | 2[a] | — |

[a]Average. [b]Minimum.

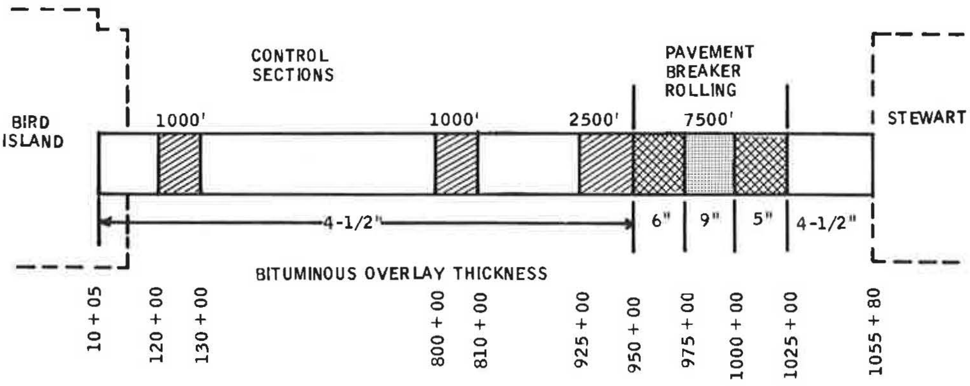


Figure 4. Project layout.

## PERFORMANCE OF THE PROJECT

The performance of the bituminous resurfaced sections was evaluated and compared in several ways. Crack surveys were made before and after rolling and 10 times since resurfacing. Profiles were taken before and after rolling, after completion of the surfacing, and in February 1960 to show the effects of frost heaves. Detailed cross sections of the bituminous surface were taken several times to detect rutting in the wheel tracks. Roughometer measurements were made annually.

Several comparison sections were selected, with which the performance of the experimental sections could be compared and evaluated. Control section 1 (Stations 120 to 130) was one of the roughest portions of the old pavement. Control section 2 (Stations 800 to 810) was an area of typical roughness. Control section 3 (Stations 925 to 950) was adjacent to the beginning of the rolled sections. Originally the section from Station 1025 to Station 1033 (adjacent to the end of the rolled sections) had been selected as a control section but was abandoned because rolling was extended to Station 1029.

The information on cracking, profiles, and roughness before and after rolling has already been discussed. These data will not be repeated here, but it must be pointed out that the condition of the pavement after rolling was the condition that influenced the performance of the surfacing in the experimental sections.

The major types of cracking discussed in this report include (a) transverse joint reflection expressed as a percentage of the joints in the old slab; (b) centerline joint reflection expressed as a percentage of the centerline joint length in the old slab; (c) longitudinal widening cracking expressed as a percentage of the maximum possible length of pavement widening (200 ft/station); and (d) transverse cracking extending into the widening expressed as a percentage of the joints in the old slab.

The crack survey data have been reduced to curved line graphs for the purpose of showing the general trends of crack progression with age. This required the striking of averages to establish certain portions of the curves, particularly at the early ages.

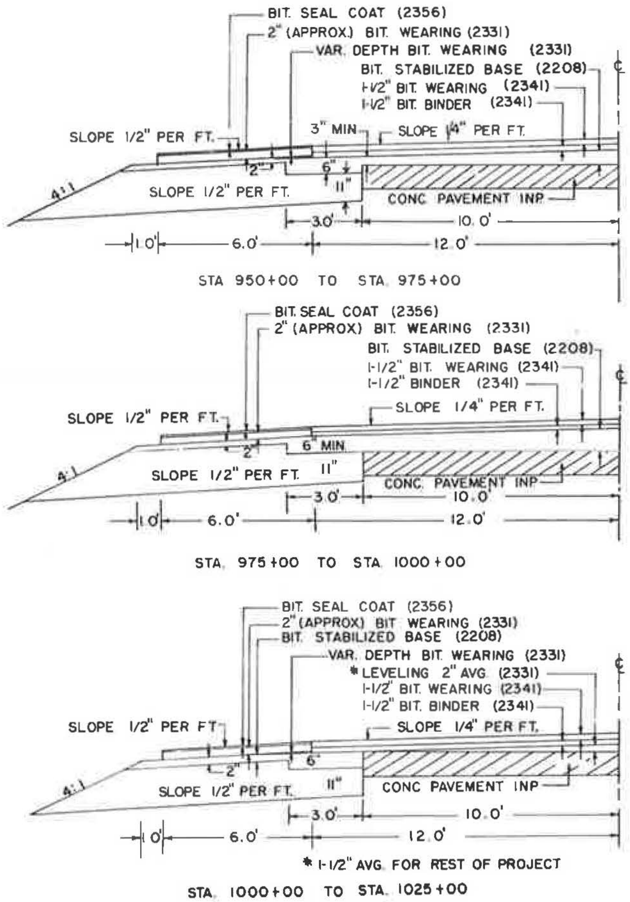


Figure 5. Typical resurfaced cross sections.

However, the curves have been drawn to fit as many points as possible, and generally the points of the curves at 5½ years (when the last survey was taken) coincide with the survey data.

## Transverse Joint Reflection

In control section 1, 98 percent of the transverse joints reflected through the overlay almost immediately after construction. The joint reflection in the other 2 control sections also began almost immediately but progressed at a much slower rate. However, after 2 years, the amount of reflection in these 2 sections was almost as great as in control section 1. These general trends in joint reflection are shown in Figure 6.

Reflection of the transverse joints began approximately 2 months after construction in the 5- and 6-in. rolled overlay sections and 6 months after construction in the 9-in. overlay section. An indication of the effect of rolling on joint reflectance can be seen by comparing the rolled section having a 5-in. overlay with the control sections, which have a 4½-in. overlay. At the end of 5½ years, 70 percent of the joints in the rolled section with a 5-in. overlay had reflected as compared to 97 percent in the unrolled sections. The amount of transverse joint reflection in the 6- and 9-in. overlay sections was slightly less than in the 5-in. overlay section indicating that this type of reflectance was affected somewhat by the thickness of the overlay, but not as much as by rolling.

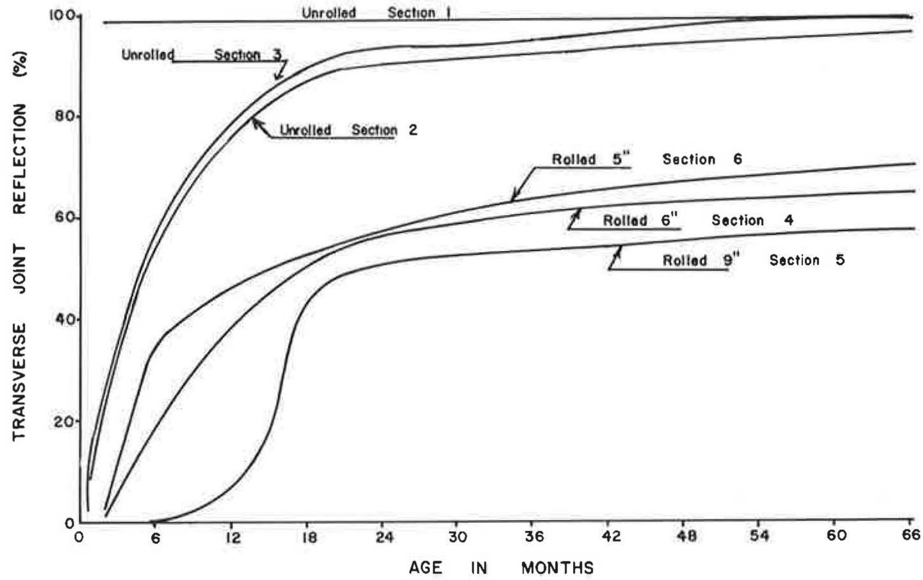


Figure 6. Increase in transverse joint reflection.

Centerline Joint Reflection

In control section 1 the centerline joint began reflecting through the overlay shortly after construction, as shown in Figure 7. The amount of crack reflection increased to 96 percent in 3 years. In the other 5 sections, only a small amount of centerline joint reflectance took place within the first year following construction. However, cracking increased considerably within the next 6 months.

Figure 7 shows that the amount of reflection of the centerline joint was reduced more by increasing the overlay thickness than by rolling the old pavement. At 5½ years after construction, 99 percent of the length of the centerline joint in the control sections (4½-in. overlay) had reflected. The rolled section with the 5-in. overlay had reflected over 95 percent of its length, a reduction of 4 percent from the control sections. The centerline joint of the rolled sections with 6- and 9-in. overlays reflected 83 and 73 percent respectively.

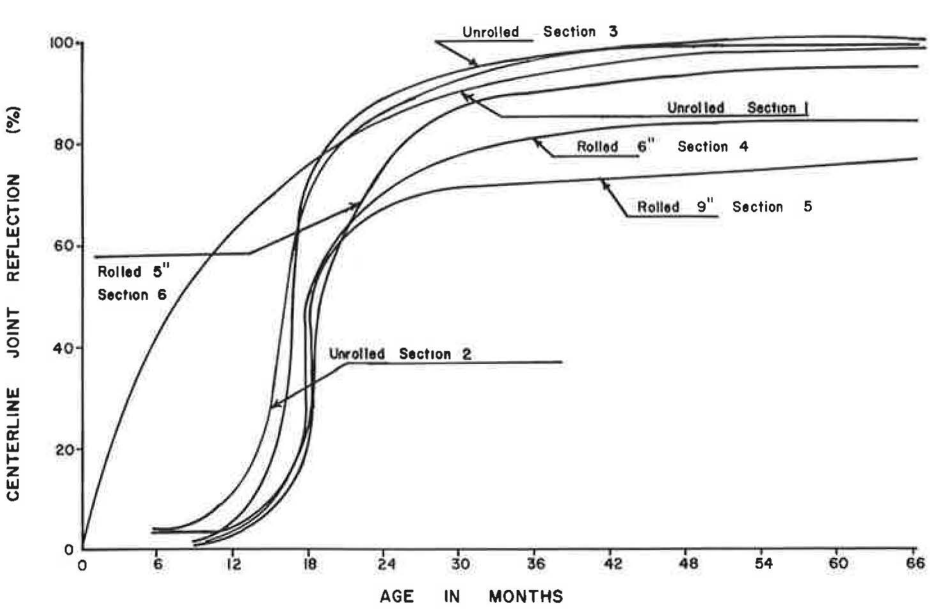


Figure 7. Increase in centerline joint reflection.

## Longitudinal Widening Cracking

The amount of reflection of the longitudinal joint between the bituminous widening and the old concrete pavement varied considerably among the separate sections. This type of cracking started shortly after construction in control section 1, 6 months after construction in control section 3 and in the rolled section with the 5-in. overlay, and 1 year after construction in control section 2.

The data indicate that the amount of longitudinal cracking was reduced by the combination of rolling the old pavement and increasing the thickness of the overlay. The average amount of longitudinal widening joint reflection of the 3 control sections was 48 percent. The amount of reflection in the rolled section with the 5-in. overlay, which is similar in thickness to the control sections, was 19 percent, which is a 60 percent reduction. Very little cracking of this type occurred in the rolled section with the 6-in. overlay and almost none in the rolled section with the 9-in. overlay.

Figure 8 shows the progression of widening joint cracking for all sections.

## Transverse Joints Extending Into the Widening

Transverse joints that extended into the widening began to appear in the 3 control sections and in the rolled section with a 6-in. overlay within a few months after construction, but they were not observed in the other 2 rolled sections until about a year later. The crack growth is shown in Figure 9.

In the 3 control sections, 84 percent of the transverse joints extended into the widening. The rolled section of similar thickness (section 6) had 57 percent transverse joint extension; this is a reduction of 32 percent. The 6-in. rolled section had 12 percent more joints that extended into the widening than the 5-in. rolled section and the 9-in. rolled section had 3 percent fewer. This indicates that the reduction in this type of cracking in the experimental sections was caused by rolling the old pavement rather than by increasing the thickness of the overlay.

