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Authors of the Papers in This Record

Chong, G. J., Ministry of Transportation and Communications, 1201 Wilson Avenue, Downsview, Ontario M3M 1J8, Canada
Darter, Michael I., Department of Civil Engineering, University of Illinois at Urbana-Champaign, Urbana, IL 61801
Del Val, John, Del Val, Inc., P. O. Box 230060, Portland, OR 97223
Dixon, William C., New York State Department of Transportation, 1220 Washington Avenue, State Campus, Albany, NY 12232
Elkins, Gary E., Austin Research Engineers, Inc., 2600 Dellana Lane, Austin, TX 78746
Fitzpatrick, Michael W., New York State Department of Transportation, 1220 Washington Avenue, State Campus, Albany, NY 12232
Florence, R. H., Jr., Florence and Hutcheson, Inc., P. O. Box 1443, Paducah, KY 42001; formerly with Purdue University
Fowler, David W., Center for Transportation Research, University of Texas at Austin, Austin, TX 78712
Jewer, F. W., Ministry of Transportation and Communications, 1201 Wilson Avenue, Downsview, Ontario M3M 1J8, Canada
Law, David A., New York State Department of Transportation, 1220 Washington Avenue, State Campus, Albany, NY 12232
McCullough, B. Frank, Center for Transportation Research, University of Texas at Austin, Austin, TX 78712
McGhee, K. H., Virginia Highway and Transportation Research Council, Box 3817, University Station, Charlottesville, VA 22903
Meyer, Alvin H., Center for Transportation Research, University of Texas at Austin, Austin, TX 78712
Phang, W. A., Ministry of Transportation and Communications, 1201 Wilson Avenue, Downsview, Ontario M3M 1J8, Canada
Virkler, Stanley J., Farm Builders, Inc., Remington, IN 47977; formerly with Purdue University
Yoder, Eldon J., School of Civil Engineering, Purdue University, West Lafayette, IN 47907

Deterioration of New York State Highway Structures

MICHAEL W. FITZPATRICK, DAVID A. LAW, AND WILLIAM C. DIXON

The analysis presented here quantifies both the deterioration rate of New York State highway structures and the cost of that deterioration. The data used are from two complete cycles of condition inspections. The condition rating for the entire structure was used to estimate an overall deterioration rate, and the inspector's determination of the quantity of nine types of repairs needed was coupled with unit work costs from maintenance records to estimate the cost of all needed repairs. These backlogged repairs totaled $323 million in 1980 and were increasing at the rate of $39 million per year. This means that maintenance work worth at least an additional $39 million per year must be performed on New York State structures to halt the decline of their condition. A model of structure deterioration was developed from the data and used to predict future costs and condition should the current level of maintenance remain unchanged. It was inferred from the data that the rate of deterioration currently being experienced is considerably higher than that which existed before 1960.

The New York State Department of Transportation (NYSDOT) Highway Maintenance Division has been developing a maintenance management system for a number of years. One part of this system is the Highway Maintenance Information Service (HMIS)--a data base that contains records of all highway maintenance activities, including person hours, equipment hours, and materials expended as well as the quantity of work accomplished for each maintenance task. The Structures Design and Construction Division has developed a detailed inventory file of both state and local structures. In addition to this file, there is a continuing detailed inspection program of all structures; most state structures are inspected by state employees and most local structures by consultants.

Now that these data are available, they can be used to analyze existing maintenance operations in order to determine more-efficient ways of maintaining structures. Thus, the director of highway maintenance requested the Engineering Research and Development Bureau to initiate a research project to analyze existing data and develop a standard methodology by which maintenance operations could be periodically optimized to meet changing conditions. A study proposal for this project, entitled Optimizing Maintenance Quantity Standards for Structures, has been prepared and was recently approved by the Federal Highway Administration (FHWA).

While FHWA was reviewing this study proposal, the maintenance director also requested that some of the summary inspection data be analyzed and that visual aids be prepared to assist in FY 1981-1982 budget preparation. (New York's fiscal year begins April 1.) This work was undertaken as part of the Engineering Research and Development Bureau's Technical Assistance Program, which is meant to address problems that can be handled less formally than can those in the regular research program. It was considered a warm-up for the researchers, who could become familiar with some of the data with which they would be working and at the same time provide the needed information. As it turned out, the amount of data analysis and the information obtained were well beyond original expectations.

DATA ANALYSIS

The data available were summaries of inventory and inspection files for structures maintained by state employees. The total number built each year and the number given each of seven possible condition ratings or general recommendations (referred to hereafter as "ratings," for brevity) were analyzed first. A rating is one of seven values that represents an inspector's overall evaluation of a structure's condition, as follows:

7: Good Condition--few, if any, repairs required;
6: Minor Repairs Required--minor cracks and spalls, bearing adjustments, etc.;
5: Repairs Required--primary structural members and substructure in relatively good condition;
4: Structural Repairs Required--considerable structural reconditioning required;
3: Major Structural Repairs Required--in need of extensive reconditioning (may be posted for less than design load);
2: Poor Condition--serious deterioration of the primary structural members and/or substructure (normally posted for less than original design load); and
1: Very Poor Condition--may be closed to traffic (should be posted for reduced load) and requires major repairs or complete replacement in the very near future.

First, these data were used to plot the age distribution of the structures for 1980. This is shown in Figure 1, in which 6335 structures are represented (7400 structures have been identified as maintained by the state but complete data were not available for all at this time). The data were summed into five-year intervals to reduce the scatter and enhance visual presentation. This distribution is decidedly bimodal. The two peaks are obviously related to historic and economic events that affected not only New York State but the nation as a whole. Although many older structures have been replaced, the remnants of the original trends are still quite apparent.

Growth of automobile and truck use in both the state and the nation led to the first structure-building period, which reached its peak during the Depression, when public works became a nationwide program. The number of new structures declined during World War II, when materials and labor were dedicated to the war effort, and remained low for the decade after the war. The larger peak of recent structures coincides with the next great national road-building program--the Interstate system. Within this perspective, it is reasonable to surmise that the age distribution of New York's structures is likely to be typical of that of other states.

Next, the FY 1979-1980 inspection data were summarized. For each five-year interval, the mean of all ratings for all structures built in that interval was plotted versus the mean age for that group (Figure 2). A rather obvious linear trend appears for structures 15-80 years old. For younger structures, fewer points were available to show the trend, but it could be linear. Thus, two linear regressions, weighted to account for the bimodal age distribution, were calculated for these mean data points. The equations for these regressions for the structures 15-80 years old are shown in Figures 2 and 3. The facts that the trends appear linear and that a definite break shows in the trends are very important, and a complete explanation of their significance follows.

It was originally thought that the cause of the double-slope trend in Figure 2 could have been treatment by inspectors of the rating 7 as a special value, that is, one not retained for long for a new structure. As will be shown here shortly, this is

Figure 1. Structures maintained by New York State (based on 1979-1980 data).

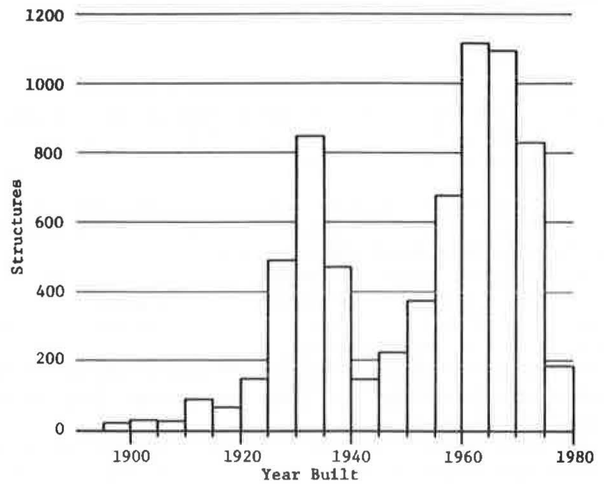


Figure 2. Rating versus age.

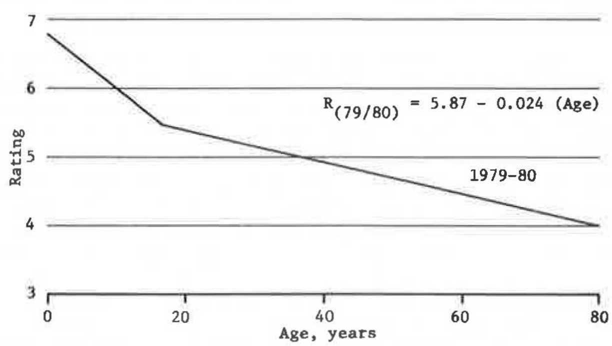
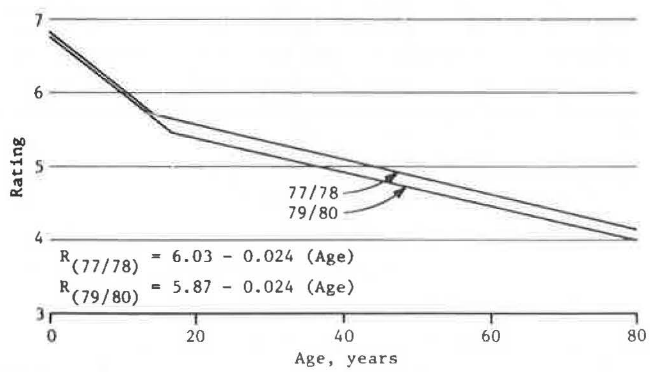


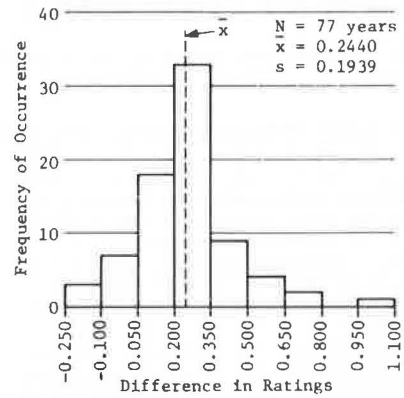
Figure 3. Rating versus age (two-year data).



not the case. Inspectors do not appear to have any bias against the rating 7. This graph could also be interpreted as showing rapid decay of newer structures followed by more-moderate deterioration after they have matured; this also proved incorrect when additional data were analyzed.

Figure 3 shows the regression through the inspection data from FY 1977-1978 with the regression from Figure 2. This fiscal year was chosen because the current inspection schedule requires two years to inspect all structures (only deficient structures rated 3 or less are inspected yearly), and earlier

Figure 4. Rating change between 1978 and 1980 inspections.



data were considered suspect due to inspector retraining during FY 1975-1976.

The fact that there are two distinct and parallel curves for structures more than 15 years old is extremely important. This indicates that a high rate of deterioration is now occurring--one that could not have been taking place for long. An average structure that was 30 years old in 1978 had a rating of 5.31, but two years later, in 1980, its rating was 5.10--an annual deterioration of 0.105. This represents the vertical separation between the curves plus two years down the curve. By taking paired differences between the two sets of data, the average annual deterioration for all structures was computed to be 0.122. This is the value used to predict the future condition of New York's structures, and the distribution of the difference in ratings is shown in Figure 4. (Note that negative values do occur. They are due to significant maintenance rehabilitation that occurred in some data pairs.)

What is shown in these data is the history of all maintenance, rehabilitation, and replacement of structures summed over the past 80 years. The regressions indicate that from 1900 to 1965 all structures on the average declined in condition at the rate of 0.023 rating points per year. Starting about 1965, all structures on the average began to decline in condition at a faster rate.

To translate this deterioration into cost terms, additional data were obtained. In each inspection, the inspector estimates the quantity of work required for each of nine maintenance tasks (known as Repairs Necessary). It was known from past experience that these nine tasks make up about three-fourths of the cost of maintenance. The amount of work required for all structures could be summed for each rating. By searching HMIS, the unit cost for each of these tasks was found. By multiplying the amount of each task required by its unit cost, summing, and then dividing by the number of structures, the average cost of Repairs Necessary for each rating was found to be as follows:

Rating	Avg Cost per Structure ($)
1	214 084
2	220 895
3	211 496
4	109 445
5	44 094
6	13 446
7	3 238

Average costs are plotted versus ratings in Figure 5; the costs have been increased by a factor of 4/3 to include all maintenance costs, not just the nine tasks. This plot is quite important. The cost of Repairs Necessary certainly does not increase linearly with declining condition. More than that, once a structure declines to a rating of 3, it incurs no further Repairs Necessary. This, in essence, implies that as a structure declines below a rating of 3 (at which it is already considered deficient) the amount of work needed to repair it does not increase. Apparently, the urgency of the work increases. This is accentuated by the fact that only about 3 percent of the structures rated 3 are posted for reduced load-carrying capacity, whereas 20 percent of those rated 2 and 100 percent of those rated 1 are so posted. The nonlinear increase of the cost of Repairs Necessary from 7 to 3 (which is nearly a pure exponential increase) argues very strongly in favor of early and preventive maintenance. Relatively modest expenditures to keep a structure at 5 or better will prevent much larger expenditures from being necessary at a future date.

For purposes of calculating total costs of Repairs Necessary, those for each rating can be multiplied by the respective number of structures and summed. Thus, current costs for 1980 were calculated and are given in Table 1. This shows that, for the present condition, the cost of backlogged needed repairs is $323 million.

Because a specific number of structures exist that on the average will incur only $217 500 each in needed repairs when deficient, Repairs Necessary costs that can occur have an upper bound. This bound, when all 6335 structures are rated at 3 or less, has the value of $1378 million. In these terms, the current $323 million cost of needed repairs represents about 23 percent of the maximum.

The current rate at which backlogged repairs are accumulating can be determined by considering each of the five-year intervals as a subset of the whole population that possesses meaningful mean ratings and distributions of ratings about that mean. (Since the smallest interval contains 30 structures--and most contain more than 200--this is a reasonable assumption.) By multiplying the number of structures at each rating within each interval by the respective cost of Repairs Necessary and summing, the total cost for that interval is obtained.

For example, for the interval 1930 through 1934, there are 856 structures with a mean condition rating of 4.80. There are 49 structures rated 7, 223 rated 6, 279 rated 5, 184 rated 4, 73 rated 3, 33 rated 2, and 15 rated 1. The total cost of Repairs Necessary for this interval is calculated to be $61.5 million, or $71 889 per structure. Performing these calculations for each interval yields the data presented in Table 2 and the plot of cost per structure versus mean rating in Figure 6. This plot is obviously linear in the range of 3.8-5.4 (which includes the current mean rating of 5.26 for all structures) and has a slope of $50 458 per structure per change in mean rating. By the chain rule of calculus, this slope can be multiplied by the slope of rating versus time (already found to be -0.122) to get the slope of cost per structure versus time. That slope is $6156 per structure per year. Multiplying by 6335 structures yields the current rate of accumulation of Repairs Necessary--$39 million per year. This is the current yearly cost of deterioration that exceeds what is being corrected by maintenance, rehabilitation, and replacement. It can be

Figure 5. Repairs Necessary costs.

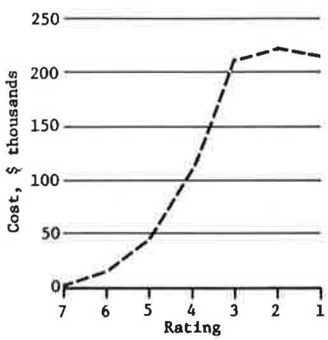


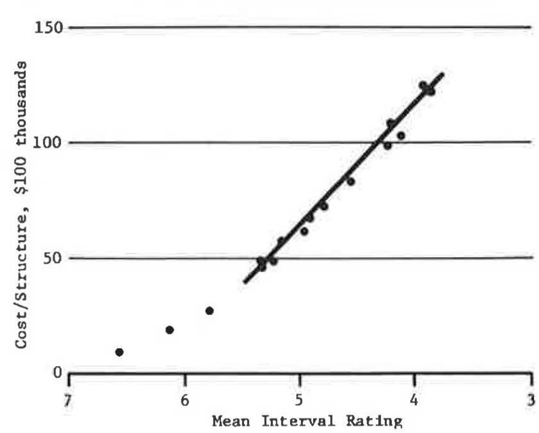
Table 1. Cost of Repairs Necessary in 1980.

Rating	No. of Structures	Avg Repairs Necessary per Structure ($)	Total Repairs Necessary ($)
1	60	217 490	13 049 000
2	170	217 490	36 973 000
3	296	211 496	62 603 000
4	931	109 445	101 893 000
5	1714	44 094	75 577 000
6	2192	13 446	29 474 000
7	972	3 238	3 147 000
Grand total			322 716 000

Table 2. Cost of Repairs Necessary for five-year intervals (1980 inspection data).

Interval	No. of Structures	Mean Rating	Repairs Necessary ($ 000 000s)	Avg Repairs Necessary per Structure ($)
1900-1904	34	3.91	4.1	120 489
1905-1909	30	3.87	3.6	121 142
1910-1914	90	4.13	9.3	103 090
1915-1919	71	4.25	7.0	98 088
1920-1924	156	4.21	16.5	105 700
1925-1929	494	4.55	41.5	83 951
1930-1934	856	4.80	61.5	71 889
1935-1939	477	5.01	28.4	60 680
1940-1944	155	4.92	10.3	66 334
1945-1949	200	5.22	10.0	49 805
1950-1954	307	5.18	17.5	56 982
1955-1959	593	5.35	27.1	45 673
1960-1964	884	5.34	41.7	47 195
1965-1969	1024	5.79	27.8	27 186
1970-1974	777	6.14	14.5	18 704
1975-1979	187	6.58	1.8	9 787

Figure 6. Cost per structure versus mean rating of intervals.

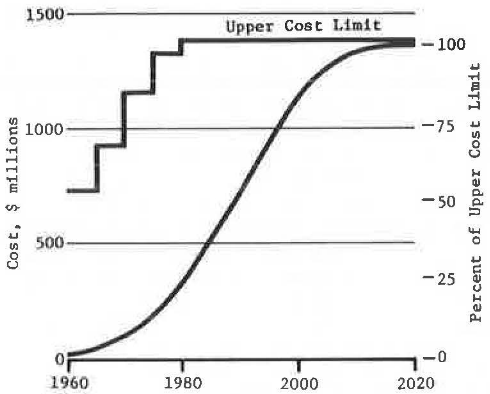


thought of as the amount of debt being incurred yearly, but just for this year. What will happen in the future is something else.

THEORETICAL MODEL

Although the calculations needed to produce these projections are not mathematically complex, the amount of work needed to calculate each point is quite time-consuming. In addition, the points themselves do not provide particularly clear insight into the process taking place. Thus, a theoretical model was sought.

Figure 7. Growth of total Repairs Necessary costs.



The calculated values of the cost of Repairs Necessary are plotted in Figure 7. The increase in cost of Repairs Necessary results from deterioration of structures. It has already been shown that, as average condition declines, the distribution of structures that have each possible rating shifts to include more deficient structures. There are currently 526 deficient structures and, as previously shown, these structures have already reached their maximum costs and will make no further contributions to the total cost. (The projection conservatively estimates 507 deficient structures.) By 1990, more than 35 percent of the structures will be at their maximum cost. Thus, as accumulated costs increase, the structures available to contribute additional costs diminish. This reduction will halt the exponential rise in Repairs Necessary and eventually cause the rate of increase to decrease, which produces the S-shaped curve in Figure 7. But which S-shaped curve would make a good model?