## Miscellaneous Transverse Cracking

Some transverse cracks were found, both in the overlay and in the widening, other than those considered under the 4 major types of cracking. These included cracks reflected from old cracks in the concrete pavement, cracks reflected from roller-breaker

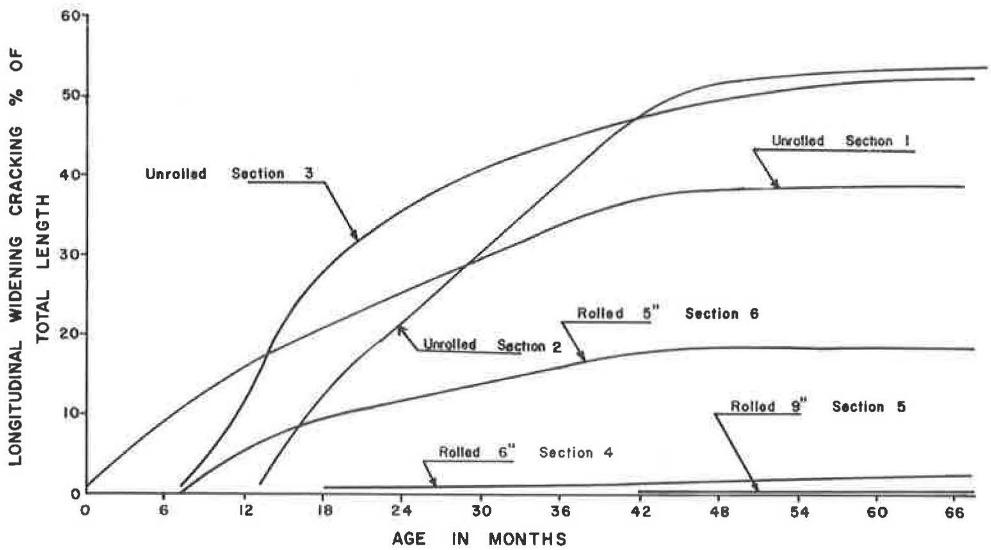


Figure 8. Increase in longitudinal widening cracking.

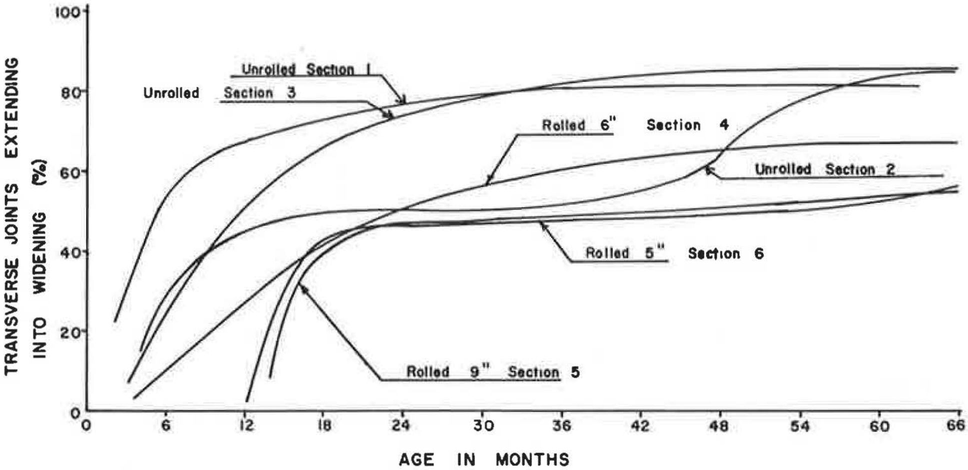


Figure 9. Increase in transverse joints extending into widening.

TABLE 3
ROUGHOMETER MEASUREMENTS

| Section | Roughness (in./mile) | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Oct. 1959 | April 1960 | Nov. 1960 | Feb. 1961 | April 1962 | May 1963 | April 1964 | May 1965 |
| Project avg | 56 | 60 | 55 | 58 | 64 | 64 | 74 | 72 |
| 1 (control) | 54 | 59 | 54 | 57 | 64 | 60 | 66 | 64 |
| 2 (control) | 49 | 57 | 54 | 52 | 65 | 63 | 75 | 70 |
| 3 (control) | 51 | 60 | 53 | 57 | 60 | 56 | 75 | 71 |
| 4 (6 in., rolled) | 52 | 57 | 54 | 57 | 63 | 57 | 61 | 61 |
| 5 (9 in., rolled) | 50 | 55 | 49 | 50 | 63 | 55 | 59 | 56 |
| 6 (5 in., rolled) | 50 | 56 | 54 | 57 | 65 | 61 | 67 | 64 |

cracks, and cracks that were completely new. However, the amount of these cracks in both the overlay (22 ft/station in the rolled sections and 14 ft/station in the unrolled sections) and the widening (less than 5 ft/station in both the rolled and unrolled sections) was very small in relation to the total amount of cracking.

## Roughness

Roughness measurements were made 8 times between the completion of the project in 1959 and May 1965 and are given in Table 3. The measurements were made with the Minnesota roughometer, which is similar to the BPR road roughness recorder.

After construction in 1959, the project averaged 56 in./mile. The roughness values appear erratic when compared chronologically; however, these variations may at least in part be attributed to seasonal changes. For this reason the most valid comparisons would be between measurements made at the same time of the year, such as April in 1960, 1962, and 1964.

The project average increased by 4 in./mile between April 1960 and April 1962 and by 10 in./mile from April 1962 to April 1964. A slight reduction was noted from April 1964 to May 1965, but this could be a seasonal change. The average roughness of the control sections was about the same as the project average each time measurements were taken. The roughness of the rolled sections was somewhat lower than the roughness of the project average and the control sections and also seemed to be increasing at a slower rate. A decrease in roughness was also noted as the thickness of the overlay increased.

Rutting

Cross-sectional measurements on the bituminous surface were made on 8 occasions since construction to determine the distortion in the wheel tracks. These measurements were made at 3 locations in each section (except section 3) by using a leveled straightedge and a scale.

No rutting occurred in control section 1. No rutting was present in control section 2 in April 1960, but slight rutting was observed in July 1960 in the eastbound lane at Station 802; and by October 1960, there was shallow rutting (less than 0.25 in.) in both lanes at each of the 3 locations. There had been no noticeable change in the depth of rutting as of the October 1964 survey.

No rutting was present in any of the rolled sections in July 1960. By October 1960, shallow rutting (0.25 in.) had occurred at all 3 locations in the section with the 5-in. overlay. By September 1961, this rutting had increased somewhat. Slight rutting was found at 2 locations in the section with the 6-in. overlay in October 1960 and at the third location in September 1961. Later surveys indicated that no noticeable change in rut depth occurred at any of the 3 locations. No rutting occurred in the section with the 9-in. overlay.

## COST COMPARISON

Construction costs of the various sections were computed on the basis of job average data. All items pertaining to construction from shoulder to shoulder, including the excavation for the widening, were taken into account. Items such as culvert extensions, subgrade treatments, sodding, and seeding were not included. On this basis the cost of the standard (control) section was $38,340 per mile. The costs of the other sections were increasingly greater as the thickness of the bituminous mixtures was increased

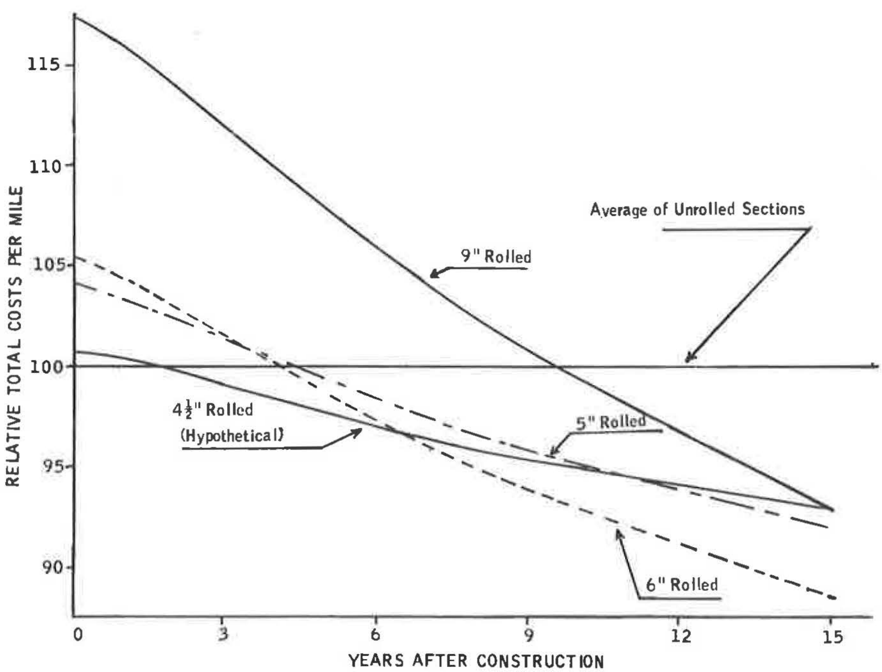


Figure 10. Comparison of construction plus estimated maintenance costs.

as follows: 5-in. overlay, \$39,908; 6-in. overlay, \$40,427; and 9-in. overlay, \$45,001. The costs of these latter sections included \$271.44 per mile for the pavement breaker-rolling.

Based on these construction costs, the standard section was the most economical. However, a valid cost comparison should include an item for maintenance. The major maintenance required on this project was crack-filling. No records were kept of this work, so the following figures are theoretical estimates only. The crack-filling materials currently used are normally effective in sealing cracks for no more than 1 winter in Minnesota. Therefore, it is usually desirable to fill them annually, although this is not usually accomplished. The present cost of crack-filling operations, including materials, equipment, and labor, is approximately 10 cents/linear ft. By using these criteria, construction plus estimated maintenance (crack-filling) costs were computed for each section and expressed as a percentage of the costs of the control sections as shown in Figure 10. The values at zero years are the relative construction costs. These cost ratios were extended to 15 years inasmuch as this approximates the service life of this type of overlay. Figure 10 shows that, approximately 5 years after construction, the additional construction costs of the 5- and 6-in. rolled sections are offset by a savings in maintenance costs, the latter being the more economical. The additional construction costs of the 9-in. rolled section would not be recovered for approximately 10 years. It appears that, even if the service life of the overlays was considerably longer than the estimated 15 years, the 6-in. rolled section would still be more economical than the 9-in. rolled section.

An analysis of Figures 6 through 9 indicates that pavement breaker-rolling reduced the various types of crack reflection by approximately the following amounts: transverse joint reflectance, 25 percent; centerline joint reflectance, 0 percent; longitudinal widening cracking, 45 percent; and transverse joints extending into the widening, 25 percent. On this basis, the relative construction plus estimated maintenance (crack-filling) costs of a 4½-in. rolled section was also computed and is shown in Figure 10. A comparison between these costs and those of the 4½-in. unrolled section indicates that the cost of pavement breaker-rolling is recovered within 2 years as a savings in estimated maintenance costs. After 6 to 7 years, the total costs of this section would exceed those of the 6-in. rolled section, which, based on the assumptions mentioned earlier in this section of the report, was the most economical.

## DISCUSSION AND RECOMMENDATIONS

On this project, most types of crack reflectance were significantly reduced by pavement breaker-rolling. This resulted in only a slight increase in initial costs and an overall savings when estimated maintenance costs are included. It also produced a smoother, more maintenance-free surface.

Pavement breaker-rolling was done on a portion of another widening and resurfacing project on T. H. 12 between Howard Lake and Montrose. This project was identical to that on T. H. 212 except that the thickness of the overlay was uniformly 3 in. plus a thin leveling course. The 10 passes of the 59-ton roller produced no visible cracking, and observations indicated no apparent reduction in crack reflectance.

The difference in results between the project on T. H. 212 and the project on T. H. 12 was probably due to the difference in the condition of the old slab. It seems that a slab will break more easily if it has a great deal of vertical misalignment at the joints and probably voids beneath the slab. It is possible that additional rolling may have caused cracking on the T. H. 12 project.

Even though it appears that pavement breaker-rolling will not always produce the same results as found on the T. H. 212 project, the cost of this operation is so small in comparison with possible savings in maintenance cost that its use on all projects involving the widening and bituminous resurfacing of old concrete pavements is recommended. On overlay projects in which no widening sections or bituminous shoulders are placed, it is questionable whether the use of a pavement breaker-roller could be economically justified because the only savings in maintenance cost would be due to a possible reduction in transverse joint reflectance. However, consideration should be given to the fact that other types of benefits, such as a smoother roadway, might be derived.

From this study it was concluded that the most economical design is a 6-in. rolled section. This was based on the assumption that cracks are filled when needed, which is usually annually. However, during 1967, a new type of hot-pour elastic-type concrete joint filler (MHD Spec. 3723) was used for sealing cracks on several high-type bituminous pavements. Sufficient time has not elapsed to evaluate this material, but supposedly it will perform better than the material previously used. If the crack-filling interval could be extended to 3 years or more, a 4½-in. rolled section might be more economical than a 6-in. rolled section.

## SUMMARY OF FINDINGS

The performance of the various sections was observed for a period of 5½ years following construction. The more important findings from the analysis of the data obtained during this period are as follows:

1. Ten passes of the 59-ton roller provided optimum cracking on this project. An additional 10 passes increased the number of cracks only slightly.
2. Pavement breaker-rolling reduced the amount of transverse joint reflection through the overlay by approximately 25 percent. Increasing the thickness of the hot-mixed bituminous overlay from 5 to 9 in. reduced the amount of this type of crack reflectance only slightly.
3. The amount of reflected transverse cracking other than joint reflection was very minor in both the rolled and unrolled sections.
4. Increasing the thickness of the overlay from 5 to 9 in. reduced the length of centerline joint reflectance approximately 20 percent. Pavement breaker-rolling caused no reduction in the amount of this type of cracking.
5. The amount of reflection of the longitudinal joint between the slab and the widening was reduced by a combination of pavement breaker-rolling and increasing the thickness of overlay. The amount of this type of cracking was reduced by approximately 60 percent when the slab was rolled and the thickness of the overlay increased from 4½ to 5 in.
6. The amount of transverse cracking that extended into the widening was reduced approximately 25 percent by pavement breaker-rolling. The amount of this type of cracking was not reduced by increasing the thickness of overlay.
7. During the 5½-year period in which observations were made, the roughness of the rolled sections was generally lower than the roughness of the project average and the control sections, which were quite similar. Roughness also decreased as the thickness of the overlay increased.
8. Little or no rutting (dishing in wheel track) occurred in any of the rolled or comparison sections.
9. The cost of pavement breaker-rolling was $271.44 per mile, or 0.71 percent of the construction costs ($38,340 per mile) for the standard (4½-in. unrolled) section.
10. After 5 years of service, the total cost per mile (construction plus crack-filling) of the standard section was greater than that of the 5- and 6-in. rolled section.

## CONCLUSIONS

Based on the performance of this project, the following conclusions can be drawn:

1. Pavement breaker-rolling significantly reduces the amount of transverse joint reflection, longitudinal widening joint reflection, and transverse cracking that extends into the widening.
2. Increasing the thickness of hot-mixed overlay from 4½ to 9 in. reduces the amount of centerline joint reflection and longitudinal widening joint reflection.
3. Roughness can be reduced both by pavement breaker-rolling and by increasing the thickness of overlay.
4. The cost of pavement breaker-rolling is a very small percentage of the total construction cost and is undoubtedly offset by savings in crack-filling costs within a short period of time.

5. The additional cost of increasing the thickness of the overlay from $4\frac{1}{2}$ to 9 in. would probably be offset by savings in crack-filling costs. Based on crack-sealing practices currently used in Minnesota, the most economical overlay design seems to be a 6-in. rolled section.

## ACKNOWLEDGMENTS

Progress reports on this study were written earlier by P. G. Velz. The cooperation and assistance of personnel from District 8 who prepared profiles and provided general engineering are gratefully acknowledged.

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# A Pavement Overlay Design System Considering Wheel Loads, Temperature Changes, and Performance

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A recent review of current overlay design procedures revealed that a few bona fide procedures exist and that even the best of them, i. e., those based on deflection, have limitations and inconsistencies. Furthermore, the review emphasized the need for considering volume change stresses. The purpose of this paper, therefore, is to briefly discuss a recently developed overlay design procedure, which has been applied to 11 miles of Interstate highway and 2 airports. A brief description of the overlay design system and its application is presented here; a detailed description is available. The general pavement system is discussed and applied to develop an overlay design system that is applied to the design of an overlay for continuously reinforced concrete pavement in Texas. This design is discussed in terms of wheel load and temperature stresses.

• A RECENT REVIEW OF CURRENT PRACTICES in the United States for the design of overlay pavements for highways (1) indicates that only a few procedures exist in which the overlay design is based on some measure of the response of the existing pavement to load (e. g., deflection), and even in the case of these procedures some question exists as to their general applicability. Moreover, none of the existing procedures considers in a quantitative manner the influence of volume changes in the existing structure on the stresses and hence on the thickness of the overlay pavement, a factor that also would appear to be of significance in determining appropriate thicknesses of overlay pavements (1, 2).

The purpose of this paper is to present the framework for an overlay design procedure (1) in which the conditions of the existing pavement together with the effects of load and volume change are considered in determining the thickness of overlay structure. The procedure is a combination of the recent application of systems engineering to pavement design by Vallerga and McCullough (3), a fatigue system by Kasianchuk and Monismith (8), and the application of stochastic concepts. To illustrate its format, the method is applied to the design of an asphalt concrete overlay for a continuously resurfaced concrete pavement in Texas. It should be noted that the method is of sufficient generality that it can be applied to airfields (3) as well as to highway pavements.

## OVERLAY PAVEMENT DESIGN SYSTEM

Recent studies have emphasized the need for considering pavement design within a systems framework (4, 5, 6, 7). The complexity of such an approach is shown in Figure 1 (5). Figure 1 also shows 3 principal distress mechanisms that can lead to a

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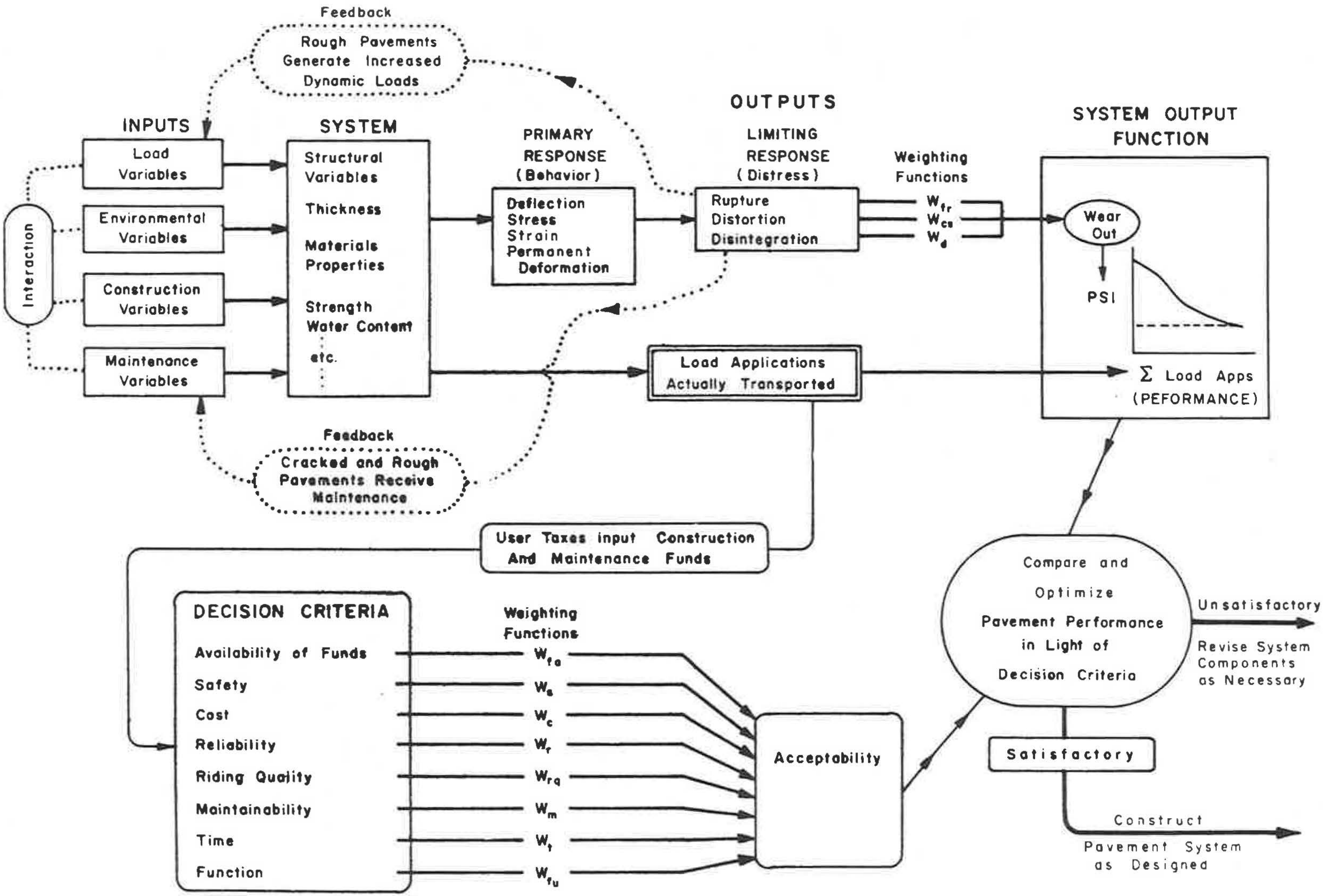


Figure 1. Block diagram of the pavement system (5).

decrease in pavement serviceability with time. Conceptually distress can be expressed as

$$\underline{DI}(\underline{x},t) = \underset{s=o}{\overset{s=t}{\underline{F}}} \; \underline{C}(\underline{x},s),\ \underline{S}(x,s),\ \underline{D}(x,s)\underline{x},t \tag{1}$$

where

$t$ = time;
$x$ = position vector of a point referred to a coordinate system;
$\underline{DI}(\underline{x},t)$ = distress index, a matrix function of space and time;
$\underline{C}(\underline{x},t)$ = measure of fracture, a matrix function of space and time;
$\underline{S}(\underline{x},t)$ = measure of distortion, a matrix function of space and time; and
$\underline{D}(\underline{x},t)$ = measure of disintegration, a matrix function of space and time.

The distress index may then be judged in terms of the decision criteria shown in the lower part of Figure 1. Failure of the system is defined as a condition where the distress from the system output exceeds an acceptable level based on the decision criteria.

Design of overlays may be considered a special case of the pavement system shown in Figure 1. As in the design of a new pavement, the process of attempting to define all of the factors that should be accounted for (Fig. 1) requires a major effort. Accordingly, in this paper only the fracture mode of distress resulting from repeated loading and from volume changes caused by temperature changes will be considered.

A pavement overlay design subsystem to consider these aspects of fracture is shown in Figure 2. The required input parameters are measured deflection, material properties, load variables, construction variables, and environmental variables. Output from this system is the recommended overlay thickness. Brief descriptions for each of the functional operations within the system are also shown in Figure 2. Flow lines with directional arrows connect the various operational steps, thus giving an indication of the procedural order as well as the data required. Generally, the preceding step must be accomplished before proceeding to the next.

For convenience, the design method is divided into 2 parts: one for wheel load stresses and the other for temperature volume change stresses.

The first part (external loading) encompasses 4 phases of operation: (a) determination of material properties utilizing field deflections and laboratory measurements; (b) estimation of the remaining life of the facility considering the past history of loadings during its period of service; (c) estimation of the cumulative stress damage for the future loadings and predicted traffic; and (d) selection of the appropriate thickness of overlay.