A probability-curve model was suggested by the shape and is supported by the following analysis of the data. The plot of the average cost of Repairs Necessary versus rating shown in Figure 5 is not a continuous curve because of the discrete nature of the scale. A plot of the slopes of this curve (the derivative) is also discontinuous and looks like a histogram (Figure 8A). This curve, however, is of

Figure 8. Sample plots.

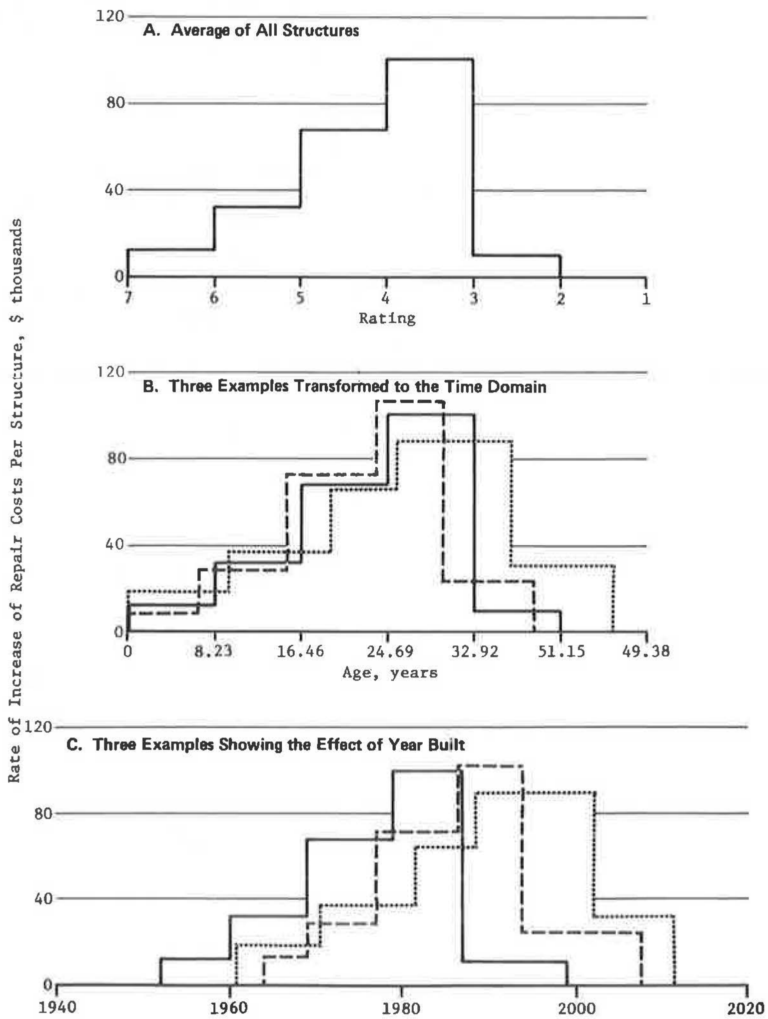


Figure 9. Growth of total Repairs Necessary costs.

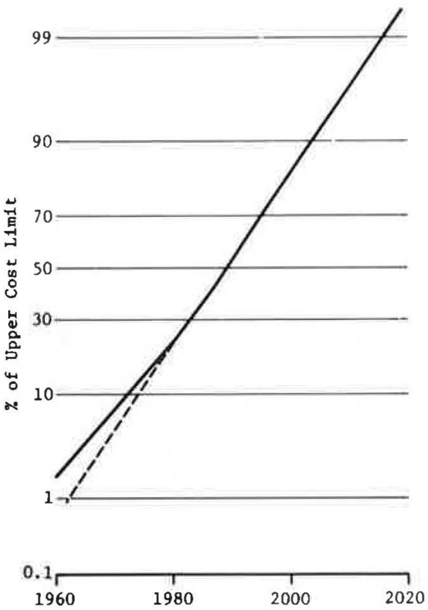
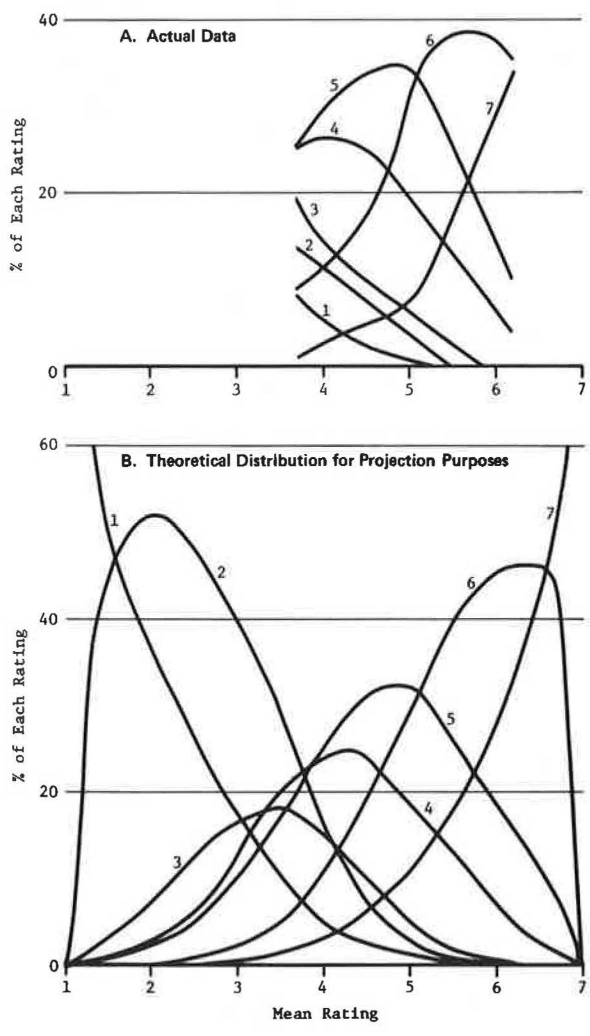


Figure 10. Distribution of ratings versus mean ratings.



no use in its present form because it is an average for all structures and not in the time domain. Three considerations are necessary before the proper curve can be derived:

1. Because this is an average plot, it must necessarily be made up from individual plots of each structure--some higher, some lower, and some with somewhat different shapes. Thus, there would be 6335 individual curves distributed in such a way that the plot in Figure 8A would be the mean.
2. Each of these 6335 plots must be transformed into the time domain. It has already been shown that the average annual deterioration rate is currently 0.122, so it would be possible to replace the rating interval with a time scale; each interval would be 8.23 years (Figure 8B). Since 8.2 years per change of rating is also an average value, it is possible to visualize that each of the 6335 individual curves (which already have different heights) has a different time for a rating-change interval. In fact, the interval need not even be constant throughout the structure's life. Now there are 6335 curves that have different heights and different lengths.
3. Each individual curve can be plotted on a calendar time scale that starts at the year when the structure was built. There are now 6335 curves that have different heights and lengths and start at 80 different times. Three examples are shown in Figure 8C.

It is reasonable to assume that summing these 6335 curves will yield a normal distribution that represents the rate of increase of the cost of Repairs Necessary. If it does, its integral will be a cumulative probability curve that represents the total cost of Repairs Necessary over time.

Plotting the calculated values of Repairs Necessary on a normal probability yields a straight line that has little error from 1980 to 2010 (Figure 9). This straight line confirms that the data conform closely to a normal distribution within most of their range. This finding is quite significant because it is now possible to use standard statistical tables for simple and accurate calculation of both the accumulated cost and the rate of increase of Repairs Necessary.

A similar type of model was developed for distribution of structural condition. It was considered that decay of a group of structures that had a rating of 7 to a rating of 6 would follow an approximately normal distribution. This type of decay is suggested by Figure 10A. In fact, Figure 10B was derived by just such a model. No matter what the distributional form, the percentage of structures at 6 and below will be 100 percent minus the percentage that remain at 7 and would be symmetrical about the 50 percent level. By the same reasoning, the percentage at 5 and below would be 100 percent minus the percentage at 6 and 7. The data are plotted on a probability scale in Figure 11. They appear as a group of straight lines that have good fit between 5 and 95 percent. The probability model obviously does not apply at very high or low percentages or, more correctly, at the extremes of the rating scale. It would be theoretically neater if they were parallel, but at least they do not intersect within the range of interest. If they did, it would be tantamount to saying that the percentage ≤ 6 would be less than the percentage ≤ 5, an obviously illogical statement.

Plotting the difference between the straight lines in Figure 11 yields the percentage of structures at each rating over time (Figure 12). This clearly shows that most of the 7's are now gone and the 6's are starting to go, too. By the year 2000, the 3's will be peaking, and the growth of 1's and 2's will be phenomenal (235 per year); only 27 percent will be greater than 3. A full 73 percent (4225 structures) will be rated deficient (not functioning as originally designed). By using the current values of posting structures for reduced load (100 percent of the 1's, 20 percent of the 2's, and 3 percent of the 3's), a plot of the number of posted structures can be derived (Figure 13), which also plots as a straight line on probability paper.

Figure 11. Cumulative grouping of structures that result from the projection.

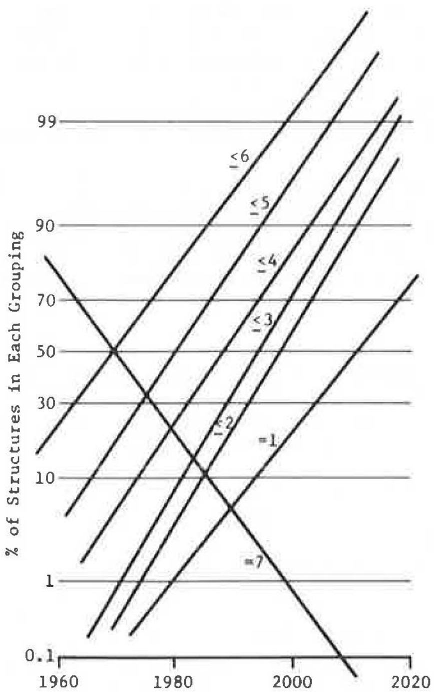
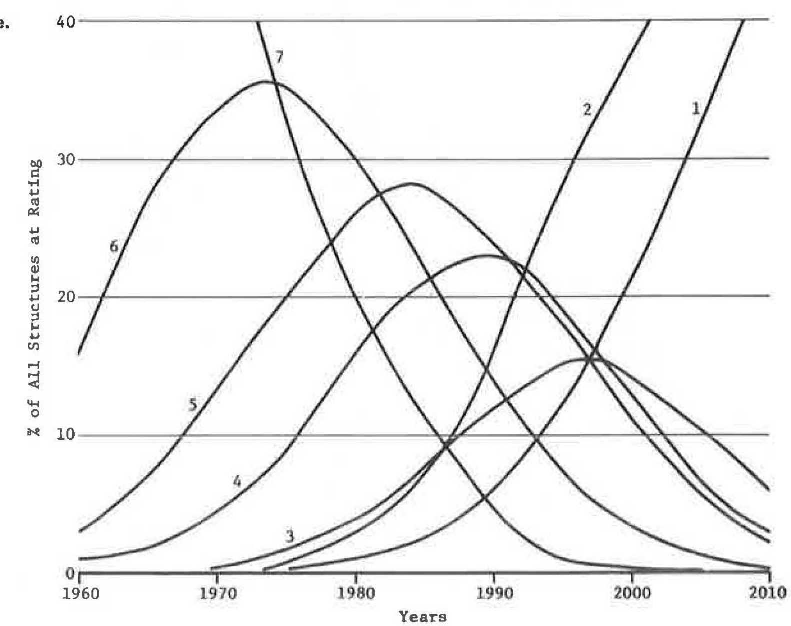


Figure 12. Projected distribution of ratings over time.



DISCUSSION OF RESULTS

Predicting Future Deterioration

The present annual rate of 0.122 was assumed to remain constant for the future. Although this rate of decay is disturbing enough, more information can be obtained by predicting the number of structures that will have each of the seven possible ratings at future dates.

This was done by first analyzing the distributions of ratings for each five-year interval. These data (Figure 10A) indicate the change in the distributions of ratings versus the average rating for an interval. To predict what would occur in the future when the condition would be much worse, it was necessary to extrapolate these data. The resulting distributions are shown in Figure 10B. It was then possible to determine the distribution of ratings for any interval average condition.

To predict the situation in the year 1990, the average rating for each five-year interval was reduced by 1.22, and the distribution of ratings for the new mean was found from Figure 10B. The number of structures for each rating was then calculated by multiplying this distribution by the number of structures in the interval. Then all the intervals were summed, which yielded the total number of structures at each rating in 1990. The results of these calculations are given in Table 3. In line with these projections, the amounts that will be required in 1990 for Repairs Necessary (in millions of dollars) were estimated by rating number to be as follows: 1, 82.4; 2, 233.6; 3, 172.3; 4, 145.7; 5, 68.0; 6, 12.0; and 7, 1.0. This sums to $715.0.

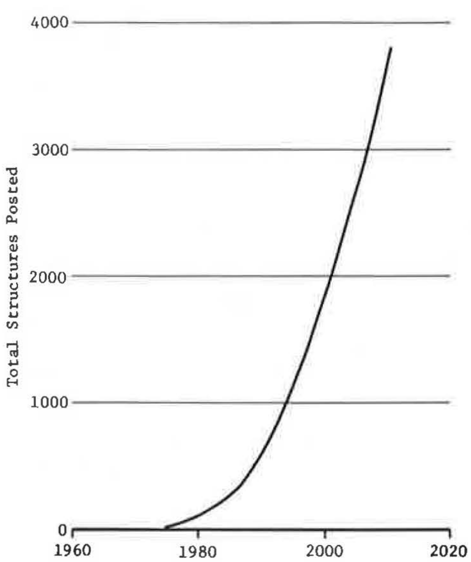
Performing these same calculations for other years yields a graphic picture of what is happening to New York's highway structures. These data are summarized below:

Year	Rating	Repairs Necessary ($000 000)	Annual Rate of Increase ($000 000)
1960	6.96	13.6	4.0
1970	6.25	97.2	14.0
1980	5.33	317.8	31.0

Year	Rating	Repairs Necessary ($000 000)	Annual Rate of Increase ($000 000)
1990	4.02	717.3	47.0
2000	2.64	1133.6	32.0
2010	1.67	1327.9	9.0
2020	1.36	1357.8	0.0

Figure 7 showed how the cost of backlogged repairs will increase without even considering inflation. In this decade, the amount will more than double if the same level of maintenance is continued. By the year 2000, it will have more than tripled. The increase of $39 million per year in backlogged repairs calculated for 1980 does not agree with this projection because the curve-fitting process used to extrapolate the data from Figure 10A to get Figure 10B caused considerable smoothing of the data. As seen above, the mean rating for 1980 was overestimated, and the total Repairs Necessary and rate of increase for 1980 were underestimated. All three are errors on the conservative side, so all projections will also be conservative. Total Repairs Necessary are accelerating with time, and the rate of change will peak before 1990. The projection indicates $47 million annually for that peak, but it could be well over $50 million annually.

Figure 13. Projection of structures posted.



As should be expected, as needed repairs are backlogged, the overall system condition declines. The mean rating will reach 3 in less than 20 years.

Significance for Maintenance Management

The calculations just performed are sufficient to explain what is happening to New York's highway structures and to indicate what in general should be done. Since it is only a macroscopic model, it cannot provide detailed answers such as which maintenance activities should have priority for funding. These answers will come only when the research project on quantity standards is successfully completed. Until then, the best judgments of maintenance managers will suffice. These calculations do, however, detail the rules of the game.

The current annual rate of increase of backlogged repair is $39 million and still increasing. The peak of this curve will be reached in 1989 when the annual rate of increase will reach a projected $47 million per year (probably higher). These figures represent the cost of additional repairs beyond those currently taken care of by maintenance forces and rehabilitation by contract.

This means that the deterioration of the overall condition of the state's structures can be stopped today by doing additional repairs equivalent to $39 million at 1979 maintenance unit prices. This additional amount of work will have to be done yearly over the long term. The overall average rating will then stabilize at the current value of 5.26. If action is delayed until 1989, the yearly effort required will be $47 million or more to stabilize the average rating at about 4.20. From then on, it would require progressively less annual expenditure to maintain a progressively worse average condition. Thus, the cost of delay is high.

These calculations give no indication of how the work should be divided between increased Highway Maintenance Division efforts and rehabilitation by contract. The data presented here show that state maintenance forces should be able to rehabilitate an average deficient structure for about $217 500 at 1979 prices. The Structures Design and Construction Division estimated that contract rehabilitation averaged $450 000 per structure at 1977 prices. If we consider inflation between 1977 and 1979, it is apparent that contract rehabilitation costs about three times as much. Thus, the cost of this needed work will range from $39 million annually if it is

Table 3. Projected 1990 distribution of structures.

Interval	Avg Rating	No. of Structures	No. of Structures for Each Rating: 1	2	3	4	5	6	7
1900-1904	2.74	34	7	15	5	3	3	1	0
1905-1909	2.87	30	6	13	5	3	3	0	0
1910-1914	2.99	90	15	37	14	12	9	2	1
1915-1919	3.12	71	11	27	12	10	9	2	0
1920-1924	3.25	156	21	54	26	26	22	5	2
1925-1929	3.37	494	58	155	86	90	78	21	6
1930-1934	3.50	856	86	240	154	171	150	43	12
1935-1939	3.62	477	42	120	82	100	93	31	9
1940-1944	3.75	155	12	35	26	34	33	12	3
1945-1949	3.88	200	13	39	32	46	46	19	5
1950-1954	4.00	307	15	52	46	74	77	34	9
1955-1959	4.13	593	26	87	82	142	157	77	22
1960-1964	4.25	884	33	108	110	212	248	133	40
1965-1969	4.58	1024	24	69	95	238	319	210	69
1970-1974	5.03	777	9	21	37	145	246	230	89
1975-1979	5.48	187	1	2	3	25	49	74	33
Total		6335	379	1074	815	1331	1542	894	300

all done by state forces to $117 million if it is all done by contract.