The second part (internal loading) of the subsystem consists of computing the overlay thickness required to reduce the volume change stresses induced from internal stresses and from the existing pavement to an acceptable level. Many of the material properties developed in the first phase are used in the second. Although the 2 phases are functionally considered separately, they are in fact dependent phases.

It should also be noted that, although other distress modes are not considered, it is assumed that they will be accounted for by other available procedures or by experience. For example, in the case of an asphalt concrete overlay, densification leading to the distortion (rutting) distress mode can be minimized through proper mix design and construction.

In the following sections a design example is presented to illustrate how the overlay design subsystem (Fig. 2) can be operated in practice and is developed for an asphalt concrete overlay for an existing continuously reinforced concrete pavement. The system is sufficiently general, however, so that it is not limited merely to this set of conditions.

## PROJECT DESCRIPTION

The project for which the procedure was applied consists of a section of Interstate 45 approximately 11 miles in length in Walker County, Texas (Fig. 3). This particular

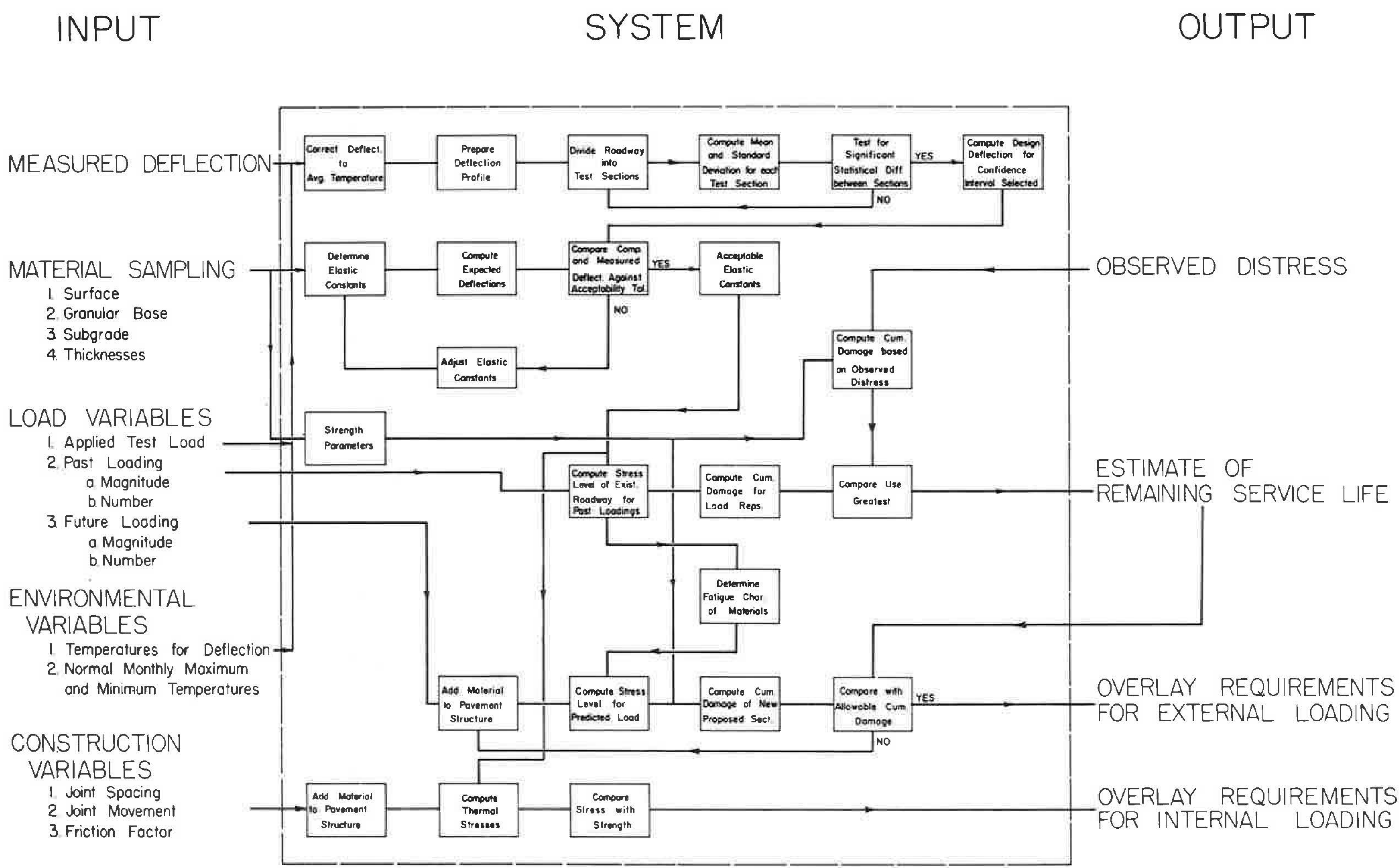


Figure 2. System diagram for overlay design considering the distress mechanisms of excessive loading, fatigue loading, and thermal changes for the fracture mode of failure.

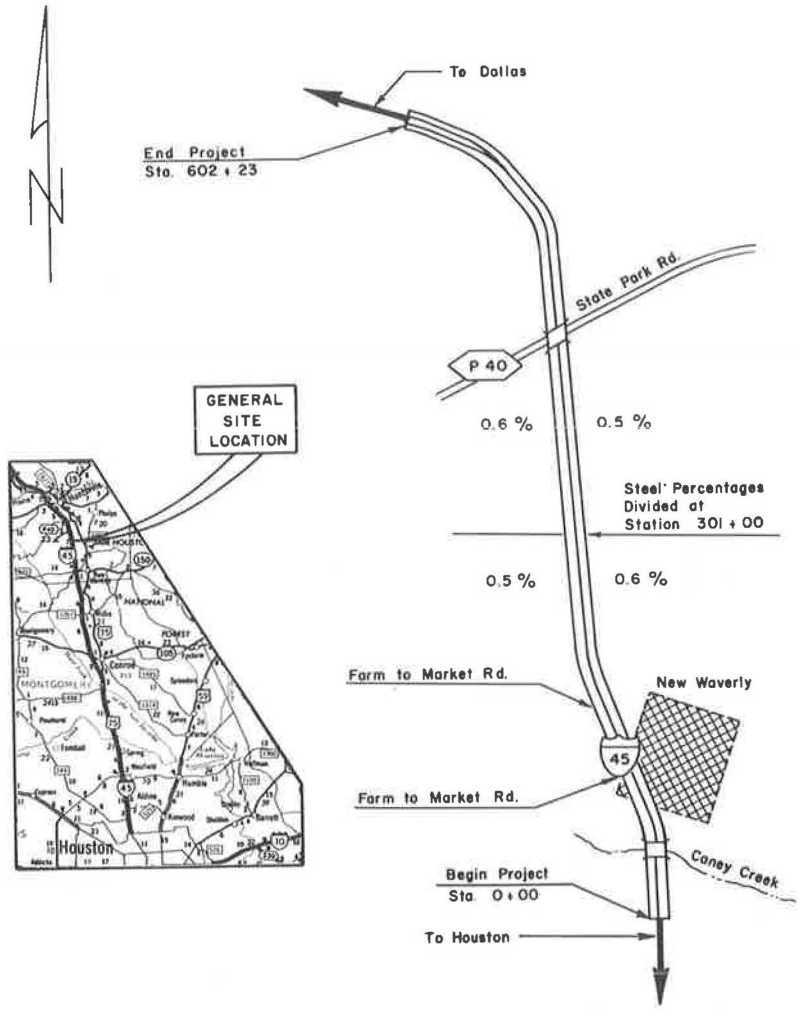


Figure 3. Location and layout of Walker County project.

highway serves as the main artery between the metropolitan areas of Houston and Dallas and accordingly has a high percentage of trucks. The 1969 ADT was 10,020 vehicles, of which 23.2 percent was trucks. A directional distribution of 60 percent southbound and 40 percent northbound was assigned to the truck traffic based on observations of the proportion of loaded trucks moving toward Houston.

The existing pavement, a cross section of which is shown in Figure 4, is a continuously reinforced concrete pavement and was opened to traffic in 1961. Distress of the type shown in Figure 5 has been experienced and is attributed to fatigue of the portland cement concrete resulting from traffic loadings.

## DESIGN EXAMPLE

### Thickness Determination Considering Wheel Loads

As noted earlier (Fig. 2), 4 general steps are followed in determining the overlay thickness required to accommodate the effects of traffic loading. These are considered in sequence in this section. Input data required for this phase include measured surface

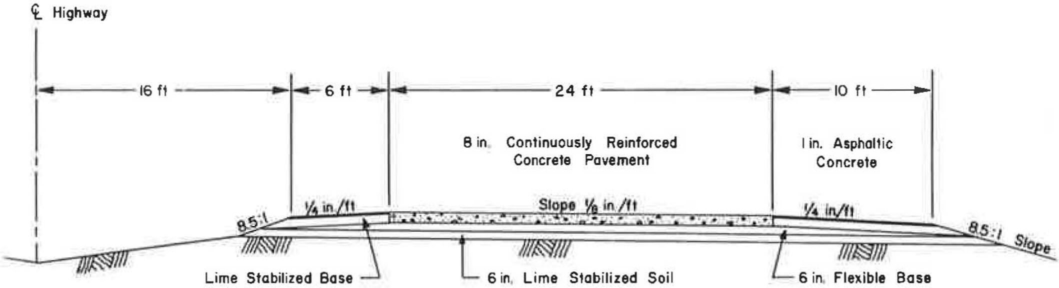


Figure 4. Typical half section for Walker County project.

deflections, material characteristics (pavement and subgrade materials), traffic data including axle load groupings together with repetitions associated with each grouping, environmental data (primarily temperature), construction variables, and observation of distress.

Determination of Properties of Materials—The deflection profile as determined by the Dynaflect was plotted, and these data were used along with prior knowledge of the project soil conditions to divide the roadway into 14 different sections. (Dynaflect loading was 500 lb per wheel on dual wheels with a contact pressure of 170 psi.) The same limits were used for both the northbound and southbound roadways. The limits of the test section recorded in the direction of traffic for each roadway are given in Table 1. Note that the lengths for various test sections range from approximately 0.3 to 1.5

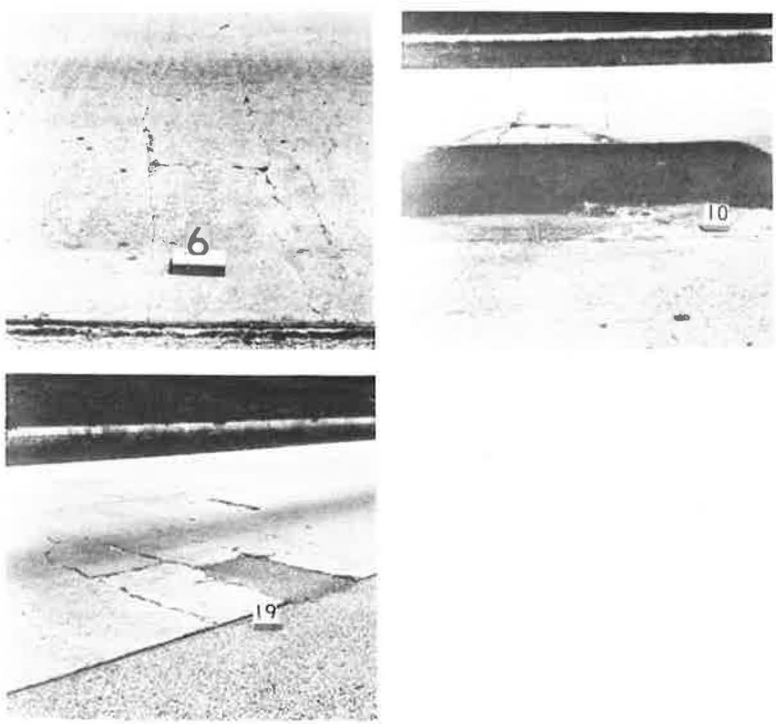


Figure 5. Typical distress areas on the Walker County project.

miles. The overlay indicated in the table refers to special test sections and are not included in this analysis.