If the state fails to meet this challenge, it will soon become evident. On the average, there is one structure for every three miles of roadway. These structures are becoming deficient at a rapid rate. One-half will be deficient by 1994 and 74 percent by 2000. Posting for reduced load lags behind this, but by 2000 nearly one-third will be posted. This means that a posted structure (one unable to carry a large truck safely) will be encountered on the average of every 10 miles. The impact of this on the state's economy will be significant. These findings are summarized for several selected years as follows:

Year	No. Rated 3 or Less	No. Posted	Avg Miles Between Posted Structures
1980	507	109	174
1990	2280	606	31
2000	4688	1838	10
2010	6018	3738	5

Possible Causes of Accelerated Deterioration

The following possible causes of accelerated deterioration were investigated for compatibility with the trends found:

1. Environment: Air pollution is known to attack and damage stone, concrete, steel, etc. However, a serious air pollution problem has existed since well before 1965. In recent years, there has actually been some improvement. This is not compatible with the increasing rate of decay. Thus, acid rain included, the environment is not considered a likely cause.
2. Design and construction: It seemed possible that new designs that incorporate defects and/or poor quality control during construction could lead to increased deterioration. Since structures of all ages are experiencing this accelerated decay, this does not match the trends and was discounted as a probable cause.
3. Dilution of maintenance: An increasing number of structures, inflation, and budget restrictions combine to increase the maintenance workload and make it difficult at best to meet the challenge. The increase in the number of structures is documented in Figure 1, and the building program of the 1960s and early 1970s occurs at the right time to be a possible cause. Serious problems with inflation and budget restrictions are more recent but are at about the right time to take over where the increase in number of structures left off. Thus, dilution of maintenance is considered a possible cause.
4. Snow-and-ice control: In the early 1960s, New York embarked on a bare-pavement policy. This increased the use of chloride chemicals for the purpose of improving safety and traffic operations during and after snow-and-ice storms. It is well known that chlorides damage both portland cement concrete and steel, perhaps more than has been suspected. The timing is right, and it would affect structures of all ages. Thus, it should be considered a possible cause.

These last two possible causes may have combined and compounded the deterioration problem, but at present there is only inference to suggest it. The data analyzed in this paper are insufficient to determine cause. Conclusive data, in fact, may not exist.

CONCLUSIONS

1. New York's structures are experiencing accelerated deterioration. The current rate of decay appears to be five times the historical rate, and this may increase still further.
2. A rapid response to this problem is needed to prevent an unacceptable decline in the condition of structures, with its attendant economic consequences. An annual expenditure of about $39 million ($6150 per structure) by state maintenance forces would halt the decline and hold the structures in their current condition. If done by contract, this work would cost slightly more than $117 million annually ($18 450 per structure).
3. Preventive maintenance applied to structures in good condition appears to be a very cost-effective strategy.

ACKNOWLEDGMENT

The work described in this paper was conducted under the supervision of William C. Burnett, director of engineering research and development of the New York State Department of Transportation, in cooperation with the Federal Highway Administration, U.S. Department of Transportation. We wish to thank the many members of the State Department of Transportation whose efforts provided the data that have been analyzed. Readers interested in a more-extended discussion of this subject should request Special Report 70 from the Engineering Research and Development Bureau, New York State Department of Transportation, State Campus, Albany, New York 12232.

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Abridgment

Precast Repair of Continuously Reinforced Concrete Pavement

GARY E. ELKINS AND B. FRANK McCULLOUGH

An initial investigation into the applicability of repairing continuously reinforced concrete pavement (CRCP) by using precast repair slabs is described. To maintain continuity in the longitudinal reinforcement of CRCP, steel connections at the ends of the repair slab are the critical part of this repair technique. These connections may be made by welding, clamping, or use of commercial rebar connectors. Polymer concrete is a fast-setting material that has excellent properties as a cast-in-place repair material for use around these steel connections. Calculations of volume change indicate the possible development of excessive steel stresses at these connections on slabs longer than approximately 7 ft (2.13 m). This is attributed to the restraint of the concrete after its development of sufficient tensile strength that resists the normal cracking, which occurs early in the age of newly constructed CRCP. The use is postulated of a weakened plane situated in the center of the slab to cause the concrete to fracture before excessive steel stresses develop.

It is important that repair of punchouts, spalls, and other severe defects in continuously reinforced concrete pavement (CRCP) be performed in a minimal time, use materials readily available, be structurally sound and long lasting, and be economical. This is of particular importance on CRCP used primarily for high-volume roadways on which hazards due to both defect and repair of the defect are great. This is a report of an initial investigation into the use of precast slabs for rapid repair of CRCP (1).

PAVEMENT REPAIR BY USING PRECAST CONCRETE

The repair of concrete pavement by using precast concrete slabs is not a new idea. Previous repair work that used precast slabs was performed on jointed concrete pavements in Michigan in 1971 (2), in Florida in 1972 (3), and in California, Virginia, and New York in 1974 (4-6). These methods consisted basically of replacing the deteriorated portions of pavement with concrete slabs cast at or near the repair site. Prestressed repair slabs were used in New York (6).

PRECAST-CONCRETE METHOD APPLIED TO CRCP

The application of the precast-concrete concept to the repair of CRCP has a complicating factor not present in its use for the repair of jointed concrete pavement. These complications arise from the necessity of maintaining steel continuity in CRCP. In terms of volume-change stress, restraint of the steel at the end of a reinforced concrete slab induces significant stress increases over the unrestrained condition. These stresses may become destructively excessive and must be accounted for in design. In short, the precast-concrete repair methodology as applied to CRCP (Figure 1) consists of replacing the deteriorated pavement with a precast slab, anchoring the steel at the ends of the repair, and then filling the space around the steel connections with a fast-setting cast-in-place material.

ALTERNATIVE STRATEGIES

Implications of alternative approaches to the design and strategies for the (a) reinforcement, (b) steel connections, (c) cast-in-place concrete, (d) weakened plane, and (e) installation elements of precast-concrete repair slabs are discussed.

Reinforcement

The simplest reinforcement design for the repair slab is to reproduce the reinforcement design in the pavement to be repaired. This design may then be altered to conform to the requirements for precast repair slabs. In general, deformed-steel reinforcement should be used because of its superior bonding properties. The grade of steel should be selected to facilitate possible bending or welding.

Fibers may be added to portland cement concrete (PCC) to increase its strength and toughness. These short fibers may be steel, glass, synthetic polymers, or cotton. The amount of fiber generally required is from 1 to 2 percent. This amount of reinforcement is greater than the amount of steel bars in CRCP and may not be economical for this application. In addition, special provisions are required for mixing the concrete when this type of reinforcement is used.

Small-diameter steel wire may be spiraled around elements subjected to stress concentrations to help prevent cracking and to maintain small crack widths if cracking does occur.

Steel Connections

To maintain continuity in the longitudinal reinforcement of CRCP, a steel connection mechanism for precast repair slabs is required. These connections may be performed by a positive, a passive, or a combination of positive and passive connections. A positive connection is defined here as a connection in which the principal connection is achieved through direct bonding of rebars. A passive connection is achieved by anchoring the bars without direct contact.

Positive Connection

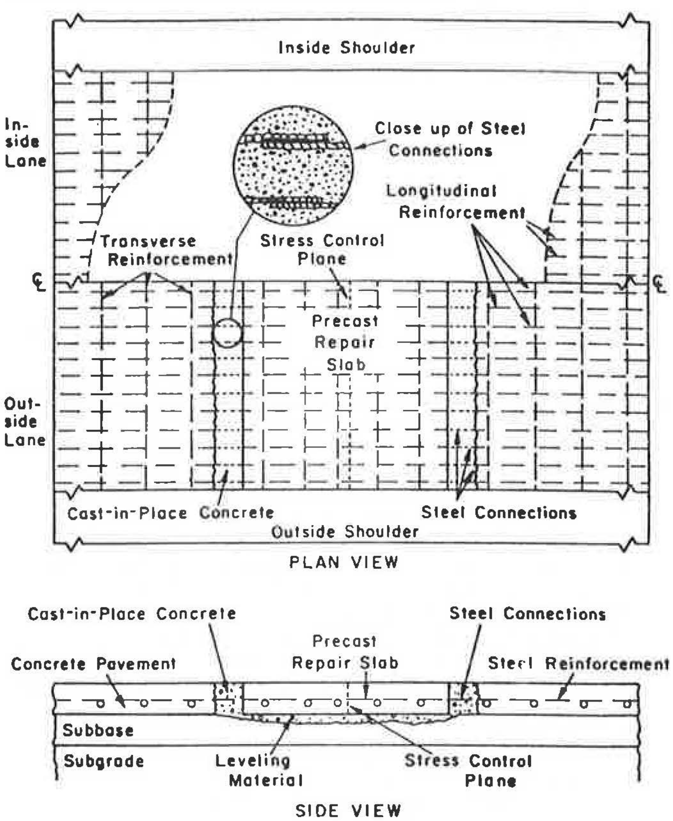
Alternatives for positive steel connections include the bonding of bars by welding, commercial splicer, or cable clamps. To adjust to variations in the alignment of reinforcement, these types of connections may require a positioning mechanism.

A weld connection may be made for each rebar in CRCP, or groups of bars may be connected.

Several types of commercially available rebar connectors have been developed for structural applications. Several companies market connectors that use a connection sleeve that is filled either with molten metal or with a fast-setting mortar, is heated and compressed around the bars, or is screwed into position through threads on the ends of the bars. For the most part, the connected rebars must be aligned end to end. In general, these connectors develop the ultimate strength of the reinforcing steel over a short length.

Another connection mechanism commercially available throughout the United States is the cable clamp. These clamps consist of a U-bolt and a curved seat. They are designed for connecting cable for applications such as telephone-pole stays. A series of simple tension tests on no. 5 deformed-

Figure 1. Precast repair methodology applied to CRCP.



steel rebars connected by means of these clamps indicate that the clamps can develop tensile forces in excess of the steel's yield point (1).

Passive Connection

The purpose of steel connections is to provide continuity in the reinforcing steel. This continuity acts to anchor the reinforcing steel at the cracks in CRCP, which restrains the concrete slabs in between. Thus, the important physical phenomenon of continuity is anchorage of the reinforcing steel relative to the adjacent concrete slabs. At discontinuities in the normal continuity of CRCP, it may be possible to anchor the reinforcing steel without direct connection of the bars to each other.

The development length of reinforcing steel is the important consideration for use of a passive connection. The development lengths of typical rebars used in CRCP simply embedded in normal PCC range from 12 to 18 in (305-457 mm). Development lengths can be reduced through the use of spherical bearing surfaces at the ends of the bars (7), spiral reinforcement around the bars, or high-strength concrete.

A high-strength fast-setting concrete that possesses excellent qualities for use in pavement repair is polymer concrete. No data on the development length or bond characteristics of this particular material could be located. Tests conducted at the Brookhaven National Laboratory showed that polymer-impregnated specimens developed between 30 and 55 percent greater ultimate strengths than the unimpregnated specimens (8). Three of the 13 impregnated specimens failed through tensile failure in the steel.

The above results are significant because the bond properties of polymer concrete are thought to be superior to those of polymer-impregnated concrete. These results also suggest that the development length for steel bars commonly used in CRCP embedded in polymer concrete is much less than 12 in (305 mm).

The principal reason for using passive steel connections for precast repairs is to reduce installation time. A time saving is realized by not providing a connection for each rebar. In addition, positioning mechanisms and extraneous hardware are not required.

Combination of Positive and Passive Connections

Positive and passive connections can be combined to provide steel anchorage. The essence of the combination connection is simply to extend the rebars past the end of the precast slab. Where the rebars in CRCP and the precast slab align, positive connections may be made. Bars not aligned could be provided with bearing surfaces at their ends. A combination connection, not strictly positive or passive, could consist of a transverse rebar positioned across the longitudinal steel in the connection zone and tied by means of wire to each rebar that it crosses.

Cast-in-Place Concrete

The void left at the steel connections between CRCP and precast repair slab must be filled with a cast-in-place material. This material may develop sufficient strength over a short period of time to obtain acceptable repair times.

One of the best known accelerators of PCC is calcium chloride. However, corrosion of reinforcing steel and increased shrinkage have been attributed to use of calcium chloride. Many state highway departments have prohibited its use for this reason.

Polymer concrete is a material that possesses qualities of a good cast-in-place repair material. Polymer concrete is a mixture of polymerized monomers and aggregate. Prior to polymerization, the monomers have a characteristically low viscosity that allows them to penetrate easily into the voids in the aggregate. After polymerization, which can be regulated to occur in less than 15 min, the polymer concrete generally exhibits greater strength than ordinary PCC, excellent freeze-thaw resistance, exceptionally strong bond to exposed concrete and steel, and low water absorption. Research and development work on this material performed by the U.S. Bureau of Reclamation (8,9) and the University of Texas at Austin (10,11) has demonstrated the applicability of polymer concrete to the repair of reinforced concrete in bridge decks, highway pavement, beams, and airfield pavement through laboratory and field experiments.

Other types of fast-setting materials that may be applicable as cast-in-place concrete include magnesia phosphate, calcium sulfate, epoxy resin, polyester resin, and high-alumina cement. Results of field tests of these products are limited, and long-term observations are nonexistent. Results of short-term trials have been mixed; thus, evaluation of materials proposed for use should be performed. Additional information on these materials may be found from various references in the literature (12-14).

Weakened Plane

The volume-change analysis indicated that precast repair slabs longer than about 6 ft (1.83 m) may have potential problems because of excessive steel stress at the ends of the slab. These stresses can be controlled by a weakened plane incorporated into the center of the slab. A weakened plane, which is simply a reduction in the effective resisting area of concrete, acts as a safety valve by causing the concrete to fracture at low levels of steel stress. This causes two shorter-length slabs to form that have lower stress levels.

A weakened plane may be formed by casting a bond breaker into the slab or by cutting a groove in the surface. By using a bond breaker, greater reduction in concrete area is possible, since saw cuts may only be made to the depth of the reinforcing steel. Another approach is the use of a bond breaker around the reinforcing steel. The major advantage of a bond breaker is that a defect is not induced onto the surface of the slab until it is required. It is possible that stresses will not become great enough to fracture the concrete and that stress control will not be necessary. In this situation, a grooved cut in the surface becomes a pavement defect and a source of potential problems. The bond breaker, however, is encased within the concrete slab and should not affect the slab's performance.

The bond breakers should be positioned so that they will not cause the slab to crack during lifting. The bond breaker should be oriented away from the side of the slab that is in tension. Alternatively, the lifting mechanism may be designed to induce very little or no tensile stress across the weakened plane.

Installation

The two important installation steps discussed here are lifting and leveling of the precast repair slab.

Lifting

A simple lifting scheme that consists of four lift points near the corners of the slab may be used. Lift points may be made by casting threaded inserts into the concrete. Swivel lifting plates may then be bolted into these inserts, and chains or cables may be attached to the swivel plates and the lifting device.

Another lifting scheme is a combination of threaded inserts and steel I-beams. The steel I-beams can be attached to the slab by means of bolts through the threaded inserts. The I-beams act to stiffen the slab during lifting.

Leveling

The most straightforward technique for leveling the patch with the surrounding pavement is to strike off a flat leveling course to the proper elevation. A screed-track configuration similar to the one used in Michigan (2) and illustrated in an earlier report (1) could be used. For sloped-crown sections, the track is positioned transversely across the pavement while the wood striker is manipulated longitudinally. For parabolic crowns, the track should be positioned longitudinally so that the precast slab aligns most closely in the wheel paths.

Another leveling technique is to mud jack or to force grout beneath a positioned slab. The precast slab can be positioned either by aligning its surface with the pavement and forcing material beneath it or by placing it on the surface of the subbase and raising it by forcing material beneath it. Holes in the slab could be combined with the lifting inserts for this purpose.

The third leveling technique is a combination of the techniques previously discussed. The leveling layer is placed and struck off to the proper elevation. The slab is placed on top of this layer. Fine adjustments can then be made by forcing more material through holes positioned above the slab.

CONCLUSIONS

The primary conclusion drawn from this study is that repair of CRCP by using precast slabs appears structurally feasible. Economic feasibility needs to be evaluated through field testing.

The detailed analytical work performed during this study resulted in many conclusions concerning (a) bending stiffness of the steel anchorage zone, (b) length of precast repair slabs, (c) critical void locations beneath repair, (d) cracking potential due to bond breakers, (e) effect of rebar size, (f) effect of area of reinforcement, (g) effect of concrete strength, (h) lifting behavior, and (i) cable-clamp steel connections. However, due to length limitations here, the reader is referred to the report by Elkins, McCullough, and Hudson (1) for detailed information.

REFERENCES

1. G.E. Elkins, B.F. McCullough, and W.R. Hudson. Precast Repair of Continuously Reinforced Concrete. Center for Transportation Research, Univ. of Texas at Austin, Res. Rept. 177-15, 1979.
2. J.E. Simonsen. Development of Procedures for Replacing Joints in Concrete Pavement. Michigan State Highway Commission, Lansing, Res. Rept. R-968, Aug. 1975.
3. R.F. Grimsley and B.G. Morris. An Approach to Concrete Pavement Replacement That Minimizes Disruption of Traffic. In Better Use of Existing Transportation Systems, TRB, Special Rept. 153, 1975, pp. 107-110.
4. Prefab Pavement Sections for PCC Repairs Slice Time and Cost for Caltrans. Better Roads, Sept. 1974.
5. M.F. Creech. Partial-Depth Precast Concrete Patching. TRB, Transportation Research Record 554, 1975, pp. 62-72.
6. J.W. Overacker. Thruway Repairs Concrete Slabs Overnight. Public Works, Vol. 105, No. 3, March 1974.
7. D.W. Lee and J.E. Breen. Factors Affecting Anchor Bolt Development. Center for Highway Research, Univ. of Texas at Austin, Res. Rept. 88-1F, Aug. 1966.
8. J.T. Dikeou and others. Concrete Polymer Materials: Fourth Topical Report. Brookhaven National Laboratory and U.S. Bureau of Reclamation, BNL 50328 and USBR REC-ERC-72-10, Jan. 1972.
9. J.T. Dikeou and others. Concrete Polymer Materials: Fifth Topical Report. Brookhaven National Laboratory and U.S. Bureau of Reclamation, BNL 50390 and USBR REC-ERC-73-12, Dec. 1973.
10. D. Fowler, M.M. Jaber, and D.R. Paul. Repair of Concrete with Polymers. Center for Highway Research, Univ. of Texas at Austin, Res. Rept. 114-3, Feb. 1975.
11. M.T. McNerney. An Investigation of the Use of Polymer Concrete for Rapid Repair of Airfield Pavements. Univ. of Texas at Austin, M.S. thesis, Dec. 1977.
12. Rapid-Setting Materials for Patching of Concrete. Paving and Structural Materials Group, Federal Highway Administration, U.S. Department of Transportation, April 1974.
13. R.G. Pike and W.M. Baker. Concrete Patching Materials. Paving and Structural Materials Group, Federal Highway Administration, U.S. De-

partment of Transportation, April 1974.
14. J.E. Ross. Rapid Setting Concrete Patching Study. Louisiana Department of Highways, Baton Rouge, Res. Rept. 84, March 1975.

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Patching of Continuously Reinforced Concrete Pavements

MICHAEL I. DARTER

This paper presents recommendations for the permanent repair of localized distress in continuously reinforced concrete pavements in Illinois. Recommendations for cost-effective patching are provided for selection of patch boundaries, sawing of the concrete, removal of concrete, replacing and splicing the reinforcing steel, preparing the patch area, placement of concrete, and curing of the patch until the area is reopened to traffic. These procedures have been validated through extensive field testing. The procedures reduce costs and lane-closure time by (a) adapting the patch size and type to fit the distress, (b) reducing reinforcement embedment length into the patch, (c) using mechanized equipment for construction, and (d) using concrete additives and curing techniques to facilitate early reopening to traffic.

The increasing amount of patching of continuously reinforced concrete pavement (CRCP) in Illinois has led to the need for procedures that are more cost-effective. Existing procedures evolved from the expensive repairs of CRCP sometimes required by the contractor to remedy errors made during new construction. CRCP is perhaps the most difficult and costly type of pavement to patch because of the unique design characteristics (e.g., continuous steel and closely spaced cracks). For these reasons, a cooperative research project was initiated in 1976 between the Illinois Department of Transportation and the University of Illinois. One major objective of the project was to develop improved patching procedures for the permanent repair of localized distress, which is briefly summarized here (1-5). The complete patching procedures are included in the report by Simonsen (6).

TYPES OF DISTRESS THAT REQUIRE PATCHING

The various types of CRCP distress that may require permanent patching are edge punchout, wide crack (ruptured steel), longitudinal joint fault, localized breakup, construction-joint failure, blowup, D-crack, and ramp-joint forced crack (steel rupture). Most of these distress types are unique to CRCP, so traditional methods used for identifying and diagnosing distress in plain or reinforced jointed concrete pavements may not be applied to CRCP. It is important that the various distress types be properly identified and diagnosed. To make a good diagnosis of the distress, the mechanisms of development must be understood. Maintenance personnel should know whether the distress is primarily a result of traffic loads, environmental conditions, or a construction defect. It is especially important that they be aware of the extent of the distress and how it has affected the slab, the reinforcing steel, the subbase, and the subgrade.

SELECTION OF PATCH BOUNDARIES

Permanent concrete patches have three distinct sections, simply referred to as the center section and the end sections (Figure 1). The correct determination of these section boundaries will increase the life of the finished patch and minimize overall annual patching costs.

For example, in most cases the subbase material beneath an edge punchout has disintegrated for some distance on either side of the distress. Signs of edge pumping or longitudinal joint faulting (settlement) will prove to be a good guide in determining how far from the edge of the visible distress the subbase has been damaged (typically 2-3 ft). Consequently, the overall length of the patch must reflect this deteriorated subbase material. Careful attention must be given to these warning signs around the distresed area and appropriate steps must be taken to ensure that the deteriorated areas are contained within the patch boundaries.

Broken reinforcement can be verified by running a thin ruler down through the crack, or any crack that has faulted 0.10 in or more can be assumed to contain broken or corroded steel or both. The patch boundaries cannot be moved too close to the distressed portion because then the possibility exists that all the deteriorated subbase may not be included, and future failure of the adjacent slab or patch or both will occur.

In order to minimize patch size and cost and at the same time provide adequate lap length and allow for cleanout of the center section, the following minimum values for overall patch length should be observed: (a) for a patch that contains tied steel, 4.5-ft minimum length; and (b) for a patch that contains welded steel, 3-ft minimum length.

A patch that contains tied steel can be placed for any distress type:

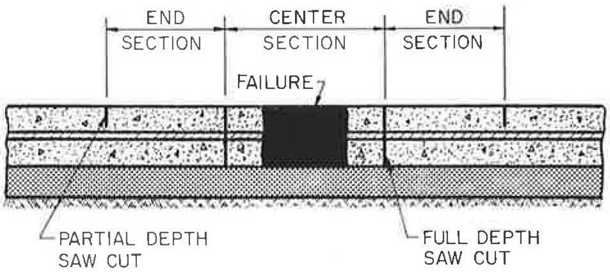
1. The distressed portions of the continuously reinforced concrete (CRC) slab and base should be incorporated within the center section of the patch, and
2. The end sections and the center section of the patch should be a minimum of 18 in long. Field and laboratory testing has shown these end and center section lengths to be adequate for typical reinforcement used in Illinois (no. 5 bars and welded deformed wire fabric).

A patch that contains welded steel can be placed whenever the proper equipment is available. This type reduces the length of the end sections and consequently saves a considerable amount of breakout time:

1. The distressed portion of the CRC slab and base should be incorporated within the center section of the patch, and
2. The end sections of the patch should be a minimum of 8 in long.

Because of the potential failure of the CRC slab between the edge of a patch and the nearest transverse crack in CRCP, the outer patch boundaries should be located at least 18 in from the nearest

Figure 1. Identification of center and end sections.



transverse crack. However, sometimes the spacing between cracks is very small, and in these cases it might be necessary to place the boundary closer to the nearest transverse crack than 18 in but not closer than 6 in. If this is necessary, one must make sure that the cracks are tightly closed and not faulted (to determine faulting one simply runs a hand over the crack in the direction opposite to that of the traffic). If a crack that is not tight or is faulted is within 18 in, the patch should be extended to include this crack. Also, the outer patch boundaries should not cross a transverse crack, since this may lead to spalling in the slab at the intersection.

In most instances the patch will be equal to a full lane's width, but this can be reduced under some circumstances:

1. Distress types such as wide cracks, large edge punchouts, blowups, and other distresses that occur over more than one-half of the lane should be patched over a full lane's width (12 ft); and
2. In some cases, such as a small edge punchout or spalling in the outer wheel path, the width of the patch need not be a full 12 ft. In these cases, a longitudinal boundary must be established. A minimum patch width of 6 ft should be used, however, to assure that the longitudinal boundary is placed between the longitudinal reinforcement bars and that the joint is not in the center of a wheel path.

Whenever a failure occurs across all tied traffic lanes, a special patching sequence is required to avoid rapid deterioration of the first patch placed. The typical situation is that in which a steel rupture or blowup has occurred across two lanes of four-lane divided Interstate highway. There is typically a large amount of movement across this crack during a given 24-h period. This movement has often badly cracked the first lane patched (normally the truck lane), and heavy truckloads during the next few weeks cause these cracks to spall and deteriorate until the patch breaks up. The patch placed in the other lane normally does not crack badly because slab movement is restrained.

The following procedure has been found to increase the likelihood of obtaining two good patches: (a) patching the passing lane (or the lane that has the lightest truck traffic) first and (b) patching the heavier truck lane(s) last. Cracks formed in the passing-lane patch from the daily temperature changes will not be likely to break down because of reduced truck traffic.

SAWING OF PATCH

Sawing of all outer boundaries of the patch is highly recommended. Experience has shown that the outer boundaries of a patch will undergo spalling if they are created by jackhammers or other breakout equipment or if they follow an existing crack. Consequently, the proper sawing equipment should be used.

The outer boundary of each end section should be a partial-depth saw cut 1.5-2 in deep that does not cut the reinforcing steel (Figure 2, step 1).

The boundary between the center section and each end section should be a full-depth saw cut through the reinforcement, unless the alternative method presented next is used (Figure 2, step 1).

The following alternative to the full-depth saw cut generally results in a saving of time and cost. The partial-depth saw cuts are placed as usual (Figure 3, step 1). Jackhammers are used to break up the concrete down to the steel, at which point it is cut by using a handsaw or a torch; this eliminates the full-depth saw cut at the boundary between the center and each end section (Figure 3, step 2).

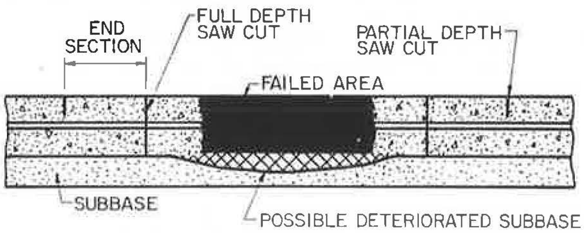
If pins or chains are to be used to lift out the center section of the patch, the saw cuts must be extended through the CRC slab. If full-depth sawing is not possible, an area approximately 5 in wide can be broken up by using a jackhammer along all edges (note that the saw cut must be deep enough to cut the steel). This will enable the slab to be lifted out without experiencing severe binding. This may be needed in hot weather even if full-depth saw cutting is available.

REMOVAL OF CONCRETE

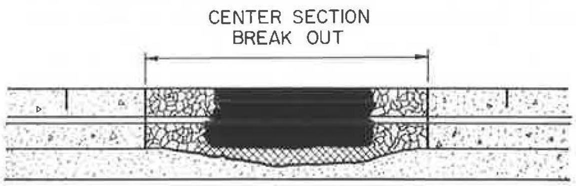
Removal of the concrete from the center section can be accomplished by several methods, depending on the equipment available (Figure 2, step 2; Figure 3, step 2; and Figure 4, step 2):

1. The most common procedure is to break up the concrete by using jackhammers and to shovel it out by using hand tools. The advantages of this method are that a minimum of equipment is needed and, if the work is done carefully, damage to the subbase and the adjacent slab is avoided. Unfortunately, this is the most time-consuming, manpower-intensive method and is thus relatively expensive.
2. The removal time can be shortened by using a pavement breaker and a backhoe. Breaking the concrete by means of a ball breaker should never be permitted, since the large shock waves will damage the adjacent concrete. The problem with this breakout method is that damage to the subbase material usually occurs.
3. A third method that has been used successfully is to lift the section out by using a front-end loader or a bulldozer. The usual procedure is to make a full-depth saw cut or to break up the concrete by using jackhammers on all sides of the center section. The front-end loader or bulldozer then lifts up one end of the slab. Chains are connected to the exposed steel at the other end of the slab and then secured to the bucket. The slab is lifted out and placed in a truck or, if it is very large, on a flatbed. This method can be accomplished in a very short time and it does not damage the subbase or the adjacent CRCP. One of the problems with this method is that the size of slab that can be lifted depends on the equipment available. Also, this method does not work very well in badly D-cracked material or where temporary bituminous patches are being replaced, and disposal of the slabs can be a problem.
4. A fourth method involves lifting the slab out by using pins or other mechanisms (Figure 4). This procedure requires two or more drilled holes, two or more lift-out pins, and some heavy equipment capable of lifting large loads, such as a large end loader. First a strip approximately 5 in wide of the CRC

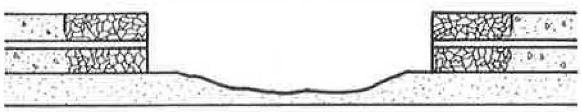
Figure 2. Standard breakout method: partial-depth and full-depth saw cuts (top, step 1); center-section breakout (middle, step 2); and end-section breakout (bottom, step 3).



Step 1: Make one partial depth saw cut at the ends of the patch and one full depth saw cut at the proper end section lengths from the edges (18 inches for tied steel patches 8 inches for welded steel patches).

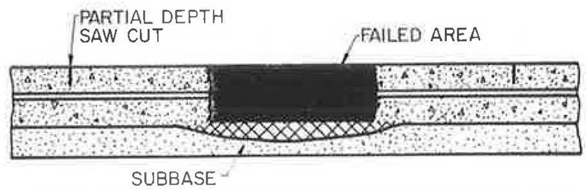


Step 2: Breakout concrete in the center section using jackhammers and remove debris by mechanical methods. Carefully remove the deteriorated subbase, if any exists, making sure not to damage the remaining subbase. The deteriorated area will be filled with portland cement concrete.

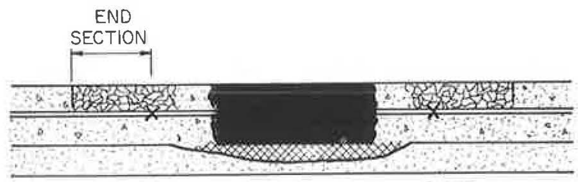


Step 3: **Break up and remove the remaining concrete in the end sections using hand methods being careful not to nick or bend the reinforcement and not to spall the existing CRC slab beneath the reinforcement. The rebar shall not be bent up for the removal of the remaining concrete.**

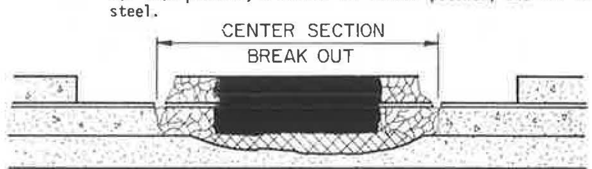
Figure 3. Alternative breakout method: partial-depth saw cut (top, step 1); end-section breakout (middle, step 2); center-section breakout (middle, step 3); removal of remaining concrete (bottom, step 4).



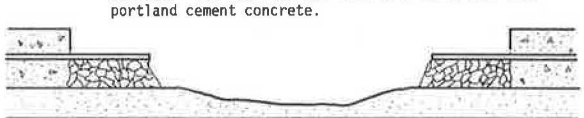
Step 1: Make one partial depth saw cut at the ends of the patch (do not cut steel reinforcement).



Step 2: Breakout concrete in end sections using jackhammers down to the steel at the proper end section length (18 inches for tied patches, 8 inches for welded patches) and cut the steel.



Step 3: Breakup center section using mechanical methods, remove debris in center using mechanical methods if the length allows. Carefully remove the deteriorated subbase, if any exists, making sure not to damage the remaining good subbase. The deteriorated area will be filled with portland cement concrete.



Step 4: Breakup and remove remaining concrete in the end sections using hand methods being careful not to spall existing CRC slab beneath reinforcement and not to nick or bend the reinforcement. The rebar shall not be bent up for removal of the remaining concrete.

center section across the lane must be broken up and removed down to the reinforcement. The steel must then be cut and taken away. The remainder of the strip down to the subbase must then be broken up and carefully removed by hand methods, care being taken not to damage the existing subbase. This may also be accomplished by making two full-depth saw cuts and then breaking out the strip of concrete by using hand methods. Simonsen (5) describes a reusable pin that has been used effectively by several states. However, some contractors have developed equipment that can lift out the slabs more rapidly. The lift-out of a given center section can be accomplished in less than a minute and normally leaves the base material and adjacent CRC slab undisturbed.

Removal of the concrete from the end sections is difficult and must be accomplished carefully (Figure 2, step 3, and Figure 3, step 4):

1. Concrete in the two end sections must be carefully removed so as not to damage the reinforcement in the lap area and to avoid spalling at the bottom of the joint (beneath the reinforcement). This task can be accomplished only by using jackhammers, prying bars, picks, shovels, or other hand tools.
2. Breaking around the reinforcing steel without nicking, bending, or in any way damaging it is difficult. However, the reinforcement must not be bent up during removal of the concrete since the bars cannot be properly straightened out afterwards. Bent reinforcement in the patch area will eventually result in spalling of the patch because of the large eccentric stresses carried by the reinforcement.
3. The use of a drop hammer or hydrahammer should not be allowed in the end sections because this equipment will typically damage the reinforcement and/or cause serious undercutting beneath the partial-depth sawed joint. It may be necessary to limit the size of the jackhammer operating at the joint to minimize undercutting beneath the reinforcement.
4. Any bent reinforcement should be carefully straightened after the breakout of the concrete. Any bends left in the reinforcement may eventually cause spalling in the completed patch.

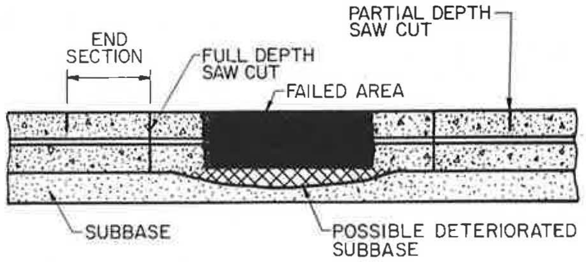
EVALUATING CONDITION OF SUBBASE AND SUBGRADE

All material that has been disturbed or that is loose should be removed and replaced with regular concrete during placement of the patch. If possible, all excessive free moisture should be removed or dried up before the placing of the concrete. If the subbase and subgrade are saturated and considerable water is present, a side French drain should be installed to facilitate drainage. This can be accomplished by cutting a narrow trench through the shoulder. The trench is then back-filled with crushed stone (no fines) and surfaced with asphalt concrete.

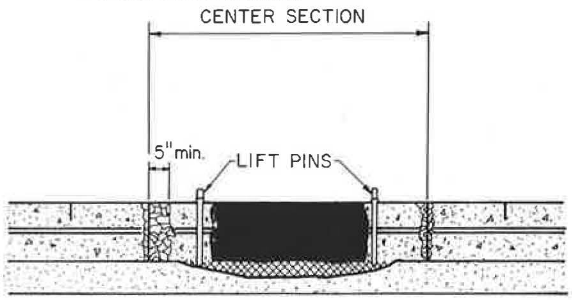
REPLACING AND SPLICING REINFORCING STEEL

The reinforcing steel at the patch ends should be inspected for damage. If more than 10 percent of

Figure 4. Center-section liftout method: partial-depth and full-depth saw cuts (top, step 1); removal of center section (bottom, step 2).



Step 1: Make one partial depth saw cut at the ends of the patch and one full depth saw cut at the proper end section lengths from the edges (18 ins. for tied steel patches, 8 inches for welded steel patches).



Step 2: Breakup concrete down to the steel in the center section at the full depth saw cuts and on one side extend the breakup at least 5 inches into the center section. Remove debris and cut the steel with a saw and remove. Breakup remaining concrete down to the subbase and remove. Drill the required number of holes and insert the lift pins. Carefully remove the deteriorated subbase, if any exists, making sure not to damage the remaining good subbase. The deteriorated area will be filled with portland cement concrete.

the steel is visibly damaged or corroded, the ends of the patch should be extended until this situation is rectified over the required lap length.

New reinforcement of similar size and strength is placed in the patch and spliced to the existing reinforcement at the patch ends by lapping. The required length of embedment of the existing reinforcement into the patch depends on the size and type of reinforcement. Illinois field tests have shown that an embedment length of 18 in is adequate for no. 5 deformed bars and welded deformed wire fabric (0.516-in diameter). This provides an embedment of 29 bar diameters for the no. 5 bar. A no. 4 bar then requires approximately 15 in and a no. 6 bar requires 22-in embedment for 29 bar diameters. This exceeds the American Concrete Institute 318 Code for basic development length. Further research may show that a shorter length is adequate. A minimum 2-in clearance should be provided between the ends of the new rebars and the existing CRCP slab face to allow for possible expansion. The number and spacing of the new reinforcement should match the existing reinforcement as closely as possible.

An alternative to splicing by means of tying the reinforcement as described above is to weld the new reinforcement in place. In some cases, however, the reinforcement in the patch will have been made from old rails, which cannot be easily welded. The origin of the reinforcement should be determined, and if it was old rails, welding should not be attempted. Welding guidelines are as follows:

1. Continuous welds for 0.25 in should be made;
2. The length of the welds should be 4 in (this length develops the full strength of no. 5 bars) and both sides of the bars should be welded; stacking the bars on top of each other is recommended;
3. American Welding Society A5.1 E70XX electrodes should be used;
4. Arc strikes outside the permanent weld area should be avoided and tack welding is expressly prohibited; and
5. To avoid potential buckling, the reinforcement should be lapped at the center of the patch (minimum lap length is 16 in).

The new reinforcing steel bars should be placed according to the following conditions:

1. Reinforcing steel is placed so that a minimum of 2.5 in of cover is provided (if existing steel is less than 2.5 in, the spliced bar is placed under the existing bar), and
2. Reinforcing steel is placed and supported by chairs or other means so that the reinforcement will not be permanently bent down during placement of the patch.

PREPARING PATCH FOR CONCRETE PLACEMENT

If the shoulder has settled below the surface of the slab or is in poor condition, wood side forms should be placed so that it will be possible to strike off the concrete.