Design Deflection—With the established test section limits, the estimated means of standard deviations of the deflections were computed separately for the test section in each roadway and also for the combined requirements for both roadways for a given test section; the results of these computations are given in Table 2. The student t-value between adjacent test sections was then determined. A significance level of 10 percent was selected as the criterion for judging the performance difference between adjacent sections.

A risk is accepted that the inference could be wrong in 10 percent of the cases. The hypothesis that the means are equal was tested against the alternate hypothesis that they are unequal in accordance with the following:

TABLE 1

LIMITS OF TEST SECTIONS USED IN ANALYSIS

| Test Section | Station Numbers[a] Northbound | Station Numbers[a] Southbound | Length (miles) |
|---|---|---|---|
| 1 | 0-14.5 | 99.5-114.0 | 1.45 |
| 2 | 14.5-22.5 | 91.5-99.5 | 0.80 |
| 3 | 22.5-30.5 | 83.5-91.5 | 0.80 |
| 4 | 30.5-36.5 | 77.5-83.5 | 0.60 |
| 5 | 36.5-43.5 | 70.5-77.5 | 0.70 |
| 6 | 43.5-52.5 | 61.5-70.5 | 0.90 |
| 7 | 52.5-66.5 | 47.5-61.5 | 1.40 |
| 8 | 66.5-74.5 | 39.5-47.5 | 0.80 |
| 9 | 74.5-78.5 | 35.5-39.5 | 0.40 |
| | Overlay | Overlay | 0.90 |
| 10 | 87.5-94.5 | 19.5-26.5 | 0.70 |
| 11 | 94.5-100.5 | 13.5-19.5 | 0.60 |
| 12 | 100.5-107.5 | 6.5-13.5 | 0.70 |
| 13 | 107.5-110.5 | 3.5-6.5 | 0.30 |
| 14 | 110.5-114.0 | 0-3.5 | 0.35 |

[a]In 0.1 mile units.

$$P\{|t| > a\} = 5 \text{ percent} \tag{2}$$

where

t = student t-value for hypothesis testing, and
a = value from standard tables for a given significance level.

The results of the iterative process for hypothesis testing of the combined data for the northbound and southbound roadways are given in Table 3. The first iteration shows 11 different areas where the deflection performance is significantly different. Deflection data from adjacent test sections whose means were not significantly different were then combined and new statistics computed. The third and final iteration indicated 9 different sections, these being test sections 1 through 8 as originally selected, and a test section encompassing the original test sections 9 through 14.

Although these procedures serve to illustrate the method by which test sections can be established, in this design example it was also necessary to consider distress that

TABLE 2

MEAN AND STANDARD DEVIATION OF DEFLECTIONS AT TEST SECTIONS MEASURED UNDER DYNAFLECT LOADING

| Test Section | Northbound and Southbound | | Northbound | | Southbound | |
|---|---|---|---|---|---|---|
| | Mean (in. × $10^{-3}$) | Standard Deviation (in. × $10^{-3}$) | Mean (in. × $10^{-3}$) | Standard Deviation (in. × $10^{-3}$) | Mean (in. × $10^{-3}$) | Standard Deviation (in. × $10^{-3}$) |
| 1 | 0.7664 | 0.1039 | 0.7764 | 0.0843 | 0.7564 | 0.1212 |
| 2 | 0.6893 | 0.0624 | 0.6475 | 0.0412 | 0.7312 | 0.0519 |
| 3 | 0.7850 | 0.1637 | 0.8725 | 0.1808 | 0.6975 | 0.0854 |
| 4 | 0.6050 | 0.1300 | 0.5450 | 0.0548 | 0.6650 | 0.1600 |
| 5 | 0.7730 | 0.0927 | 0.7700 | 0.0990 | 0.7766 | 0.0943 |
| 6 | 0.8672 | 0.1385 | 0.8188 | 0.0840 | 0.9155 | 0.1542 |
| 7 | 0.7514 | 0.1174 | 0.6850 | 0.0752 | 0.8230 | 0.1144 |
| 8 | 0.9218 | 0.1992 | 0.8162 | 0.1766 | 1.0275 | 0.1688 |
| 9 | 0.7012 | 0.1280 | 0.6725 | 0.1868 | 0.7300 | 0.0346 |
| 10 | 0.6946 | 0.1949 | 0.6450 | 0.1462 | 0.7533 | 0.2406 |
| 11 | 0.5969 | 0.1307 | 0.4933 | 0.0547 | 0.6857 | 0.1086 |
| 12 | 0.7307 | 0.1220 | 0.6828 | 0.0707 | 0.7785 | 0.1483 |
| 13 | 0.6566 | 0.1039 | 0.5866 | 0.0346 | 0.7266 | 0.1048 |
| 14 | 0.7925 | 0.1723 | 0.8500 | 0.2404 | 0.7350 | 0.1345 |

was observed. If distress is not observed, the delineation of test sections can be based primarily on the type of analysis given in Table 3.

Varying amounts of distress present in both roadways required that each roadway be treated independently, and an analysis similar to that given in Table 3 for the separate roadways indicated that in all but a few cases the test sections were significantly different. Therefore, because the amount of distress also varied among test sections, the limits given in Table 1 and the statistical parameters given in Table 2 were retained.

After establishing the test sections, a design deflection was determined for each of the areas. The design deflection is used as a representative deflection for each of the test sections. For this analysis a significance level of 1 percent was selected. By assuming that stress in the pavement is proportional to deflection, the use of such a low value would ensure that the stress associated with the design deflection would thus be exceeded only 1 percent of the time. The computed values of the design deflection associated with the Dynaflect utilizing data given in Table 2 are given in Table 4 for both the northbound and southbound roadways.

TABLE 3

SIGNIFICANCE STUDY FOR COMBINED DATA FROM THE 2 ROADWAYS

| Test | n | t | a | Significance |
|---|---|---|---|---|
| First Iteration | | | | |
| 1 vs 2 | 34 | 3.003 | 1.692 | Different |
| 2 vs 3 | 32 | 3.041 | 1.695 | Different |
| 3 vs 4 | 28 | 3.024 | 1.701 | Different |
| 4 vs 5 | 25 | 3.647 | 1.708 | Different |
| 5 vs 6 | 31 | 2.061 | 1.696 | Different |
| 6 vs 7 | 45 | 2.945 | 1.680 | Different |
| 7 vs 8 | 43 | 3.455 | 1.682 | Different |
| 8 vs 9 | 24 | 2.729 | 1.711 | Different |
| 9 vs 10 | | | | Same |
| 10 vs 11 | 26 | 2.081 | 1.706 | Different |
| 11 vs 12 | 27 | 0.389 | 1.703 | Same |
| 12 vs 13 | 20 | 1.232 | 1.725 | Same |
| 13 vs 14 | 10 | 1.387 | 1.812 | Same |
| Second Iteration for Combining Like Sections and Recomparing | | | | |
| 8 vs 9 | 35 | 3.604 | 1.930 | Different |
| 9-10 vs 11-14 | 58 | 0.445 | 1.671 | Same |
| Third Iteration for Combining Like Sections and Recomparing | | | | |
| 8 vs 9-14 | 66 | 2.210 | 1.664 | Different |

Note: All significance tests performed at $\alpha$ = 5 percent.

<u>Prediction of Deflection Profile</u>—The response of the pavement to load was predicted by using layered system elastic theory, which requires a measure of the elastic response of each of the materials comprising the pavement section. Such data based on testing of the various components are shown in Figure 6. Only the subgrade has been assigned a range in values. All other components are represented only by single values.

As indicated earlier, one of the portions in this subsystem requires prediction of measured values of deflections utilizing measured material properties to ensure a compatible system prior to extending the analysis to selection of the overlay thickness. In this instance the data shown in Figure 6 were used to predict the Dynaflect deflections (design values given in Table 4).

Because of the magnitude of this analysis (i.e., the number of test sections), it was not feasible to prepare a solution for each test section independently. Therefore, 4 subgrade moduli values of 3,000, 6,000, 10,000, and 15,000 psi (covering the observed range) were selected to develop a set of general computations. Such a generalized

TABLE 4

DESIGN DEFLECTION VALUES FOR A 99 PERCENT CONFIDENCE LEVEL

| Test Section | Design Deflection (in. x $10^{-3}$) Northbound | Southbound | Test Section | Design Deflection (in. x $10^{-3}$) Northbound | Southbound |
|---|---|---|---|---|---|
| 1 | 0.9649 | 1.0274 | 8 | 1.2111 | 1.4049 |
| 2 | 0.7396 | 0.8472 | 9 | 1.0902 | 0.8074 |
| 3 | 1.2768 | 0.8885 | 10 | 0.9719 | 1.2913 |
| 4 | 0.6675 | 1.0228 | 11 | 0.6156 | 0.9285 |
| 5 | 1.2128 | 0.9875 | 12 | 0.8409 | 1.1101 |
| 6 | 1.0086 | 1.2603 | 13 | 0.6640 | 0.9609 |
| 7 | 0.8554 | 1.0788 | 14 | 1.3875 | 1.0357 |

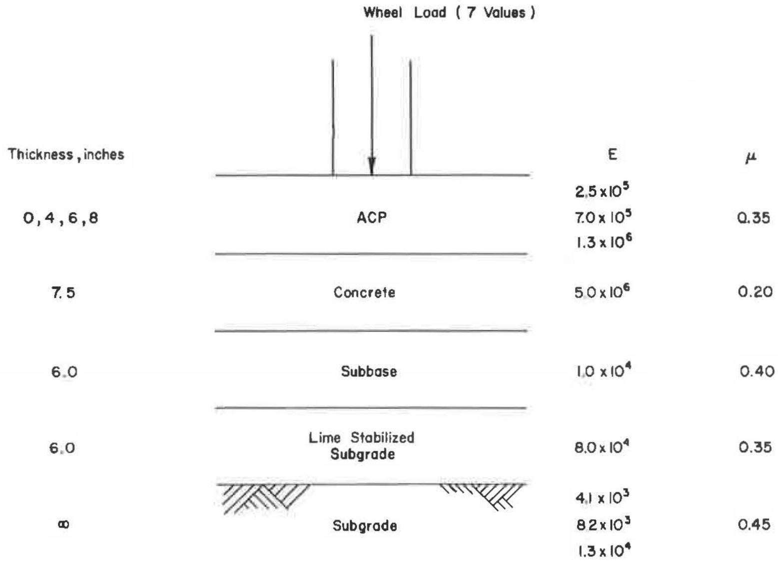


Figure 6. Material properties and geometry of the pavement structure.

approach permits interpolation for a specific test section. Deflections were computed for average measured values of pavement thickness and for (±) 1 and 2 standard deviations from the average. Results of these analyses showing deflections in terms of subgrade modulus for various pavement thickness are shown in Figure 7.

Material Properties—Material properties representative of each test section were established by using stochastic principles. For purposes of analysis, the variation of deflection within a test section (as represented by the standard deviations given in Table 2) is attributed to variations in both subgrade modulus and concrete pavement thickness. The other factors shown in Figure 6 were assumed to be constant as noted earlier. Within this framework a variation may be assigned to each factor and acceptable minimum limits established as follows:

$$P(\sigma_i > \sigma_{max}) = P(E_5 < b_{E_5}) \times P(D_1 < b_{D_1}) \tag{3}$$

where

$\sigma$ = tensile stress in concrete resulting from an applied load;
$E_5$ = subgrade modulus;
$b_{E_5}$ = limiting value of modulus of elasticity for the subgrade;
$D_1$ = thickness of portland cement concrete layer; and
$b_{D_1}$ = limiting value of thickness of portland cement concrete layer.