It is important to achieve a strong bond between the new patch and the existing concrete. Since concrete bonds better to dry surfaces than to wet surfaces, the ends of the existing CRCP should not be wetted down before concrete placement. Just before placement of the concrete, a neat cement-water grout should be brushed onto the dry ends of the existing CRCP slab. The concrete must be placed before the neat cement-water grout has dried.

PLACING CONCRETE IN PATCH

The concrete should be obtained from a nearby approved ready-mix cement plant. A seven-bag mix of portland type 1 cement has been found to be adequate. The detrimental effects of increased shrinkage and the increase in cost outweigh the increase in strength that a higher cement factor or other types of cement provide. This of course may vary among plants.

Calcium chloride is recommended as a set accelerator in the patching concrete according to the following conditions:

1. The calcium chloride should be added to the ready-mix concrete at the site when the ambient temperature is above 70°F. When the temperature is below 70°F, the calcium chloride can be added at the site or at the plant as long as the length of time from mixing to delivery is less than 15 min. When the calcium chloride is added at the site, a standard solution should be prepared in accordance with standard practice. The ready-mix truck should mix the concrete an additional 40 revolutions after the addition of the calcium chloride at the site.
2. At all times the percentage of calcium chloride by weight of cement is limited to a maximum of 2. It is recommended that no more than 1 percent be used when the ambient temperature is above 80°F because greater percentages can bring on a flash set. It must be noted that on warm days the initial set of the concrete can occur as soon as 30 min after the addition of calcium chloride. Consequently, the patch should be placed and finished as quickly as possible.

On some days, additional early strength could make the difference between opening the patched area to traffic at the end of the day or waiting until the next morning (Table 1). If the engineer fore-

Table 1. Recommended minimum times from placement of patch to opening of patched area to traffic.

Ambient Temperature at Placement (°F)	Minimum Time to Opening (h)			
	Regular Patch	Regular Patch with Addition of		
		Superplasticizer	Insulation	Superplasticizer and Insulation
40-42	54	35	26	22
43-45	49	33	23	20
46-48	44	31	21	18
49-51	39	29	19	16
52-54	35	27	16	13
55-57	32	25	14	11
58-60	29	23	11	8
61-63	26	21	9	7
64-66	23	19	9	6
67-69	21	17	8	5
70-72	18	15	7	5
73-75	15	13	6	5
76-78	13	10	5	5
79-81	10	8	5	5
82-84	9	7	5	5
85-87	8	6	5	5
88-90	7	5	5	5
91-93	6	5	5[a]	5[a]
Above 93	5	5	5[a]	5[a]

[a]Insulation should not be used when ambient temperature is <90°F.

sees this to be the case, an approved superplasticizer may be added to the concrete in order to gain the additional strength:

1. When a superplasticizer is to be used, the mix at the ready-mix plant should be altered to produce the following: (a) a 1-in slump concrete and (b) a concrete that has approximately 8 percent entrained air.
2. The superplasticizer should always be added at the site. If calcium chloride is to be added at the site also, the calcium chloride should be added according to the provisions of the previous section and before the addition of the superplasticizer. The superplasticizer should be added immediately after the calcium chloride has been thoroughly mixed.
3. The superplasticizer should be added in accordance with the instructions supplied by the manufacturer to provide a 7-in slump concrete for easy placement (never more than a 9-in slump, because segregation will occur). If the concrete begins to stiffen by the time the second or third patch is to be poured, an additional reduced dose of the superplasticizer may be added. (Note: This may be repeated as many times as is necessary but, except for the initial dose, no more should be added than the amount necessary to increase the slump 1 in at any one time.) It is recommended that a large plastic container that has volume markings along the outside be used to measure the superplasticizer. The ready-mix truck should mix for 2 min at high speed after every addition of the superplasticizer (including the initial dose).

Addition of water to the concrete at the site should be avoided unless absolutely necessary because of the detrimental effects this has on the strength development and increased shrinkage.

1. If calcium chloride is being added at the plant and the concrete is consistently too stiff on arrival at the site, the calcium chloride should be added at the site.
2. If, after the addition of calcium chloride at the site, the concrete is too stiff, the operator of the ready-mix plant should be notified to increase the slump by an appropriate amount.

The entire patch area should be consolidated by an appropriate-sized spud vibrator, particularly around the edges of the patch. The use of a vibrator not only consolidates the concrete but also helps in the finishing process, thus avoiding unnecessarily high slumps.

The casting of patches before noon is not recommended, especially during the summer, since expansion of the CRCP end sections leads to crushing of the weak concrete patch. However, experience in some localities may not show this to be a problem for CRCP.

CURING OF PATCH CONCRETE

A liquid membrane curing compound should be sprayed over the concrete in a uniform manner. On days when the wind speed is more than 10 mph, light-colored or clear polyethylene sheeting should be placed over the concrete to reduce the amount of moisture loss from the surface. It can be held down by means of reinforcing steel along the edges or by any other similar weight.

The placement of insulation 4 in thick over the patch is highly recommended to maintain a high temperature for curing and permit early opening of the patched area. However, insulation should not be placed when the ambient temperature is more than 90°F.

1. Polyethylene sheeting should be placed on the concrete before the insulation is laid down, and
2. The insulation should be held down by reinforcing steel or a similar weight.

OPENING PATCH TO TRAFFIC

There are many factors that influence the length of time necessary for concrete to develop strength sufficient to safely resist traffic loads. As a result of a comprehensive study (5), the ambient temperature at placement has been found to be by far the most influential factor on the strength development of concrete patches. Consequently, Table 1 gives the minimum number of hours that the concrete must be allowed to cure before the patched area is opened to traffic, and this time is solely dependent on the ambient temperature at placement. Included in the table are reduced curing times when a superplasticizer or insulation or both are used according to the provisions set forth in this paper. The minimum times before the opening to traffic are based on achieving a modulus of rupture of at least 300 psi within the patch, which was found through field tests to be adequate (5).

SUMMARY

Based on many laboratory and field tests and analyses, new cost-effective permanent patching procedures have been developed. The procedures reduce costs and lane-closure time by (a) adapting the patch size and type to fit the several different distress types and extent, (b) reducing reinforcement embedment length into the patch, (c) using mechanized equipment for construction wherever possible, and (d) using concrete additives and curing techniques so that there can be an early opening to traffic. Practical field-tested procedures were developed for the efficient and long-lasting patching of CRCP.

ACKNOWLEDGMENT

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The contents of this paper reflect my views and I am responsible for the facts and the accuracy of the data presented here. The contents do not necessarily reflect the official views or policies of the Illinois Department of Transportation or the Federal Highway Administration. This paper does not constitute a standard, specification, or regulation.

REFERENCES

1. S.A. LaCoursiere, M.I. Darter, and S.A. Smiley. Performance of Continuously Reinforced Concrete Pavement in Illinois. Univ. of Illinois, Urbana-Champaign; Illinois Department of Transportation, Springfield; and Federal Highway Administration, U.S. Department of Transportation, Rept. FHWA-IL-UI-172, 1978.
2. M.I. Darter, S.A. LaCoursiere, and S.A. Smiley. Structural Distress Mechanisms in Continuously Reinforced Concrete Pavement. TRB, Transportation Research Record 715, 1979, pp. 1-6.
3. D.J. Maxey, M.I. Darter, and S.A. Smiley. Evaluation of Patching of Continuously Reinforced Concrete Pavement in Illinois. Univ. of Illinois, Urbana-Champaign; Illinois Department of Transportation, Springfield; and Federal Highway Administration, U.S. Department of Transportation, Rept. FHWA-IL-UI-176, 1979.
4. D.J. Maxey, M.I. Darter, and S.A. Smiley. Evaluation of CRCP Patching. TRB, Transportation Research Record 727, 1979, pp. 9-16.
5. M.I. Darter, T.L. Barnett, and D.J. Morrill. Repair and Preventive Maintenance Procedures for CRCP. Univ. of Illinois, Urbana-Champaign, Res. Rept. 901-4, in preparation.
6. J.E. Simonsen. Development of Procedures for Replacing Joints in Concrete Pavement. Michigan Department of Transportation, Lansing, Final Rept. R-1020, 1976.

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Evaluation of Several Maintenance Methods for Continuously Reinforced Concrete Pavement

ELDON J. YODER, R.H. FLORENCE, JR., AND STANLEY J. VIRKLER

Research on continuously reinforced concrete pavement (CRCP) has been going on at Purdue University for the Indiana State Highway Commission since 1971. The primary objective of the overall research program has been to evaluate the performance of CRCP in Indiana and to make recommendations relative to design and construction techniques that might improve the performance of this type of pavement. Primary factors found to contribute to performance of CRCP in Indiana have been subbase type, method of construction, and traffic. The usual method of maintaining CRCP has been to patch failures by using reinforced concrete. This research has been done to evaluate other techniques for maintaining CRCP to determine the most cost-effective method. A test pavement on Interstate-65 south of Indianapolis was used. Maintenance methods investigated included normal concrete patching, bituminous patching, overlay by using asphalt concrete with and without prior undersealing and with and without installation of edge drains, undersealing by using asphalt only, drainage, and concrete shoulders. The results show that overlaying the pavements by using asphalt concrete completely stopped the progression of failures within the time frame of this research. Undersealing by using asphalt was also effective. The edge drains and concrete shoulders were not effective. The performance data were also analyzed in light of the cost of maintenance.

The use of continuously reinforced concrete pavement (CRCP) in Indiana dates back to 1938, when an experimental project was first built on US-40 near Stilesville, Indiana. The mileage of CRCP increased until the end of 1971, when 1120 km (696 miles) of equivalent two-lane CRCP were in service in the state.

In 1973, a continuing study of the performance of CRCP was initiated by the Joint Highway Research Project at Purdue University. The objective of the study was to evaluate performance and to recommend design and construction techniques.

HISTORY OF RESEARCH ON CRCP IN INDIANA

Primary emphasis was placed on construction of CRCP from 1967 to 1971. By the spring of 1972 it became apparent that distress was occurring on some of the pavements. Purdue University was first contacted in July 1972 regarding the problem; at that time, plans were made for a long-range research project.

The procedure below briefly summarizes the process of the research.

1. Detailed study of performance on Interstate-65, a major Interstate between Indianapolis and Chicago;
2. Statewide performance survey of all CRCP in Indiana;
3. Detailed study of selected pavements, including field measurements;
4. Laboratory evaluation of materials obtained in step 3;
5. Analysis of factors that influence performance;
6. Construction of test sections of several types of maintenance in the fall of 1975 on I-65 south of Indianapolis; and

7. Evaluation of test pavements through the spring of 1979.

Significant Factors That Influence Performance

After several months of research on CRCP in Indiana, it became apparent that several factors were showing up as statistically significant with respect to design and performance of this type of pavement in Indiana. These factors have been discussed in detail in several reports (1-4). The factors that influence the performance of CRCP include (a) subbase type, (b) type of steel fabrication, (c) interaction between steel placement methods and construction, and (d) traffic.

In the following discussion, some of the features that have been found to influence the performance of CRCP are mentioned. They are not necessarily discussed in the order of importance, since it is believed that there is no single factor that has influenced performance. Rather, it is a combination of factors that have set up a series of circumstances that have caused failures to occur.

Subbase Type

The early surveys showed that pumping was occurring on the pavements that showed the greatest amount of distress. Most of the pumping was occurring on the gravel subbases, whereas the crushed-stone and slag subbases were apparently showing good performance.

Table 1 describes the variation in California bearing ratio (CBR), permeability K, and degree of compaction with subbase type. Although no clear differences in the properties of the gravel subbases were evident between sections that had failures and those that showed no apparent distress, the gravel subbases were not sufficiently compacted and in addition had relatively low permeability and strength.

The results brought to light important differences among the properties of the different subbase types. Crushed-stone subbases were the most permeable, whereas slag subbases had the lowest permeability. The relatively poor water-transmission characteristic of the slag subbase was more than balanced by its high strength, as indicated by the CBR.

It is worthwhile to note that concrete pavement performance is also a function of the interaction between subbase permeability and strength (CBR). In Figure 1, the estimated field permeability values are plotted against field subbase CBR values measured at the interface between the shoulder and the slab. These values pertain to 46 test locations of the detailed field study. Data points for crushed-stone and slag subbases are shown by using subscripts. In addition, values obtained at failed test locations are differentiated from the values at good test locations. The data were grouped in nine categories that corresponded to three levels each of subbase CBR and permeability. For low subbase strength (CBR < 40 percent), mainly gravel subbases, the percentage of failed test locations decreased from 53 percent in the low-permeability group [K < 0.05 cm/s (142 ft/day)] to 25 percent in the high-permeability group [K > 0.5 cm/s (1418 ft/day)]. For medium subbase strength (40 percent < CBR < 80 percent), no failures were observed when permeability was greater than 0.5 cm/s. When subbase strength (CBR > 80 percent) was high (only slag and crushed-stone subbases), no failures were indicated irrespective of permeability.

It had been suggested in reports submitted to the Indiana State Highway Commission from the outset that pumping was a major contributor to pavement distress and that the gravel subbases have shown the poorest performance. The cold-mix bituminous stabilized bases showed fair to good performance. It was recommended that the Indiana State Highway Commission discontinue use of gravel subbases and that crushed-stone subbases (drained) or stabilized subbases be used. Data obtained suggest that slag subbases are showing good performance.

Percentage of Steel

A standard of 0.6 percent steel had been adopted by most states for use in CRCP up to the time of this research.

The temperature drop used by the Indiana State Highway Commission for the design of these pavements is 38°C (100°F). However, in the central and northern portions of Indiana, temperature changes from extreme highs during the construction season in midsummer to lows in the winter may be as great as 52°C (125°F) or more (4). According to computations of the effect of total temperature drop on required percentages of steel by using split-tensile-strength values from pavement cores, a temperature drop of 52°C requires more than 0.6 percent steel. This suggests that 0.6 percent steel was too low for conditions in Indiana.

Steel Placement

The survey data have shown that a major contributor to failures that occur on CRCP in Indiana can be associated with use of preset steel set on chairs. It was recommended that the state discontinue use of preset steel. Performance surveys suggested that bar mats showed the poorest performance.

Indiana pavement design has followed the standard recommendations of using a steel reduction factor of about 0.75. Various individuals in the United States have suggested that reduction in thickness should not be permitted. In spite of the lack of substantiating data, it is probable that the thickness design used on CRCP in Indiana was at best marginal.

Construction and Other Factors

The discussion so far has dealt primarily with factors that can be assumed to be in the category of design. The total research study, however, has dealt with other factors that might affect the performance of CRCP. Reference is made in particular to the report by Faiz and Yoder (3) that dealt with physical properties determined in the field on selected pavements during the summer of 1973.

Among the more-important significant factors was the effect of the bulk density of the concrete on performance of the pavements. Concrete at failed locations had lower bulk densities than did that at good locations. Likewise, the dynamic modulus of elasticity was found to be a contributing factor. On the other hand, the split-tensile-strength values were not found to be significantly different between failed and structurally sound locations.

The results of the study showed that there was little difference between the properties above and below the steel reinforcement.

The results of the field survey indicated that no significant difference existed in the mean crack interval between good and failed locations. It was shown that most of the failed sections were associated with intersecting cracks.

Summary of Field Performance Findings

The comments offered below pertain to research on

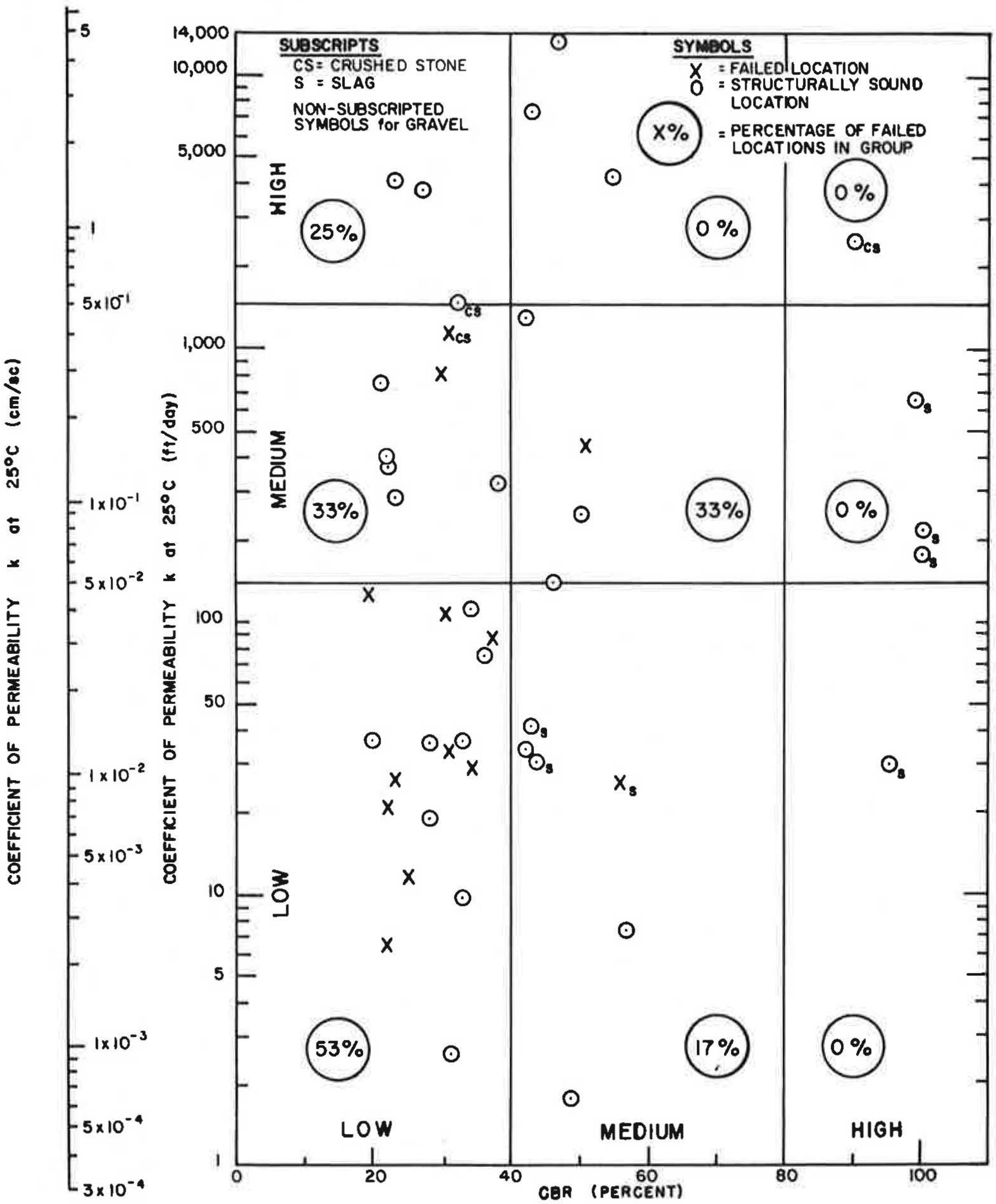
Table 1. Effect of subbase type on pavement condition.

Type of Subbase	Condition of Test Section	No. of Test Sections	Subbase CBR (%) Sampled at Pavement Edge	Subbase CBR (%) Sampled at Center Through Core Hole	Permeability K (cm/s)	Compaction (%)
Gravel	With failures	13	33.5	38.0	0.4[a]	91.5[a]
	Without failures	10	35.9	45.2	0.25[b]	93.2[b]
Slag	With failures	1	95.0	100.0	0.01	117.2
	Without failures	3	81.0	96.6	0.05	100.6
Crushed stone	With failures	1	32.0	41.0	0.7	89.9
	Without failures	1	90.0	59.0	1.5	85.3

Notes: Data are from structurally sound test locations.
1 cm/s = 2835 ft/day.

[a]Average of values from nine test sections. [b]Average of values from 10 test sections.

Figure 1. Effect of subbase strength and permeability on CRCP performance.



factors that have influenced performance of CRCP in Indiana.

In the detailed field evaluation, the comparison of test sections that had failures with sections that did not have failures relative to material properties and performance characteristics evaluated at structurally sound test locations resulted in a number of significant results. Similarly, the evaluation of sectionwide pavement characteristics also established some significant trends. These findings bring to light deficiencies inherent in the pavement structure that eventually lead to distress. The following is a summary of the significant results:

1. Subgrade properties: The only significant result in the analysis of subgrade properties was that subgrade soils at sections without failures were relatively more coarse grained and sandy than those soils at sections in which failures had occurred.

2. Subbase properties: This analysis clarified the reasons for the better performance of certain subbase types. Crushed-stone subbase at the section without failures was found to possess high strength (CBR, 90 percent) and excellent internal drainage [more than 0.7 cm/s (1985 ft/day)]. The failure at another section that had a crushed-stone subbase was a function of poor stability (very low CBR), which resulted from inadequate compaction. The good condition of pavements on slag subbases was due to the very high stability (CBR, more than 100 percent) of this subbase. At structurally sound locations, gravel subbases were found to have moderately high permeability but poor stability characteristics, probably a function of insufficient compaction.

3. Concrete properties: Sections that showed no failures were paved with concrete that had a higher degree of slump. The results of the data analysis further indicated that the modulus of elasticity of concrete had a significant bearing on pavement condition. Concrete cores obtained from sections that had no distress were tested to have an average dynamic modulus of elasticity of 4.32×10^9 kPa (6.15×10^6 psi), whereas cores obtained from good locations on sections that had failures had an average dynamic modulus of 3.49×10^9 kPa (4.97×10^6 psi).

4. Dynamic pavement deflection: Dynamic pavement deflections were shown to be a good indicator of pavement condition if used judiciously. Once the continuous slab breaks up into discrete segments, the usefulness of deflection measurements is impaired. As expected, at good test locations no difference in dynamic deflections was observed between sections that had failures and sections that did not have failures.

An evaluation of sectionwide deflection measurements taken 1.82 m (6.0 ft) from the pavement edge showed that for 228-mm (9-in) CRCP, dynamic deflections measured by Dynaflect less than 127×10^{-2} mm (0.5 mil) are indicators of good pavement condition. Deflections in the range of 1.54-2.28 x 10^{-2} mm (0.6-0.9 mil) indicate a potential distress condition, whereas values above 2.28×10^{-2} mm are indicators of severe distress in areas where there is a high probability of pavement breakups.

5. Crack width: It was noted that cracks observed at test sections that had failures were significantly wider than those measured at good test sections, even though crack widths were measured only at structurally intact locations. The average crack width at good sections was 0.218 mm (8.6×10^{-2} in).

6. Crack spacing: No difference in either the mean crack spacing or the variance of crack intervals was observed between sections in the two categories. The variance of crack spacing at failed test locations was significantly higher than the variance at good locations. Frequent incidence of bifurcated cracks as well as closely spaced cracks that may intersect at a later date was observed to be associated with failures. Also, high incidence of very closely spaced cracks is indicative of incipient failure.

PURPOSE OF RESEARCH

It was the purpose of this research to design, construct, and evaluate several different types of maintenance for CRCP. Deflection, the amount of cracking, and the amount of breakups were used as significant indicators of performance and were the basis of selection of the experimental sections.

Previous research on CRCP in Indiana suggested that deflections measured by the Dynaflect that are greater than 2.3×10^{-2} mm indicate poor pavement condition. Hence, it follows that reduction of deflections is one method of reducing potential failure of the pavement. Poor drainage, often caused by the previously mentioned factors, was considered to be a major contributor to high deflection values. A third factor was found to be the use of bar mats and preset steel on chairs.

A section of I-65 south of Indianapolis was chosen for this project. This section extends south from the Greenwood exit of I-65 to the Whiteland exit and the test pavements included both the northbound and southbound lanes. This resulted in approximately 7.35 km (4.57 miles) in each direction or a total of about 14.71 km (9.14 miles) of test pavement.

This section was selected for study since it contains all the significant features identified as major contributors to poor performance of CRCP. It has a gravel subbase, and bar mats and chairs were used--three important criteria discussed earlier that indicate potential failure. The grade is relatively flat, soils are fine-grained glacial till, and drainage is generally poor. This pavement has shown, as predicted, very poor performance.

METHOD OF ESTABLISHING TEST SECTIONS

Initial Tests

Deflection measurements and a condition survey of the test pavement were made in the fall of 1974. Deflection readings were taken by using the Dynaflect at 7.6-m (25-ft) intervals over the study area. The condition survey was made by noting the size and location of breakups and the location and linear feet of closely spaced parallel cracks, intersecting cracks, and combined parallel and intersecting cracks. Figure 2 shows the criteria used for determining the linear feet of cracking. Linear feet of cracking (L, Figure 2) was the longitudinal distance observed for a specified type of crack.

Selection of Study Sections

By using the data derived from the above field observations, three factors were chosen as indicative of the overall condition of the pavement. These were (a) linear distance of cracks less than 762 mm (30 in) plus distance of intersecting cracks per 30.48-m (100-ft) section, (b) total area of patching or breakups per station, and (c) maximum deflection per 30.48-m section. By using this technique, the sections of pavement were then stratified and assigned rating numbers of 1-12 as shown in Figure 3.

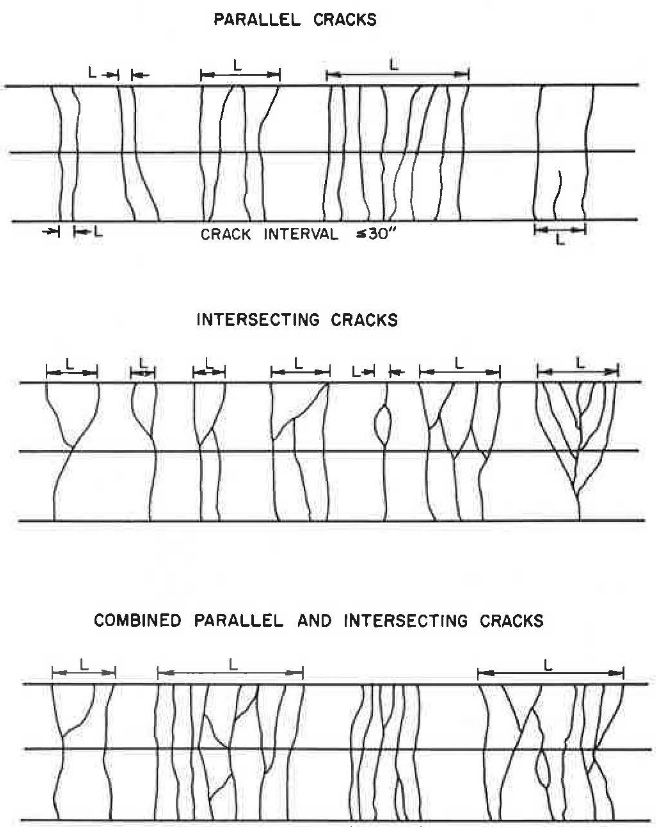
Selection of Maintenance Methods

The types of maintenance that were considered to be appropriate for the given rating numbers are given below. (This list was used as a "shopping list" from which maintenance types could be chosen to fit construction needs.) An attempt was made to apply as many types of appropriate maintenance as possible to the various ratings, but in some cases not all the types of maintenance were used.

Rating	Type of Maintenance
1	None
2	None, patch
3	None, patch

Rating	Type of Maintenance
4	None, underseal and overlay, underseal, overlay, concrete shoulders, drain
5	None; patch, underseal, and overlay; patch and underseal; patch and overlay; patch and concrete shoulders; patch and drain; patch, drain, and concrete shoulders
6	None; patch, underseal, and overlay; patch and underseal; patch and overlay; patch and concrete shoulders; patch and drain
7	None, drain, overlay
8	None, patch and overlay, patch and concrete shoulders, patch and drain, patch
9	None, patch and overlay, patch and concrete shoulders, patch and drain, patch
10	None, underseal and overlay, concrete shoulders, drain
11	None; patch, underseal, and overlay; patch and drain; patch and concrete shoulders; full-depth bituminous
12	None; patch, underseal, and overlay; patch and drain; patch and concrete shoulders; full-depth bituminous; patch, underseal, overlay, drain, and concrete shoulders; patch, drain, and concrete shoulders

Figure 2. Typical crack patterns.



Note: 1 in. = 25 mm.

Practical combinations of techniques that were possible are as follows:

1. Underseal prior to overlay,
2. Underseal prior to construction of concrete shoulders,
3. Install drains prior to overlay, and
4. Install drains prior to construction of concrete shoulders.

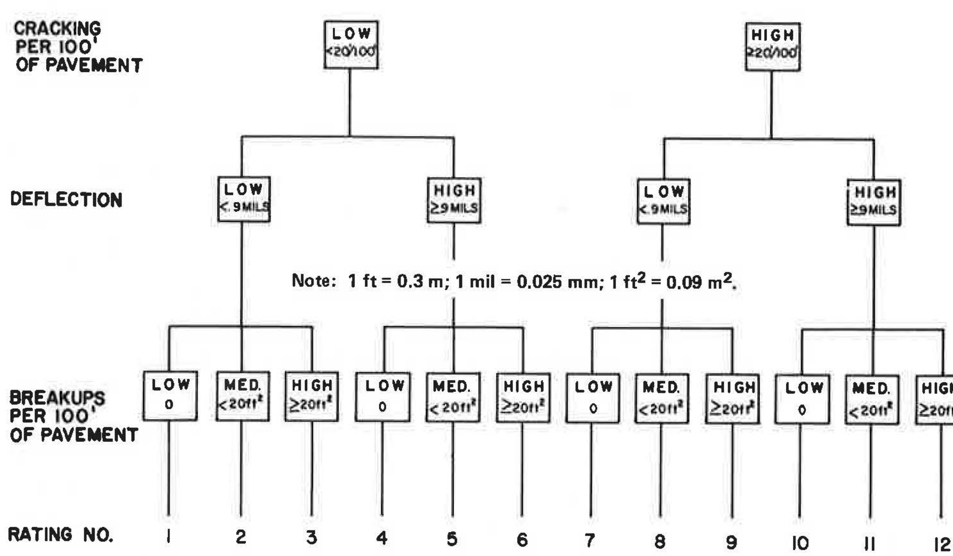
The list was compiled during a conference of personnel from Purdue University, the Indiana State Highway Commission, and the Federal Highway Administration.

Layout of Study Sections

The layout of study sections of a given maintenance type was governed by four criteria:

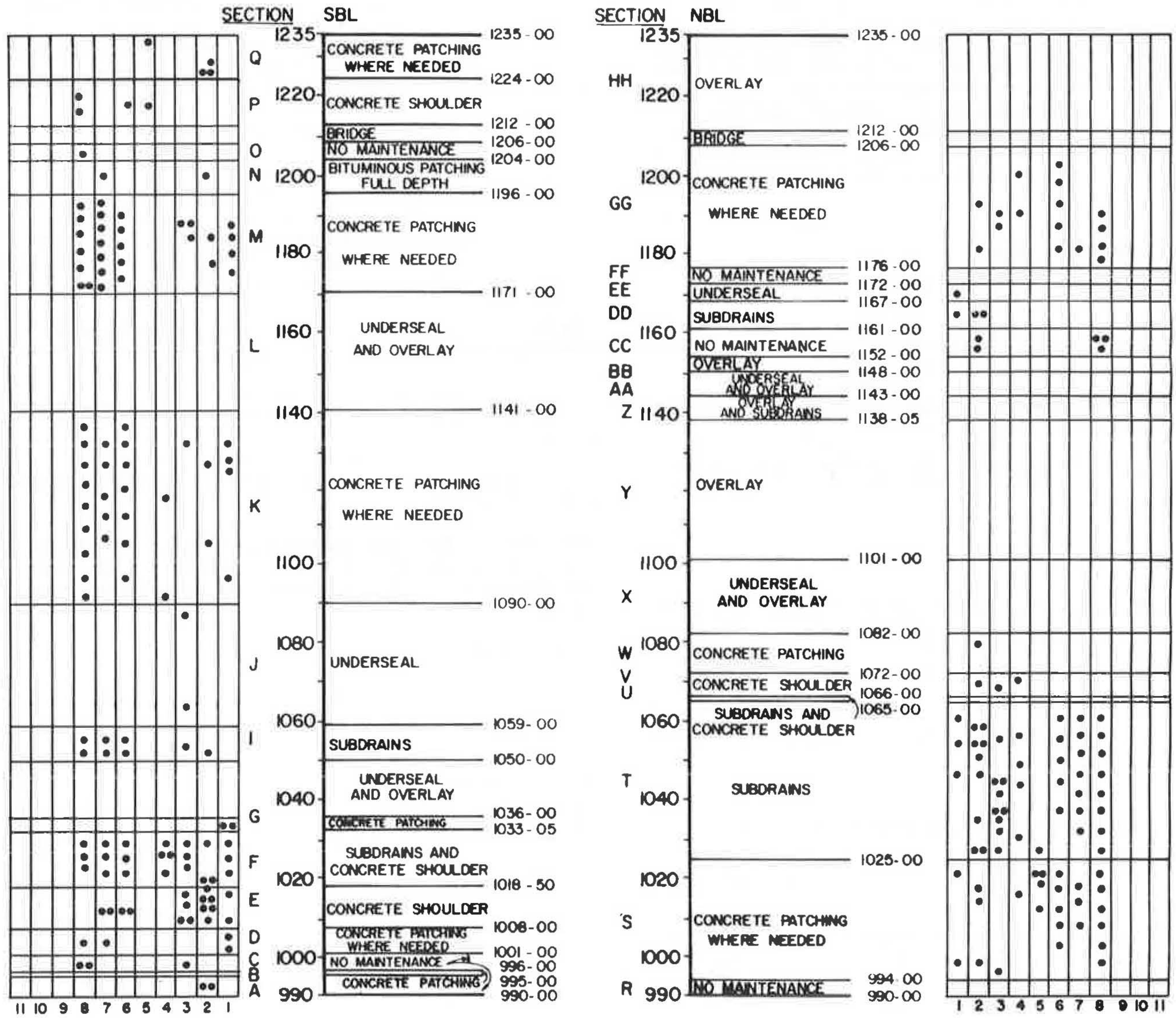
1. All sections were patched by using concrete as minimum maintenance. Hence, the performance of all sections started on a uniform basis, i.e., completely repaired pavement.
2. It was desirable to have a section of one type of maintenance be as long as possible.
3. At least one control section was retained for each rating number (1-12); these control sections were patched as needed.
4. As many different types of maintenance methods were used as possible for each rating number.

Figure 3. Rating of test sections.



Note: 1 ft = 0.3 m; 1 mil = 0.025 mm; 1 ft^2 = 0.09 m^2.

Figure 4. Failure as a function of location and time.



MAINTENANCE METHODS USED

The maintenance test sections were constructed by using standard techniques under the supervision of project engineers, as is normally done. Purdue personnel merely advised when needed on technical matters that arose during the construction.

All the breakups on the project were patched by using concrete regardless of the type of maintenance. The exceptions were those sections designated bituminous-patch sections, in which bituminous patches were used in lieu of concrete patches for the purpose of evaluating them, and the no-maintenance sections, which required no maintenance at all. Figure 4 shows the layout of the maintenance test sections as they were finally constructed.

Concrete Shoulders

The intent of using the concrete shoulders was to stiffen the pavement and thereby reduce deflection and subsequent pumping and failure of the pavement. The concrete shoulders were constructed so as to have contraction joints spaced at 4.5-m (15-ft) intervals. The slabs were 152 mm (6 in) thick on the outside edge and 229 mm (9 in) thick on the inside edge. They were tied to the existing pavement by using no. 4 tie bars 762 mm (30 in) long.

The tie bars were spaced at three different intervals on centers: 0.304 m (1 ft), 0.609 m (2 ft), and 0.762 m (2 ft 6 in). One section 365 m (1200 ft) long was tied into an existing keyway, whereas the remaining 0.609 m was tied to the vertical face of the pavement; thus, a butt joint was formed. The tie bars were omitted 0.762 m on either side of the construction joints to permit independent movement between the existing CRCP and the concrete shoulder.

After excavation and grading of the existing asphalt concrete shoulder, holes were drilled into the existing pavement to a depth of 355 mm (14 in) by using a tractor-mounted drill. The tie bars were grouted into the existing pavement by using a combination of epoxy and a nonshrinking grout.

Rumble strips were installed every 18.5 m (60 ft) along the concrete shoulder. Construction joints were installed at the end of each day's pour. These joints were always located at the middle of a slab and tied by using tie bars.

Subdrains

One of the major contributors to performance of this pavement was pumping, which resulted from free water on top of the subbase. For this project, subdrains were installed directly adjacent to the pavement edge. This was done in an effort to trap and remove water from the top of the subbase as quickly as possible and to drain infiltration water from between the pavement and the shoulder.

The backfill for the subdrains was carried up beyond the bottom edge of the pavement and to within 77 mm (3 in) of the top of the pavement. The top 77 mm was filled with asphalt concrete shoulder material.

Undersealing

Since pumping was considered one of the primary modes of failure for this pavement, undersealing (both with and without an overlay of asphalt concrete) was a major variable in the experimental layout. The underseal was intended to fill voids that might exist between the pavement and subbase.

The pavement was undersealed by using asphalt at specified locations. Holes of 0.07-mm (2-in) diameter were drilled through the pavement in the right-hand lane. After pumping had been completed, the hole was filled by a wooden plug.

When the undersealing operation first began, the proper spacing of the holes was not known. Hence, it was necessary to establish criteria that could be used by the contractor for spacing the holes and pumping the asphalt with a minimum of delay during the construction process.

In general, one row of holes spaced 2.4 m (8 ft) on centers was used at locations at which deflections were greater than 2.28×10^{-2} mm and there was uniform crack spacing. Two rows staggered every 1.2 m (4 ft) were used at locations that had high deflections associated with closely spaced cracking. At least two holes were drilled on each side of a potential failure or on each side of a potential patch.

If the deflection was less than 2.28×10^{-2} mm, the pavement was not undersealed. Hence, within each underseal section, some of the pavement was not undersealed, depending on the deflection value. Deflection was used as a criterion for undersealing to preclude creation of possible voids by lifting the pavement from the subbase.