Because the design deflection was computed for a confidence level of 1 percent, the product of Eq. 3 should be equal to 1 percent. For this problem, equal weights were assigned to the variables of thickness and subgrade modulus, thus the confidence level for selecting the minimum values of these 2 factors was set at 10 percent. The variation of pavement thickness on this project was ascertained through a series of core measurements (Appendix, Table 12), but the variation of subgrade modulus is an unknown quantity that must be established from the deflection data. The standard deviation for the measured pavement thickness and a significance level of 10 percent were used to obtain an acceptable minimum thickness of 7.75 in. Having established this minimum thickness, we determined the minimum allowable value for the subgrade modulus (Fig. 8).

More specifically, the design value for a test section is entered on the deflection scale and projected horizontally across to the minimum acceptable pavement thickness

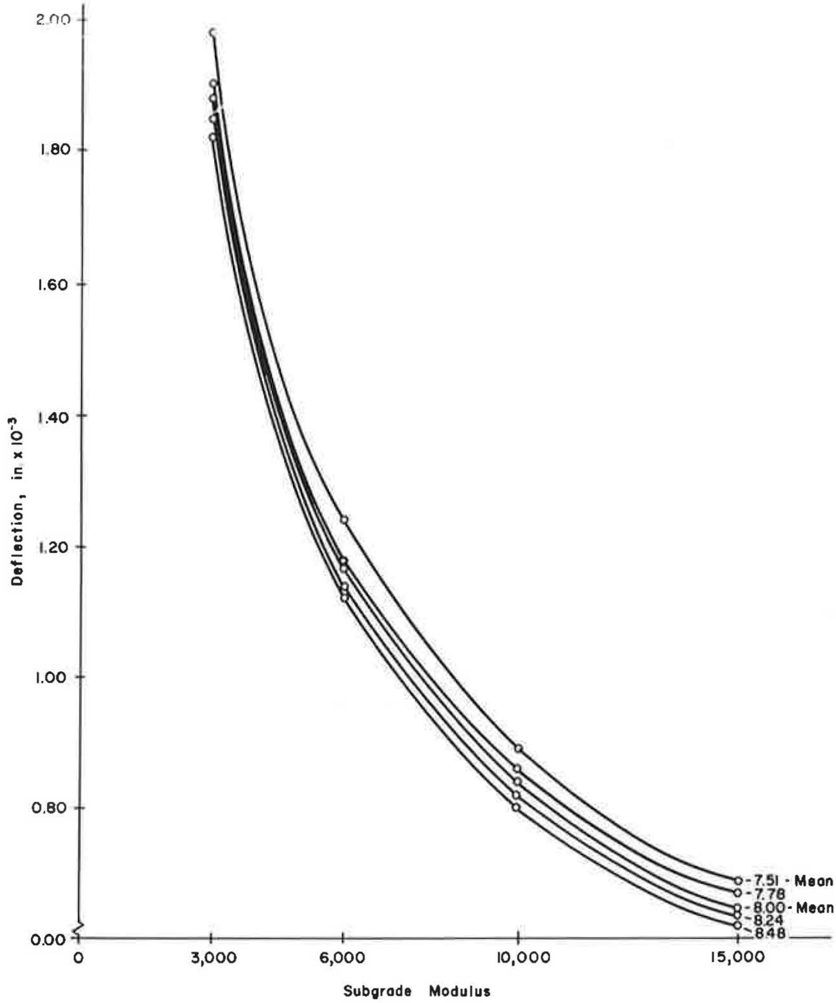


Figure 7. Computed deflection as a function of subgrade modulus and pavement thickness.

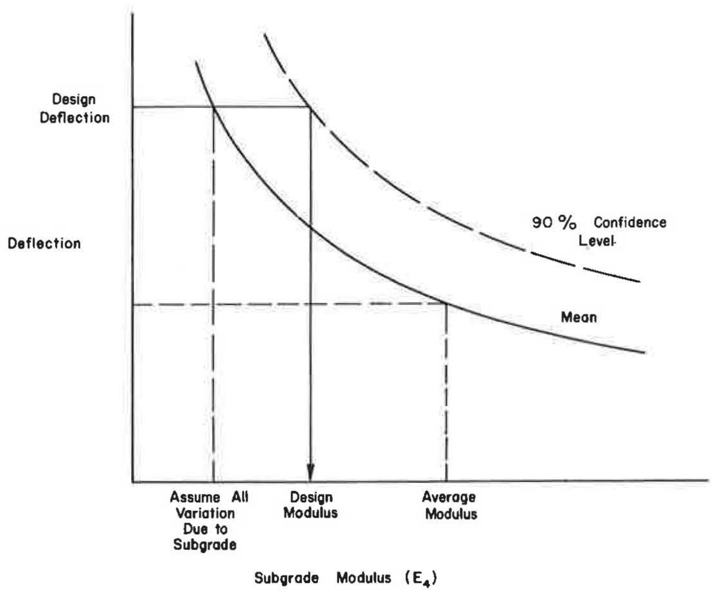


Figure 8. Procedure used to determine the subgrade modulus accounting for variations in deflection and pavement thickness.

TABLE 5

DESIGN SUBGRADE MODULUS VALUES DERIVED FROM DESIGN DEFLECTIONS

| Test Section | Northbound (psi) | Southbound (psi) | Test Section | Northbound (psi) | Southbound (psi) |
|---|---|---|---|---|---|
| 1 | 8,850 | 8,000 | 8 | 6,200 | 5,000 |
| 2 | 13,500 | 10,850 | 9 | 7,300 | 11,750 |
| 3 | 5,700 | 10,100 | 10 | 8,650 | 5,700 |
| 4 | 15,800 | 8,100 | 11 | 20,000 | 9,350 |
| 5 | 6,200 | 8,500 | 12 | 11,000 | 7,150 |
| 6 | 8,250 | 5,900 | 13 | 16,300 | 9,200 |
| 7 | 10,700 | 7,900 | 14 | 5,100 | 7,850 |

and then to the subgrade modulus value on the abscissa. This value is the design subgrade modulus for the test section. Changes in deflection caused by variations in subgrade modulus and pavement thickness are pictorially represented in the figure. An examination of the figure also indicates that the design value of the subgrade modulus is less than average but greater than the minimum established by assuming that all variation in deflection is attributable to variation in the subgrade modulus. (The problems of designing with average or maximum values are obvious because in the first instance 50 percent of the area would have a higher stress and in the latter the design would be extremely conservative.)

The design values of the subgrade modulus so established are given in Table 5 and are based on the design deflections given in Table 4.

Estimation of Remaining Life—As shown in Figure 2, with the material properties and other characteristics of the pavement established, it is then possible to estimate the stresses or strains or both in the existing pavement resulting from wheel loads of differing intensities. By combining these data with fatigue data for the paving material under consideration, it is then possible to estimate the damage accumulated in the pavement caused by past traffic (8). This section illustrates a method by which this estimate may be accomplished.

Stress Computations—Stresses in the pavement were computed by using elastic layer theory. (The pavement structure under consideration is a continuously reinforced concrete pavement, and the joints are not a consideration in stress determinations; accordingly existing elastic layer theory provides a reasonable engineering analysis of the situation.) Tangential stresses on the underside of the concrete pavement were computed for 13 different wheel load values (Table 6) using a tire pressure of 70 psi. Stresses under the tandem axles were determined for an axle spacing of 49 in. and the maximum stress from both wheels obtained by superposition. The results of these computations are given in Table 6.

TABLE 6

TANGENTIAL STRESSES IN THE EXISTING PAVEMENT FOR VARIOUS SUBGRADE MODULUS AND WHEEL LOAD VALUES

| Wheel Load (lb) | Subgrade Modulus | | |
|---|---|---|---|
| | 4,100 psi (psi) | 8,200 psi (psi) | 13,000 psi (psi) |
| Single Axle | | | |
| 2,650 | 79 | 71 | 67 |
| 4,500 | 120 | 110 | 106 |
| 7,200 | 175 | 164 | 157 |
| 8,700 | 204 | 191 | 183 |
| 9,600 | 220 | 207 | 198 |
| 10,700 | 240 | 225 | 215 |
| Tandem Axle | | | |
| 5,300 | 90 | 78 | 73 |
| 9,000 | 138 | 123 | 116 |
| 14,400 | 203 | 184 | 173 |
| 17,400 | 238 | 215 | 202 |
| 19,200 | 258 | 234 | 219 |
| 21,400 | 282 | 255 | 238 |
| 2,500 | 320 | 289 | 269 |

Equation 3 was used to obtain a statistical level of confidence for the stresses; because material properties and pavement thickness were input with a significance level of 10 percent, the probability that the stresses computed for each test section and wheel load would be exceeded is only 1 percent.

Cumulative Damage Computations—Stresses given in Table 6 were used together with concrete fatigue data and past wheel load repetitions (supplied by the Texas Highway Department) as input data

TABLE 7

ESTIMATE OF THE CUMULATIVE DAMAGE AND REMAINING LIFE FOR THE VARIOUS SUBGRADE VALUES

| Roadway | Subgrade Modulus (psi) | Cumulative Damage[a] for a Design Confidence Level | | | | | Remaining Life for a Design Confidence Interval | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Avg | 15 Percent | 10 Percent | 5 Percent | 1 Percent | Avg | 15 Percent | 10 Percent | 5 Percent | 1 Percent |
| NB | 4,100 | 2.346 - 07 | 9.393 - 03 | 1.087 - 01 | 2.860 + 01 | 2.582 + 04 | 1.0000 | 0.9906 | 0.8913 | | 0 |
| NB | 8,200 | 7.214 - 09 | 2.889 - 04 | 3.342 - 03 | 8.795 - 01 | 7.932 + 02 | 1.0000 | 0.9997 | 0.9967 | 0.1205 | 0 |
| NB | 13,000 | 6.299 - 10 | 2.522 - 05 | 2.918 - 04 | 7.679 - 02 | 6.932 + 01 | 1.0000 | 1.0000 | 0.9997 | 0.9232 | 0 |
| SB | 4,100 | 3.519 - 07 | 1.469 - 02 | 1.630 - 01 | 4.290 + 01 | 3.872 + 04 | 1.0000 | 0.9859 | 0.8370 | 0 | 0 |
| SB | 8,200 | 1.082 - 08 | 4.333 - 04 | 5.013 - 03 | 1.319 + 00 | 1.191 + 03 | 1.0000 | 0.9996 | 0.9956 | 0 | 0 |
| SB | 13,000 | 9.448 - 10 | 3.783 - 05 | 4.377 - 04 | 1.152 - 01 | 1.040 + 02 | 1.0000 | 1.0000 | 0.9996 | 0.8848 | 0 |

[a]Expressed in exponential form.

for the fatigue program. A uniform traffic distribution was assumed, i.e., one-twelfth of the total traffic assigned to each month. Cumulative damage computations utilizing the linear summation of the cycle ratios procedure were made at the average value for concrete strength as well as for confidence levels of 1, 5, 10, and 15 percent. Results of these analyses are given in Table 7 for the various confidence levels. The data given in Table 7 are the output of a computer program for cumulative damage originally developed in another investigation (8).

The data are presented for a range in subgrade moduli inasmuch as the stress data were input in this form.

Values for the remaining life may be obtained by subtracting the cumulative damage from unity. Note that the cumulative damage value increases (remaining life decreases) as the confidence value is reduced or as the subgrade modulus decreases.

Remaining Life Estimation—From an examination of the subgrade modulus data given in Table 5 and the observed distress data (Appendix, Table 13), it was noted that varying degrees of distress were observed for sections that had the same subgrade modulus and would therefore be considered equal. These apparent discrepancies were assumed to be the result of localized areas where differences in the standard deviation of the flexural strength are greater or less than the value derived for the total project.

Table 8 gives a summary of the various test sections together with their remaining lives estimated by this procedure. These test sections were obtained by combining the test section limits from the distress analysis with the limits from the deflection analysis given in Table 1. In cases where the test section limits from each analysis did not coincide, the test sections from the deflection analysis were subdivided based on the limits indicated by the distress analysis and assigned letter designations, e.g., test sections 7A and 7B.