Asphalt Concrete Overlay

The intent of the asphalt concrete overlays was to reduce deflection but, in addition, to perhaps prevent infiltration of water and thus improve performance. It was not known at the time of construction how thickness of overlay might influence performance, particularly from the standpoint of reflection cracking. Therefore, thickness of overlay was included as one of the variables of the experiment.

As can be seen from Figure 4, overlay was used on the southbound lanes in sections H and L. In the northbound lanes the primary overlay areas were in sections X and Y.

In general, a standard overlay thickness of 51 mm (2 in) was used, except as noted below. The existing pavement was overlaid by using a bituminous base 75 mm (3 in) thick on all locations except the longest overlay on the northbound lane (NBL) between stations 1082+00 to 1152+00 (sections X to BB). In this section an overlay wedge was formed by varying the base thickness as shown below (1 mm = 0.04 in):

NBL Station Limits	Thickness (mm)
1082+00-1110+00	51
1110+00-1119+00	76
1119+00-1129+00	127
1129+00-1152+00	76

Concrete Patching

As mentioned previously, concrete patching was required on all sections as the basic form of repair prior to the placing of any other type of maintenance. This is with the exception of section N, in which the basic experiment called for full-depth bituminous patching. The layout of the experiment required that any location in which breakup had occurred must be patched in addition to use of any other kind of maintenance.

In Figure 4, it may be noted that there were several no-maintenance sections in the experiment. These were sections of pavement that were considered to be in good repair. Hence, they were excluded from further analysis.

Full-Depth Bituminous Patching

Full-depth patching was included to compare the effect of bituminous patching with that of the concrete patching previously discussed. A minimal amount of full-depth patching was used. In addition, however, during the subsequent maintenance of the pavement, bituminous patches were placed if small failures existed.

Construction of the full-depth bituminous patches was similar to that of the concrete patches; however, all the existing steel within the patched area was removed along with the concrete to the pavement depth. After the subbase had been compacted, a no. 5 asphalt base was installed with a type-B surface, which brought the patch back to the existing grade.

PERFORMANCE OF TEST PAVEMENT

The maintenance test sections were completed in the fall of 1975. A complete performance survey was made immediately prior to construction of the test sections. The performance surveys were repeated immediately after construction of the test sections and then were repeated again each spring and fall until the fall of 1977. These surveys included complete logging of the extent of cracking that had occurred and of distress (punchouts) that had taken place and measurement of deflection by using the Dynaflect at 7.6-m (25-ft) intervals. Deflections were made by using only the sensor immediately under the vibrating wheels of the Dynaflect.

For the purposes of this paper, emphasis is placed on the effectiveness of each of the maintenance methods. Data relative to the cost-effectiveness of each of the methods will be presented.

The primary factor considered here is the occurrence of breakups (primarily punchouts) on each of the test sections and comparison of this occurrence with that on the corresponding control sections. It is to be recalled that each test section had a corresponding control section that had an identical original rating number determined by the criteria in Figure 3.

Progression of Breakup Occurrence with Time

Figure 4 shows the number of failures that have occurred on the test pavement during various time periods. This test pavement is considered to be a severe test of maintenance methods since it contained most of the features that have been known to contribute to poor performance of CRCP in Indiana.

Table 2. Summary of failure data.

Item	Failures per 100 m	
	Through Fall 1977	Through Spring 1979
Control section	0.79	1.656
Maintenance method		
Underseal and 51-mm overlay	0	0
Underseal and 76-mm overlay	0	0
51-mm overlay	0	0
76-mm overlay	0	0
127-mm overlay	0	0
Subdrain and 76-mm overlay	0	0
Underseal	0[a]	0[a]
Bituminous patch	0.41	0.94
Subdrain	1.73	3.281
Concrete shoulder	1.84	6.43
Subdrain and concrete shoulder	2.92	4.941

Note: 1 mm = 0.04 in.

[a]Several failures have occurred in the underseal section but not at locations actually underseated.

(The numbered columns in Figure 4 relate to the periods of survey, as tabulated in the next paragraph.)

The occurrence of failures bore a general relationship to the season of the year. The data showed that a large portion of the failures occurred during fall, winter, and spring and that the occurrence of failures was reduced during the summer. As shown below, 65.8 percent of the failures occurred during fall, winter, and spring, and 34.2 percent occurred during spring and summer:

Survey	Period	Season	Total (%)
1	Aug. 1975-March 1976	Winter	10.5
2	March 1976-Aug. 1976	Spring-summer	17.5
3	Aug. 1976-Dec. 1976	Fall-winter	12.3
4	Dec. 1976-July 1977	Winter-spring	6.1
5	July 1977-Oct. 1977	Summer	3.1
6	Oct. 1977-April 1978	Fall-winter	15.7
7	April 1978-Oct. 1978	Spring-summer	13.6
8	Oct. 1978-June 1979	Fall-winter	21.2

Results of Failure Surveys

Solely on the basis of the occurrence of additional failures after construction of the test sections, the data indicated that the overlay sections performed better than any of the other test methods did. A summary of failures that have occurred since construction of the test sections is listed in Table 2. In this table, the data are arranged in descending order relative to number of failures per 100 m.

It is apparent from these data that the concrete shoulders were not effective in this particular experiment nor were the subdrains. A large portion of the concrete-shoulder sections were concentrated at the southern end of the southbound lanes at which poor performance had been demonstrated prior to the experiment; hence the data may have been influenced by factors other than the use of shoulders themselves.

The subdrains that were installed in this particular test project were relatively deep and are no doubt draining the subgrade as well as the subbase. The back-filling operation adjacent to the pavement edge was always carried up to the mid-depth of the concrete pavement, and therefore any water that lies on the subbase should go through the drains. It is known that the drains are functioning. However, the subbase under the pavement is impervious and contains a high percentage of fines. Therefore, rapid percolation of the water through the subbase itself is probably not taking place, and any water that is drained from the subbase is that which lies at the interface between pavement and subbase.

COST ANALYSIS

The cost analysis in this study consisted of an evaluation of the cost-effectiveness of the various methods. These data are summarized in Table 3. It may be seen that the overlay method was very effective in stopping the progression of failures but, except for the concrete-shoulder sections, it was the most costly.

The underseal appeared to be a cost-effective means of at least slowing down the progression of failures, since no failures have been found in those sections that have actually been undersealed.

The subdrain and concrete-shoulder sections were found to be the least cost-effective means of stopping progression of failures. It is to be recalled, however, that the concrete shoulders and subdrains were used mainly at the extreme southern end of a construction job on which an initial high incidence of failures had taken place prior to construction of the test pavements.

Construction Costs Compared with Patching Costs

Table 3 illustrates that only seven of the methods (the first seven listed) were effective in reducing or completely stopping occurrence of failures in the test pavement. It may be seen that all but the last of these includes an overlay of asphalt concrete. The last method is undersealing without the use of an overlay. In the analysis that is presented in Table 4, the last four methods in Table 3 and the control section are excluded.

For the data in Table 4 it is assumed that all the failures were patched during the life of the test, although all the failures were not necessarily patched. It is further assumed that failures that occurred during the summer, fall, and winter months would be repaired during the next construction season. The cost of patching is given in 1975 dollars, and the average cost of patching for this experiment ($1515 per patch) is discounted at the rate of 6 percent.

The data indicate that the overlay, although very effective in stopping failures, was not cost-effective during the period of the experiment when cost of constructing the maintenance was compared with the average cost of maintaining the pavements (last row of table). The underseal sections were constructed at an average cost of $1020/100 m ($311/100 ft). This technique slowed down occurrence of failures. The data suggest that undersealing is a viable method for slowing down the occurrence of distress in pavements of this type.

PERFORMANCE SUMMARY

Three types of maintenance were investigated in this research project other than the usual concrete patching done on a routine basis in the state. Various combinations of these maintenance techniques were used, and variations of the methods were included in some cases. The basic methods include the following: (a) overlay by using asphalt concrete both with and without drainage and undersealing, (b) installation of subdrains at the outside edge of the pavement, and (c) concrete shoulders.

It is pertinent to note first that the test pavement was already among the worst-performing pavements in the state at the time the experiment was set up. Because of the high incidence of failures in the test pavement prior to establishing the main-

Table 3. Summary of performance data.

Item	Length (m)	Cost[a] ($/100 m)	Failures per 100 m Through Fall 1977		Failures per 100 m Through Spring 1979	
			Maintenance Section	Control Section	Maintenance Section	Control Section
Control section	6214	2 385	-	0.787	-	1.656
Maintenance method						
Underseal and 51-mm overlay	853	4 771	0	1.788	0	3.281
Underseal and 76-mm overlay	1219	6 772	0	0.731	0	1.916
51-mm overlay	274	4 351	0	0.862	0	1.043
76-mm overlay	1373	6 404	0	1.082	0	1.906
127-mm overlay	304	6 598	0	1.289	0	2.46
Subdrain and 76-mm overlay	150	7 149	0	0.935	0	1.873
Underseal	1097	1 020	0[b]	0.397	0[b]	1.036
Bituminous patch	243	3 780	0.41	0.82	0.935	1.886
Subdrain	1676	3 307	1.729	1.007	3.281	2.083
Concrete shoulder	870	9 876	1.837	1.115	6.43	1.902
Subdrain and concrete shoulder	443	11 519	2.92	0.935	4.941	1.4

Note: 1 mm = 0.04 in.

[a]Includes cost of patching before construction of maintenance section.
[b]Three failures have occurred in the underseal section at locations that were not undersealed.

Table 4. Cost of maintenance sections and potential cost of patching control sections.

Maintenance Method	Cost of Maintaining Pavements ($/100 m)				
	Maintenance Section	Control Section			
		Year 1	Year 2	Year 3	Year 4
Underseal and 51-mm overlay	4711	649	1709	2792	4209
Underseal and 76-mm overlay	6772	377	990	1653	2483
51-mm overlay	4351	203	538	882	1335
76-mm overlay	6404	374	990	1653	2483
127-mm overlay	6598	485	1260	2093	3156
Subdrain and 76-mm overlay	7149	364	971	1591	2401
Underseal	1020	203	538	879	1325
Avg	-	380	997	1647	2483

Notes: Patching costs are based on the assumption that all failures are patched and at an average cost of $1515/patch. Costs are accumulated with time.
Patching costs are in 1975 dollars discounted at 6 percent.
Time is measured in years after completion of construction in fall of 1975.
Added user costs during patching are not considered.

tenance experiment, it can be argued that the results of the experiment itself have been influenced by the initial condition of the pavement. In other words, some of the techniques that were investigated in this experiment that did not prove successful might have shown better performance if the pavement had not already deteriorated to the extent that it had prior to establishment of the experiment. Doubtless, if the pavement had been only slightly in distress, some of the techniques investigated could have acted as preventive maintenance, and the results may have been somewhat different.

In the performance summary that follows, the methods are given in the order of most effective to least effective in preventing further deterioration of this specific pavement regardless of cost. The reader is referred to Table 4 in which the cost-effectiveness of each of the methods is presented.

1. Asphalt overlay: The asphalt overlay sections have performed very well and no correlation has been found between the performance of the overlay and its thickness. No reflection cracking has been noted other than on the feather sections, in which there is a transition from the overlay to a pavement that has no overlay, and at two isolated locations at which transverse cracks have been seen over known concrete patches.

2. Underseal and overlay: These sections have performed very well and no additional failures have occurred.

3. Overlay and subdrain: Again, the pavements that had an overlay have shown excellent performance, and the data do not suggest that the installation of subdrains along with the overlay has aided substantially in the performance of the pavement.

4. Underseal: Performance of the underseal sections has been quite satisfactory. It is to be recalled that undersealing was actually done only at locations at which the deflection was greater than 2.3×10^{-2} mm as measured by the Dynaflect. No failures have occurred where undersealing was done, although three have occurred in the underseal sections at locations other than those that were undersealed. The undersealing was done at a cost of $1020/100 m, and the data suggest that this is a cost-effective method of preventive maintenance.

5. Bituminous patch: The bituminous patches have shown no distress, and no failures have been found associated with the patches. Some settlement has occurred and some roughness is noted in riding over the patches.

6. Subdrain: Numerous failures have occurred on the sections in which subdrains were installed. The subdrains are known to be removing water from under the pavement as shown by a decrease in pumping and by visual inspection of the drain outlet. However, the numerous failures in these sections have suggested that additional work could well be done to determine the best means of draining pavements of this type.

7. Concrete shoulder: The concrete shoulders have not performed well, and deflection measurements suggest that continued distress might occur. One of the test sections included a keyway at the pavement edge, whereas the others did not. The keyed section has performed very well, but the unkeyed sections have not.

In the decision-making process it is necessary to evaluate performance of pavements in light of the economics of the problem. For this case, the primary question asked was what the most effective means of maintaining CRCP was.

For this experiment, overlaying by using asphalt concrete completely stopped further deterioration of the pavement. However, the average cost of patching during the time span of this test was $2483/100 m ($757/100 ft) for the control sections, whereas the initial construction costs of overlay were higher in all cases--$4351/100 m ($1326/100 ft) for the 51-mm

(2-in) overlay. Hence it can be concluded that, solely from the standpoint of cost, overlay is not cost-effective. Further, neglecting inconvenience and safety during patching operations, a cost-effective means would be to merely let breakups occur and to patch them as needed.

Undersealing also halted further deterioration and was done at a cost of $1020/100 m, which was less than the cost of maintaining the control sections. Hence it was concluded that this was the most cost-effective means of maintaining this pavement. The primary mode of distress for this pavement was pumping caused by high deflections and an impervious subbase. Undersealing effectively corrected this defect.

For this analysis it was assumed that all breakups would be repaired during the next summer if the failure had occurred during the winter or spring or during the same summer if the failure had occurred during the summer. It is known that all the breakups were not patched by using concrete. In some cases, temporary measures were taken, and the breakup still exists on the test pavement. Hence the economic analysis reported here is conservative.

As in any engineering project, the decision relative to type of maintenance to be applied must consider all factors and not merely cost. For this particular pavement the question of the desirability of expending large sums of money for complete removal of the possibility of further distress is moot.

Perhaps the primary conclusion to be reached from studies of this type is that, to be effective from the point of view of both performance and cost, maintenance procedures must be evaluated in the light of the defects that are to be corrected and planned accordingly. It is axiomatic that in an energy-constrained environment, design and maintenance should be carefully evaluated and based on sound engineering principles.

REFERENCES

1. A. Faiz and E.J. Yoder. Factors Influencing the Performance of Continuously Reinforced Concrete Pavements. TRB, Transportation Research Record 485, 1974, pp. 1-13.
2. E.J. Yoder. Primary Factors Influencing Performance of Continuously Reinforced Concrete Pavements in Indiana. Joint Highway Research Project, Purdue Univ., West Lafayette, IN, June 1974.
3. A. Faiz and E.J. Yoder. Evaluation of Parameters Significantly Influencing the Performance of Continuously Reinforced Concrete Pavements. In Roadway and Airport Pavements, American Concrete Institute, Detroit, MI, ACI Publ. SP-51, 1975.
4. A. Faiz. Evaluation of Continuously Reinforced Concrete Pavements in Indiana. Joint Highway Research Project Advisory Board, Purdue Univ., West Lafayette, IN, Final Rept., Oct. 1975.

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Patching Jointed Concrete Pavements

K.H. McGHEE

The experiences of the Virginia Department of Highways and Transportation in the repair of jointed concrete pavements over the past 10 years are summarized. Persons involved in pavement repair are cautioned to give careful consideration to pavement geometrics and dynamics. Also emphasized is the need for proper consolidation, adequate quality-control procedures, and care in the selection of repair materials. The conclusion is that serviceable repairs to concrete pavements can be achieved if these factors are given full consideration.

In the repair of jointed concrete pavements, the choice of the procedure to be used must include a consideration of how the pavement responds to its environment. The two major components of this environment are traffic loadings and climatic conditions.

The response of the pavement to traffic loadings is within the purview of the design activity and will not be considered at length here. It is evident, however, that when a pavement no longer provides the service for which it was designed, the selected maintenance procedure should restore it to the original level of service. For example, when pavement support conditions change so that wheel loads become more destructive, the maintenance engineer must attempt to restore the original support conditions through undersealing, slab-jacking, and improving the drainage.

The response of the pavement to climatic conditions also is treated in the design analysis, in which curling, warping, and joint movements are considered. Maintenance personnel, however, generally seem to be much less aware than designers are that the pavement is more or less in constant motion and that service conditions and repair techniques must make allowances for this motion. As a result of this lack of awareness, one often sees repairs that are incompatible with service conditions, e.g., unjointed patches on jointed pavements.

Instances in which pavement movement of one type or the other must be accommodated will be pointed out in succeeding sections of this paper. For now, it is sufficient to emphasize that pavements are dynamic and that, whether one is dealing with design, construction, or maintenance, those dynamics must be considered. From the standpoint of pavement repairs, one can state simply that the repair must be geometrically compatible with the dynamic pavement system.

DISTRESS MECHANISMS AND MANIFESTATIONS

Joint failures of various types probably lead to the necessity for much more maintenance than do problems with the slabs. Numerous factors can contribute to joint failures, and the failures can be manifested in several ways. Because the repair procedure to be used is often dependent on the nature of the distress, some of the mechanisms and manifestations of joint failures are discussed below.

Metal Inserts

Tubular metal inserts were used to form transverse contraction joints in many of the concrete pavements constructed in the early 1960s. Placed in the fresh concrete, these devices provided a weakened vertical plane at which contraction joints would occur. After the concrete had hardened, the inserts were crimped downward and a poured sealant was introduced.

A survey of these metal joint inserts in 1968 by the Virginia Highway and Transportation Research Council (VHTRC) found their condition to be fair to rusted and badly rusted after four to six years of service. Because the inserts were anchored by small flanges that extended into opposing faces of concrete at the contraction joints, they created weakened planes that were later aggravated by corrosion products. Weakened planes of this type, not initially evident on the surface, propagate under repeated wheel loadings and eventually intercept the pavement surface at 6-12 in from the joint face (Figure 1).

Figure 1. Semicircular spall caused by corrosion of insert.

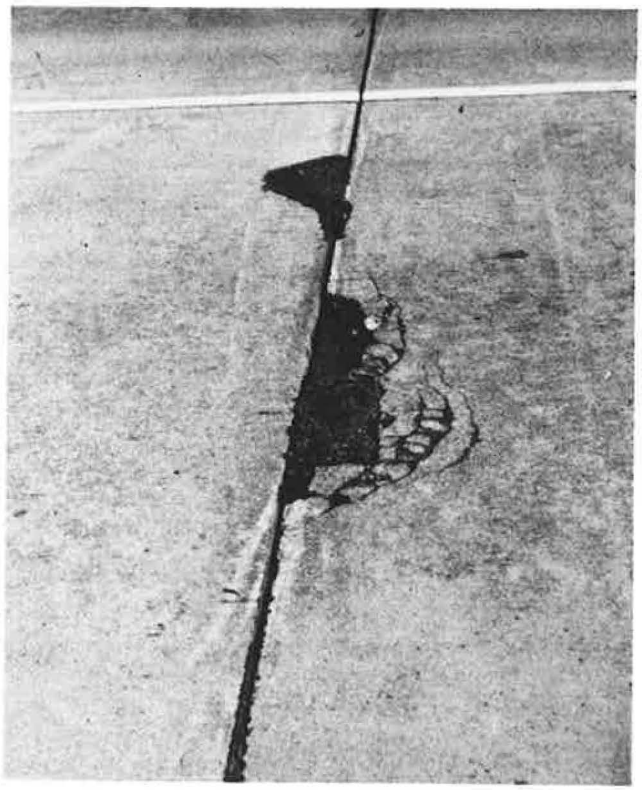


Figure 2. Conditions prior to blowup.