TABLE 8

ESTIMATED FLEXURAL STRENGTH STANDARD DEVIATION AND REMAINING LIFE FOR THE ESTABLISHED TEST SECTIONS

| Test Section[a] | Estimated Standard Deviation for Flexural Strength (psi) | Remaining Life[b] |
|---|---|---|
| | Northbound | |
| 1 | 129 | 0.757 |
| 2 | 143 | 0.939 |
| 3 | 116 | 0.773 |
| 4 | 116 | 0.940 |
| 5 | 100 | 0.930 |
| 6 | 106 | 0.973 |
| 7A | 113 | 0.957 |
| 7B | 192 | 0.971 |
| 8 | 167 | 0.946 |
| 9 | 174 | 0.961 |
| 10 | 250 | 0.915 |
| 11 | 200 | 0.950 |
| 12 | 122 | 0.620 |
| 13 | 110 | 0.980 |
| 14 | 100 | 0.913 |
| | Southbound | |
| 1 | 140 | 0.419 |
| 2A | 124 | 0.947 |
| 2B | 247 | 0.840 |
| 2C | 247 | 0.700 |
| 3A | 228 | 0.706 |
| 3B | 121 | 0.940 |
| 4 | 217 | 0.790 |
| 5 | 220 | 0.780 |
| 6A | 204 | 0.760 |
| 6B | 201 | 0.660 |
| 7A | 300(+) | 0.660 |
| 7B | 217 | 0.840 |
| 8A | 229 | 0.580 |
| 8B | 300(+) | 0.550 |
| 9 | 278 | 0.740 |
| 10 | 132 | 0.770 |
| 11 | 148 | 0.840 |
| 12 | 139 | 0.730 |
| 13 | 147 | 0.850 |
| 14 | 142 | 0.909 |

[a]The A, B, and C designations are subdivisions where observed distress was significantly different.

[b]Based on allowing 1 percent additional distress during the design life.

Prediction of Future Life—Computations for prediction of future life were performed in the same manner as that described in the previous section, i.e., by utilizing estimated stresses and the same cumulative damage hypothesis.

Stress Computations—As before, computations for stress were made in a general format because of the large number values. This was accomplished by formulating a 3-3-3-13 factorial to encompass the known variation in subgrade modulus, asphalt concrete stiffness, pavement thickness, and wheel load. Included were (a) subgrade moduli of 4,100, 8,200, and 13,000 psi; (b) stiffness values of 250,000, 750,000, and 1,300,000 psi for the asphalt concrete overlay; (c) overlay thicknesses of 4, 6, and 8 in.; and (d) the 13 loads given in Table 6. Results of these stress computations for a subgrade moduli of 4,100 psi and an overlay thickness of 4 in. are given in Table 9. An earlier paper (1) contains additional data for other combinations.

TABLE 9

TANGENTIAL STRESSES IN THE PORTLAND CEMENT CONCRETE PAVEMENT FOR THE DESIGN WHEEL LOADS AND ASPHALT CONCRETE STIFFNESSES

| Wheel Load (lb) | Axle Type | Aspahlt Concrete Stiffness 250,000 psi (psi) | 700,000 psi (psi) | 1,300,000 psi (psi) |
|---|---|---|---|---|
| 2,650 | Single | 59 | 57 | 49 |
| 4,500 | Single | 92 | 83 | 76 |
| 7,200 | Single | 137 | 123 | 111 |
| 8,700 | Single | 161 | 144 | 130 |
| 9,600 | Single | 175 | 157 | 141 |
| 10,700 | Single | 193 | 172 | 155 |
| 5,300 | Tandem | 69 | 64 | 58 |
| 9,000 | Tandem | 109 | 99 | 91 |
| 14,400 | Tandem | 164 | 149 | 136 |
| 17,400 | Tandem | 194 | 175 | 160 |
| 19,200 | Tandem | 211 | 191 | 174 |
| 21,400 | Tandem | 233 | 210 | 192 |
| 2,500 | Tandem | 267 | 221 | 219 |

Note: Data are for a subgrade modulus of 4,100 psi and an overlay thickness of 4 in.

Cumulative Damage Computation—The general stress data output from the previous section was then used as input to the fatigue program and cumulative damage determined. In this determination, the primary differences in input when compared to the analysis of the existing pavement are the stress levels for the selected asphalt concrete pavement stiffnesses and the predicted wheel load repetitions. In addition, it must be recognized that the average stiffness of asphalt concrete varies because the average

TABLE 10

DESIGN SUMMARY OF CUMULATIVE DAMAGE FOR VARIOUS OVERLAY THICKNESSES

| Subgrade Modulus (psi) | Overlay Thickness (in.) | Cumulative Damage[a] for a Confidence Interval of Design: Avg | 15 Percent | 10 Percent | 5 Percent | 1 Percent |
|---|---|---|---|---|---|---|
| Northbound | | | | | | |
| 4,100 | 4 | 4.266 - 10 | 1.708 - 05 | 1.976 - 04 | 5.201 - 02 | 4.695 + 01 |
| 4,100 | 6 | 8.361 - 12 | 3.348 - 07 | 3.873 - 06 | 1.019 - 03 | 9.201 - 01 |
| 4,100 | 8 | 1.078 - 13 | 4.315 - 09 | 4.992 - 08 | 1.314 - 05 | 1.186 - 02 |
| 8,200 | 4 | 1.097 - 11 | 4.394 - 07 | 5.084 - 06 | 1.338 - 03 | 1.208 + 00 |
| 8,200 | 6 | 1.603 - 13 | 6.419 - 09 | 7.426 - 08 | 1.954 - 05 | 1.764 - 02 |
| 8,200 | 8 | 2.034 - 15 | 8.146 - 11 | 9.424 - 10 | 2.480 - 07 | 2.239 - 04 |
| 13,000 | 4 | 8.829 - 13 | 3.535 - 08 | 4.090 - 07 | 1.076 - 04 | 9.716 - 02 |
| 13,000 | 6 | 1.278 - 14 | 5.117 - 10 | 5.920 - 09 | 1.558 - 06 | 1.406 - 03 |
| 13,000 | 8 | 1.425 - 16 | 5.706 - 12 | 6.601 - 11 | 1.737 - 08 | 1.568 - 05 |
| Southbound | | | | | | |
| 4,100 | 4 | 6.399 - 10 | 2.562 - 05 | 2.964 - 04 | 7.801 - 02 | 7.042 + 01 |
| 4,100 | 6 | 1.254 - 11 | 5.022 - 07 | 5.810 - 06 | 1.529 - 03 | 1.380 + 00 |
| 4,100 | 8 | 1.616 - 13 | 6.473 - 09 | 7.488 - 08 | 1.971 - 05 | 1.779 - 02 |
| 8,200 | 4 | 1.646 - 11 | 6.592 - 07 | 7.626 - 06 | 2.007 - 03 | 1.812 + 00 |
| 8,200 | 6 | 2.404 - 13 | 9.628 - 09 | 1.114 - 07 | 2.931 - 05 | 2.646 - 02 |
| 8,200 | 8 | 3.051 - 15 | 1.222 - 10 | 1.414 - 09 | 3.720 - 07 | 3.358 - 04 |
| 13,000 | 4 | 1.324 - 12 | 5.303 - 08 | 6.135 - 07 | 1.614 - 04 | 1.457 - 01 |
| 13,000 | 6 | 1.917 - 14 | 7.675 - 10 | 8.879 - 09 | 2.337 - 06 | 2.109 - 03 |
| 13,000 | 8 | 2.137 - 16 | 8.559 - 12 | 9.902 - 11 | 2.606 - 08 | 2.352 - 05 |

[a]Expressed in exponential form.

daily temperature varies from month to month. Thus a different stiffness of asphalt concrete for each month was used as input to the fatigue program. The input for the predicted wheel load repetition was for the design period (in this case to 1981).

The output from this general analysis is given in Table 10 in terms of predicted cumulative damage for various confidence levels, subgrade moduli, and pavement overlay thicknesses.

Overlay Thickness—The data from the estimation of remaining life study and the prediction of future life were used to estimate the required overlay thickness for each test section. A general design graph similar to that shown in Figure 9 was plotted by using the data given in Table 10. Overlay thicknesses were determined as shown in the figure by using the flexural strength standard deviation given in Table 8, the subgrade modulus given in Table 5, and remaining life given in Table 8 for each test section. Results of this study are shown in Figure 10, which shows the overlay thicknesses required along each of the roadways for the predicted traffic. Resulting thicknesses range from 1.5 to 9 in. depending on the subgrade modulus and the distress experienced. Figure 10 shows that the overlay thicknesses for the southbound roadway are greater than those for the northbound roadway. These differences may be attributed to the 60 to 40 traffic split alluded to earlier and to the larger amount of distress experienced in the southbound roadway.

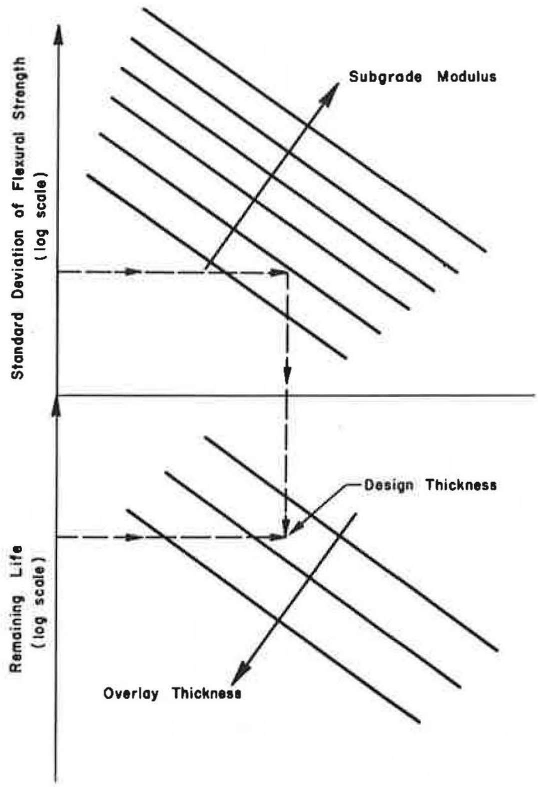


Figure 9. Design nomograph used in determining the required overlay thickness for each test section.

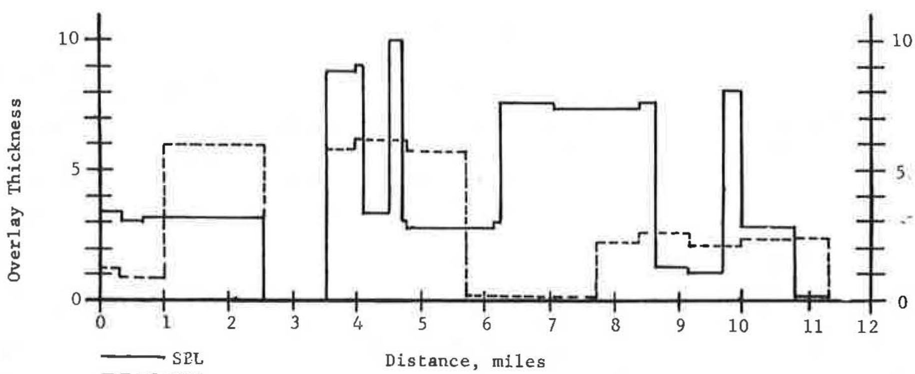


Figure 10. Recommended minimum overlay thickness required along project for predicted traffic assuming 1 percent additional distress.

TABLE 11

MINIMUM OVERLAY THICKNESSES TO REDUCE VOLUME CHANGE STRESSES TO A SATISFACTORY LEVEL

| $T_2$ | Recommended Minimum Overlay Thicknesses (in.) | |
|---|---|---|
| | 0.5 Percent Longitudinal Steel | 0.6 Percent Longitudinal Steel |
| 1.0 | 3.02 | 2.74 |
| 0.75 | 2.27 | 2.05 |

## Thickness Determination Considering Volume Change Stresses

Thickness of asphalt concrete overlay required to reduce or eliminate cracks in the asphalt bound layer caused by volume changes resulting from changes in temperature was estimated from

$$D_A \geq \frac{\left[R_2\left(\frac{0.482}{e^p \times K^{2.03}}\right) \alpha_C \times E_C \times D_1\right] \Delta T}{f_{AC}(T) - \alpha_{AC} \sum_{T_o}^{T_C} S(\Delta T)(\Delta T)} \tag{4}$$

where

$\Delta T$ = temperature change;
$f_{AC}(T)$ = tensile strength of asphalt concrete;
$\alpha_C$ = coefficient of thermal expansion, portland cement concrete;
$\alpha_{AC}$ = coefficient of thermal expansion, asphalt concrete;
$p$ = percentage of longitudinal steel in portland cement concrete;
$e$ = Napierian base;
$R_2$ = factor to account for slippage between asphalt concrete and portland cement concrete; and
$K$ = resistance factor for material supporting concrete pavement.

This is in part based on data obtained from an extensive field study of movements at transverse cracks in continuously reinforced concrete pavements (9).

From a study of temperature data at the site, a temperature change of 50 F was selected for design because the probability of this value being exceeded on any given date is only 2 percent. A temperature of 60 F, representative of the average daily values for January and February when the 50 F change occurs, was used as the base temperature.

The thermal coefficient of $1.3 \times 10^{-5}$ in./in./deg F for the asphalt concrete was not directly determined; rather, it was obtained from the literature (10). An estimated stiffness versus temperature relationship for the asphalt concrete together with the other necessary parameters was used to determine overlay thicknesses from Eq. 4 for a temperature change of 50 F. Results of this analysis are given in Table 11 and range from 2 to 3 in. depending on the assumed slippage coefficient and the percentage of steel in the concrete.

## SUMMARY

The overlay thicknesses resulting from the analyses for wheel load stresses and for volume change stresses are shown in Figure 10 and given in Table 11 respectively. For temperature stresses, it is recommended that the overlay thicknesses corresponding to $R_2 = 0.75$ be used. No specific rationale can be given at this time other than to state that a value of $R_2 = 1.0$ appears to be too conservative based on observations of the special test sections that had been incorporated within the bounds of the project by the highway department.

At this point, the data from the 2 phases must be combined to obtain the recommendation for the total overlay design system shown in Figure 2. The controlling criterion for predicting the overlay thickness for wheel load stresses was the stress in the portland cement concrete at the base of the slab; whereas for the volume change stresses, the stress in the asphalt concrete was the controlling design criterion. Because the critical stresses for the 2 design phases occur in separate layers, a combination of stresses is not required. Therefore, the largest minimum value for thickness from the 2 design phases may be taken as the controlling value. With this as a guide, a minimum thickness of $2\frac{1}{2}$ in. is recommended with larger thicknesses used where required in accordance with the data shown in Figure 10.

The data presented in this paper emphasize that considerable analysis is required to properly design an overlay in accordance with the system shown in Figure 2. This example required 410 solutions for the computer program for stresses in a multilayered structure, 72 solutions of the fatigue computer program, and more than 1,000 hand solutions of various equations.

Although considerable effort has been indicated for the design phase for this project, it is important to note that the direct construction expenditures for the overlay will be approximately $1,000,000. Thus, an analysis of the type presented here is a small part of the total costs of the project and is well worth the effort to ensure the best available solution to the problem.

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# *Appendix*

**Core measurements taken to ascertain variations in pavement thickness are given in Table 12. Observed distress data are given in Table 13.**

**TABLE 12**

**TEST DATA ON CORES**

| Sample No.* | Density by Absolute Vol. lb/ft$^3$ | Density by Comparative Volume | Measured Thickness, in. | Splitting Tensile, psi | - Flexural, psi | Condition** |
|---|---|---|---|---|---|---|
| 1 - T | 146.3 | 140.0 | | 535 | 866 | G |
| 1A - B | 145.0 | 134.0 | | 567 | 918 | G |
| 1XX - T | 144.8 | 122.5 | | 289 | 468 | G |
| 1X - B | 145.0 | 117.5 | 7.750 | 327 | 529 | G |
| 2 - T | 142.5 | 136.2 | | 417 | 675 | C |
| 2A - B | 141.7 | 126.6 | 7.625 | 395 | 640 | C |
| 3 - T | 142.6 | 136.8 | | 423 | 685 | C |
| 3A - B | 140.3 | 128.1 | 8.125 | 429 | 695 | C |
| 4 - T | 140.9 | 132.7 | | 458 | 742 | C |
| 4A - B | 140.8 | 128.2 | 8.125 | 263 | 426 | C |
| 5 - T | 140.8 | 135.2 | | 443 | 717 | C |
| 5A - B | 138.8 | 125.9 | 7.750 | 326 | 528 | C |
| 6 - T | 140.6 | 134.2 | | 412 | 667 | G |
| 6A - B | 142.8 | 127.3 | 8.000 | 458 | 742 | G |
| 7 - T | 142.2 | 136.4 | | 576 | 933 | C |
| 7A - B | 137.3 | 120.0 | 8.125 | 214 | 346 | C |
| 8 - T | 140.3 | 131.1 | | 509 | 824 | G |
| 8A - B | 140.3 | 125.9 | 8.000 | 358 | 580 | G |
| 9 - T | 142.4 | 137.5 | | 445 | 721 | C |
| 9A - B | 141.6 | 126.9 | 7.750 | 356 | 576 | C |
| 10 - T | 142.3 | 136.1 | | 465 | 753 | G |
| 10A - B | 141.7 | 134.3 | 7.875 | 387 | 627 | G |
| 11 - T | 144.2 | 138.0 | | 451 | 730 | C |
| 11A - B | 141.8 | 118.3 | 8.250 | 265 | 429 | C |
| 12 - T | 142.5 | 136.3 | | 416 | 674 | G |
| 12A - B | 142.5 | 129.6 | 8.250 | 409 | 662 | G |
| 13 - T | 142.3 | 135.9 | | 409 | 662 | C |
| 13A - B | 143.6 | 130.4 | 8.000 | 564 | 913 | C |
| 14 - T | 143.0 | 137.6 | | 472 | 764 | G |
| 14 - B | 143.5 | 125.0 | 8.125 | 323 | 523 | G |
| 15 - T | 141.4 | 135.4 | | 490 | 794 | C |
| 15A - B | 140.8 | 128.0 | 8.000 | 496 | 803 | C |
| 16 - T | 141.1 | 134.8 | | 592 | 959 | G |
| 16A - B | 144.2 | 132.3 | 8.125 | 447 | 724 | G |
| 17 - T | 135.9 | 127.8 | | 373 | 604 | C |
| 17A' - B | 137.9 | 114.8 | 7.875 | 345 | 559 | C |
| 18 - T | 138.8 | 132.6 | | 396 | 641 | G |
| 18A - B | 141.7 | 121.5 | 8.500 | 354 | 573 | G |
| 19 - T | 138.9 | 132.4 | | 497 | 805 | C |
| 19A - B | 137.9 | 105.8 | 8.000 | 164 | 265 | C |
| 20 - T | 139.4 | 134.6 | | 420 | 680 | G |
| 20A - B | 140.0 | 122.2 | 7.750 | 309 | 500 | G |
| 21 - T | 144.6 | 137.0 | | 552 | 894 | C |

TABLE 12 (Continued)

| Sample No.* | Density by Absolute Vol. lb/ft$^3$ | Density by Comparative Volume | Measured Thickness, in. | Splitting Tensile, psi | - Flexural, psi | Condition** |
|---|---|---|---|---|---|---|
| 21A - B | 140.1 | 122.0 | 8.250 | 303 | 491 | C |
| 22 - T | 142.0 | 136.7 | | 472 | 764 | G |
| 22 - B | 140.6 | 116.0 | 8.125 | 286 | 463 | G |
| 23 - T | 144.7 | 139.2 | | 577 | 934 | C |
| 23A - B | 141.1 | 126.0 | 8.375 | 363 | 588 | C |
| 24 - T | 141.0 | 134.5 | | 510 | 826 | G |
| 24 - B | 140.5 | 128.1 | 8.125 | 384 | 622 | G |
| 25 - T | 143.1 | 136.6 | | 441 | 714 | C |
| 25 - B | 140.9 | 124.2 | 8.000 | 216 | 350 | C |
| 26 - T | 142.7 | 138.3 | | 470 | 771 | G |
| 26 - B | 145.3 | 127.7 | 7.875 | 607 | 983 | G |
| 27 - T | 145.5 | 141.2 | | 609 | 986 | C |
| 27 - B | 142.4 | 124.7 | 7.750 | 421 | 682 | C |
| 28 - T | 146.4 | 140.6 | | 536 | 863 | G |
| 28 - B | 145.6 | 117.6 | 7.750 | 416 | 674 | G |
| 29 - T | 142.5 | 134.1 | | 528 | 855 | C |
| 29 - B | 140.0 | 119.3 | 8.625 | 426 | 690 | C |
| 30 - T | 145.4 | 138.3 | | 599 | 970 | G |
| 30 - B | 141.8 | 123.5 | 7.750 | 392 | 635 | G |
| 31A - T | 143.6 | 138.5 | | 487 | 789 | C |
| 31A - B | 141.9 | 127.0 | 7.625 | 383 | 620 | C |
| 32 - T | 141.8 | 135.4 | | 554 | 897 | G |
| 32 - B | 140.9 | 130.0 | 8.125 | 413 | 669 | G |
| 33 - T | 143.7 | 137.9 | | 488 | 790 | C |
| 33 - B | 143.1 | 111.4 | 7.875 | 327 | 530 | C |
| 34 - T | 140.1 | 133.2 | | 521 | 844 | G |
| 34 - B | 141.1 | 119.3 | 7.625 | 327 | 530 | G |
| 35 - T | 147.4 | 137.2 | | 496 | 803 | C |
| 35 - B | 141.4 | 127.3 | 7.875 | 356 | 576 | C |
| 36 - T | 142.4 | 136.6 | | 474 | 767 | G |
| 36 - B | 144.4 | 127.0 | 8.500 | 602 | 975 | G |
| 37 - T | 140.1 | 135.8 | | 463 | 750 | C |
| 37 - B | 140.8 | 129.9 | 7.875 | 434 | 703 | C |
| 37A - T | 144.8 | 137.0 | | 581 | 941 | C |
| 37A - B | 142.3 | 124.4 | 8.000 | 377 | 610 | C |
| 38 - T | 143.6 | 136.8 | | 466 | 755 | G |
| 38 - B | 146.5 | 133.8 | 8.000 | 502 | 813 | G |

* T - Top half of the core
B - Bottom of the core

** G - Pavement in good condition
C - Pavement cracks showing signs of deterioration

TABLF 13

**DESIGN SECTIONS WITH SIGNIFICANTLY DIFFERENT PERCENTAGES OF STRUCTURAL DISTRESS IN CRCP OF WHEELPATH NO. 1 (95 PERCENT CONFIDENCE LEVEL)**

| Lane | Limits (Dynaflect Stations) | Average of Distress Linear Ft/Ft Wheel Path No. 1 | Standard Deviation |
|---|---|---|---|
| North Bound | 0-36 | 2.37 | 3.74 |
| | 36-57 | 6.38 | 0.78 |
| | 57-Overlay | 7.82 | 6.23 |
| | Overlay-104 | 8.34 | 9.06 |
| | 104-End | 1.20 | 1.13 |
| South Bound | 0-Overlay | 3.60 | 3.56 |
| | Overlay-41 | 15.65 | 5.57 |
| | 41-45 | 3.40 | 1.85 |
| | 45-47 | 23.10 | 6.79 |
| | 47-62 | 3.08 | 2.60 |
| | 62-86 | 11.72 | 7.70 |
| | 86-97 | 1.32 | 1.74 |
| | 97-99 | 13.65 | 3.61 |
| | 99-108 | 3.30 | 2.02 |
| | 108-End | 0.60 | 0.71 |