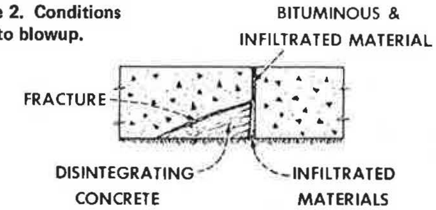
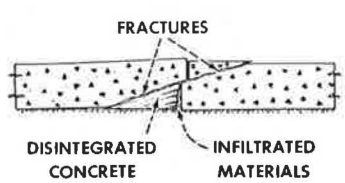


Figure 3. Blowup in which characteristic fractures and pavement uplift are shown.



Subbase and Shoulder Materials

Most surface water can be expected to find its way to the underside of pavement slabs through the edge joint, which is difficult to seal. Recent studies have shown that 70 percent of the surface water that flows over a crack that is only 1/32 in wide will enter the crack and proceed to the lower pavement layers (1). Depending on the permeability of the subbase and shoulder materials, this water may remain for significant periods of time after rainfall has ceased.

As wheel loads cross the joint, pressures on trapped water are reduced under the approach slab and increased under the leave slab. A net deposition of soil particles is caused under the approach slab by this pumping action. The accompanying faulting produces a less satisfactory pavement from the standpoint of rideability but, more important, it causes increased stress at the joint. Pumping may result in a migration of fine, incompressible material into the contraction joints from their outer edge and bottom portions. Unchecked, the reservoir can extend toward the pavement centerline while progressively undermining the structure and infiltrating the remaining bottom portion of the transverse joint. Early indications of pumping are soil staining of the shoulders and, later, faulting of transverse joints.

Joint Sealants

The infiltration of incompressible materials, water, and deleterious chemical agents such as deicers into the longitudinal and transverse joints of a concrete pavement promotes conditions that hasten the need for repair, reduce the riding quality, and generally shorten the useful life of the pavement.

Incompressible materials restrict the free movement of the joint (expansion and contraction), which results from variations in temperature and moisture, and give rise to localized high compressive stresses that may cause blowup of the pavement.

A widely accepted theory of the development of blowups was given by Giffin in 1943 (2). The first stage of failure occurs when compressive stresses become high enough to fracture the concrete below the surface but no distress is evident on the pavement surface. This condition is shown schematically in Figure 2 [adapted from Giffin (2)], in which it may be seen that disintegrating concrete at the lower joint corner forms an inclined plane with the undamaged concrete above. As the compressive stresses increase, the situation changes to that indicated in Figure 3 [adapted from Giffin (2)], in which one slab has moved up the inclined plane with sufficient force to shear the corner of the adjacent slab. In this manner, the observed blowup characteristics of one slab that is overriding the other are generally explained.

The succeeding sections of this paper deal with the repair procedures used in Virginia. All procedures used fall into two broad classifications--partial depth and full depth. Procedures in both classifications have evolved over approximately the past 10 years, primarily as the result of trial and error but with a good deal of attention to engineering requirements.

PARTIAL-DEPTH REPAIRS

Successful partial-depth repairs depend to a great extent on the type of material used in patching.

Many concretes can be used in partial-depth, cast-in-place pavement repairs on the basis of satisfactory laboratory evaluations. For the successful employment of any concrete in the field, however, there are numerous procedural factors that, if ignored, can adversely affect the service life of a patch. These procedural factors are encountered during the preparation of the pavement for patching and during the installation of the repair concrete. The modification of repair procedures to eliminate the destructive effects of these factors has resulted in durable patches that have service lives that currently exceed five years and have a reasonable expectation of a number of years of additional service.

Other factors that affect the service life of pavement patches are traffic intensity and weather conditions. Requirements dictated by these factors can normally be satisfied by selecting concretes on the basis of satisfactory performance in laboratory evaluations of strength and durability.

Location of Unsound Concrete

A visual survey can provide a reasonable initial estimate of the extent of needed repairs on a roadway. However, the actual extent will always be greater, since a plane of rupture extends beyond the visually identified limits of joint spalls and in the early stage of spall formation the weakened plane may exist even though there are no visible indications.

The technique that has been used to identify the total area affected by spalls relies on hammer soundings. A ball peen hammer is tapped on the pavement surface; areas that issue a clear ringing sound are judged to be serviceable, whereas those that emit a dull sound are considered to be weakened by ruptures and are marked for removal.

Joint spalls usually affect an area that extends 6-12 in from the transverse joint; however, experience has shown that it is advisable to remove the concrete for a minimum of 12-18 in from the joint. The areas removed along the joint are typically 1-3 ft in width; however, the area may extend along the full width of the lane.

Sawing

A saw cut that has a minimum depth of 1 in should be made along the perimeter of the pavement area to be removed. The area removed should be rectangular to provide a vertical face against which to cast the repair concrete, to give an aesthetic appearance to the finished patch, and to prevent feather edging and resultant raveling of the repair concrete along the perimeter.

Removal

The partial-depth removal of the designated pavement areas can be accomplished quickly by using jackhammers. The removal can be started by using an 80-lb hammer but should be finished by using a 20-lb hammer to allow removal of all loose concrete in the area and prevent damage to the underlying and surrounding concrete.

The depth to which the concrete is removed typically varies from 1 to 4 in and averages approximately 3 in over the majority of the patch areas. Removal of the concrete by this technique provides a very irregular surface that is ideal for achieving mechanical interlock between the cast-in-place repair concrete and the existing pavement.

The occurrence of spalls in areas adjacent to partial-depth patches has been practically eliminated since inspectors and maintenance supervisors have implemented the above procedures for locating, sawing, and removing unsound concrete.

Occasionally in partial-depth patching, the full depth of the slab may be removed. This is particularly true for spalls that occur at the corner of the slab adjacent to the shoulder material (3). However, if full-depth removal is consistently needed to eliminate unsound concrete in this and other areas of the transverse joint, a full-depth repair technique is probably needed for the roadway (4).

Patch Installation

Regardless of the type of cast-in-place concrete to be used in a patch, the installation must be made in similar logical steps. In the following paragraphs these steps are discussed and, when appropriate, examples are cited to demonstrate the problems encountered if the steps are not performed properly.

Concrete Production

The volume of concrete required for partial-depth patches may vary up to several cubic feet. The use of ready-mix trucks for the supply of fresh concrete is therefore not desirable, since the maximum allowable mixing times for specified temperature ranges would be exceeded, even with normal type 2 cement concrete, after only a small portion of the batch was used.

An increasingly popular method of preparing concrete for partial-depth repair operations is to use a continuous mixer in which the materials are continously batched by volume. The auger feed on such a mixer can be stopped when the correct amount of concrete has been discharged, and a minimum of concrete is wasted. The guidance provided by ASTM C685 is helpful when a continuous mixer is used. This equipment will not always be available, however, and the size of the job may not be sufficient to effectively utilize its capacity.

In such instances, small portable mixers of about 2-ft^3 capacity have been effectively used, particularly with prepackaged patching materials in which water is the only ingredient to be added on the job (5).

Forming

Prior to the placement of repair concrete in a prepared area, it is important that all needed forms be in place. The three locations that require attention are the centerline and transverse joints and the shoulder.

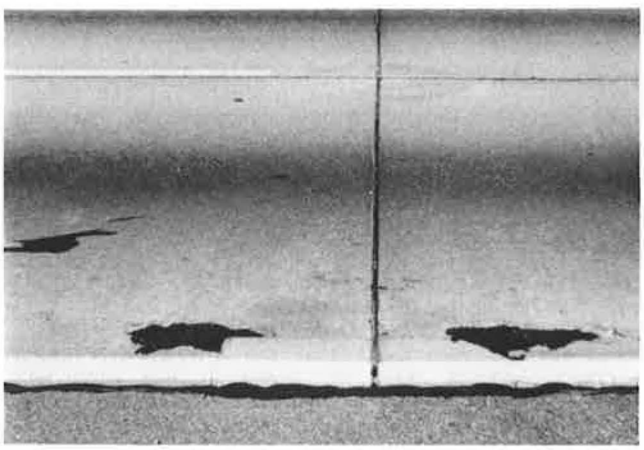
The centerline joints between pavement lanes are provided to prevent warping stresses. The patch shown in Figure 4 was cast directly against the adjacent slab at the centerline joint, and raveling and minor spalling resulted from movement at this location due to normal warping stresses. This problem can be overcome by inserting a polyethylene strip along the centerline joint prior to casting the repair concrete.

A large number of partial-depth patch failures have resulted from using no forming or inadequate forming at the transverse joint. Figure 5 shows two patches cast together across the transverse joint. Before the transverse joint could be sawed and sealed, the slab movements had disrupted the patches.

Studies have shown that many patch failures have resulted from failure to form the total depth of the

Figure 4. Raveling and minor spalling of partial-depth patch cast without forming at pavement centerline.

Figure 5. Spalling of adjacent pavement that resulted from compressive stresses transmitted through patches.



patches at the transverse joints. The lower portions of the partial-depth patches were therefore in contact with the adjacent slabs and the patches were subjected to the destructive effect of hydrothermal slab movements (Figure 5).

If shoulder material adjacent to the pavement is removed by erosion or disturbance during removal of the concrete, a vertical form should be placed parallel to the shoulder to prevent the repair concrete from filling the void and forming a key into the shoulder. Otherwise, the normal hydrothermal movements of the slab can cause the patch to be disrupted by resistance from the stationary shoulder.

Consolidation and Bonding

In order for repair concrete to become an integral portion of a slab, it must fill the space created by the removal of deteriorated concrete, and it must develop an adequate bond to the existing hardened concrete. Both requirements are related to the consolidation of the fresh concrete, since adequate consolidation should produce a patch that has maximum density and should provide the greatest amount of contact between the fresh and existing concrete for development of bond and for mechanical interlocking.

Consolidation of concrete is the process of removing randomly occurring volumes of entrapped air. The remaining mass, which contains uniformly distributed entrained air volumes, approaches a theoretical maximum density that can be determined by the volumetric proportions of the constituents.

Methods employed in Virginia to consolidate concretes in partial-depth patches have been manual rodding and tamping and the use of vibrating screeds or internal vibrators. Each of these methods has advantages and disadvantages, which will not be discussed here except to state that, of these methods, internal vibrators have given the most uniform and best-quality consolidation. Also, since standard models of internal vibrators seem to be more readily available among contractors than are the small versions of vibrating screeds, the trend in Virginia has been to encourage the proper use of internal vibrators for the consolidation of concretes in pavement patches.

Figure 6 shows a core drilled from the central portion of a patch that emitted a hollow sound when struck with a hammer. The large void in the zone between the repair concrete and the underlying old concrete is clearly indicative of inadequate consolidation. This patch and others that emitted a similar hollow sound when struck failed by cracking and spalling after being in service for less than one year. These patches were installed by using the manual rodding and tamping method of consolidation.

It has been demonstrated that the elimination of visible voids at the interface between the repair concrete and the existing pavement can readily and consistently be accomplished by using an internal vibrator. An internal vibrator that has a head diameter of 1 in has been successfully used for consolidating repair concretes. After a patch area has been overfilled with enough fresh concrete to allow for a reduction in volume during consolidation, the vibrator is held at a small angle (15-30°) from the horizontal and moved through the concrete in such a way that the full patch is vibrated. The speed at which the vibrator is moved is determined by observing the surface of the concrete. Adequate consolidation is indicated when there is no further reduction in the concrete depth, the emergence of air bubbles ceases, and a smooth layer of mortar occurs on the surface. A typical core section from a patch that received this treatment is shown in Figure 7, in which it may be observed that the irregularities along the interface with the existing concrete are completely filled and that the repair concrete is not segregated.

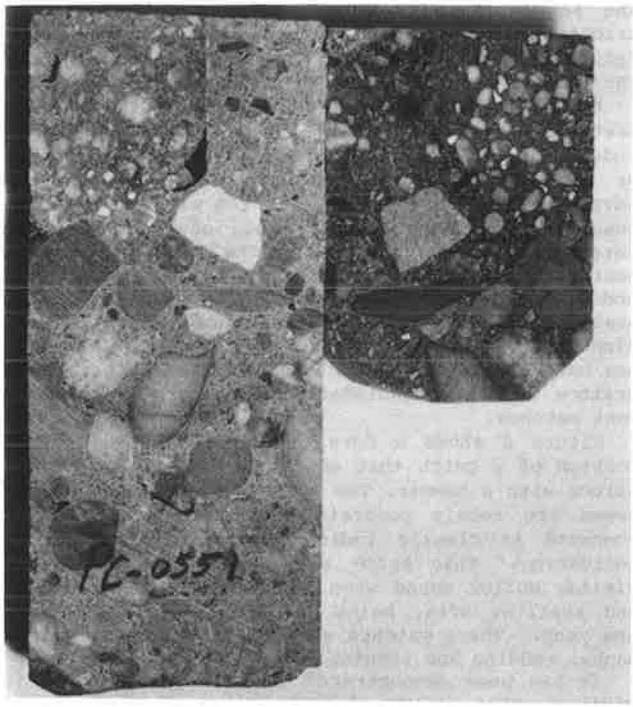
Shear-bond tests of four systems used in Virginia yielded the results given in Figure 8. The resulting strength may be interpreted on the basis of published data that show that strengths of 200 psi are adequate for good performance. Note that all four systems had developed excellent bond by the age of 25 days; however, only cement slurry had adequate bond at 24 h of age.

Screeding and Finishing

Cast-in-place patches can be manually screeded to the proper grade by using a stiff board. The small size of the patches normally allows space for the board to rest on the adjacent surface of the slab being repaired so that the screed can advance in a direction perpendicular to the transverse joint. If large sections of the transverse joint are being repaired or if repairs are being made on both sides of the joint, the patch can be screeded across the joint by using appropriate precautions to avoid disturbing the form inserted in the transverse joint.

Hand troweling of the patch surface removes any

Figure 6. Incomplete consolidation in vertical section of core (core dimensions, 4x9 in).



remaining irregularities and, most importantly, provides a good finish for the edge of the patch adjacent to the transverse joint into which a sealant must be installed after hardening of the concrete. A textured finish can easily be applied to the patches by using a broom that has stiff bristles so that the patches will have a texture similar to that of the surrounding concrete.

Curing

The application of curing materials to the surface of patches is important, because moisture losses can occur quickly from the relatively large surface of these shallow placements. Moist burlap and polyethylene must be removed when the roadway is opened to traffic, and the sudden drying can cause shrinkage cracks. The most effective cure can be provided under hot-weather conditions by using a white-pigmented curing compund, since that reflects radiant heat, allows the heat of hydration to escape, and can provide curing protection for several days until it has been worn away by traffic. When patching is done under cold-weather conditions, the loss of heat can be reduced by the addition of insulating materials such as blankets, straw, sand, or burlap or heat may be supplied from an external source if conditions are severe.

Performance of Partial-Depth Repairs

Studies on one heavily traveled toll road on which several thousand partial-depth repairs had been installed by using the procedures described above showed that more than 80 percent of the repairs were in excellent condition five years after installation. In this instance, five rapid-curing materials (including $CaCl_2$-accelerated conventional concrete) were used successfully, and lanes were reopened after about 6 h of curing. However, inspection and quality control were very stringent on this project, so that one can conclude that under nearly ideal conditions good repairs can be realized by using a variety of materials.

On most projects, quality control and success have been less dramatic, and up to 50 percent of the repairs have failed in about two years. Most failures have been related to (a) poor quality control of concrete, (b) poor consolidation, (c) failure to remove all unsound concrete, and (d) geometric incompatibility between the repair and surrounding pavement features.

FULL-DEPTH REPAIRS

Full-depth repair of portland cement concrete (PCC) pavement as addressed in this paper refers to one of two procedures:

1. Full-depth restoration, in which the existing pavement joints (which includes load-transfer devices) are restored in their original positions, and
2. Full-depth replacement, in which no effort is made to restore the original transverse joints, two transverse joints are provided, and load transfers may or may not be restored. In this case, both cast-in-place and precast repairs have been used effectively. Only cast-in-place methods will be discussed.

Cast-in-Place Restoration

Virginia pavements that have a small number of joint failures have been repaired by restoring the transverse joints along with the placement of cast-in-place concrete.

Concrete Removal

Transverse saw cuts approximately 1.5 in deep are made along the two limits of the repair parallel to the transverse joint. Jackhammers are used to remove the concrete to the full pavement depth. This procedure allows ties to be made to the existing reinforcing fabric located at the 2-in depth, and it further produces irregular faces against which the repair material can be cast to provide load transfer through aggregate interlock. About 6 in of the protruding steel is left in place to provide a tie-in with the repair concrete.

Repair Concrete

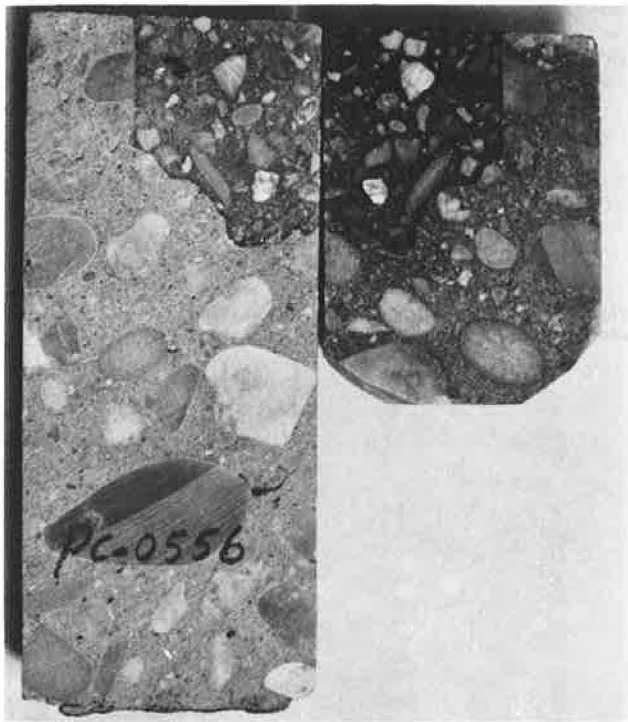
Both high-alumina cement (HAC) concrete and high-cement-factor type 3 PCC have been used successfully in these repairs. However, experience has shown that HAC concrete does not perform as well as had been hoped because of the destructive effects of the very high early heat evolution and subsequent cooling. Conventional concrete that has high early strength is now the preferred mixture.

Installation

The precut steel-fabric reinforcement and prefabricated dowel assembly used in this type of repair are shown in Figure 9. The reinforcement is tied to the existing reinforcement that protrudes from the slab. The dowel assembly is fixed to the subbase and includes a bituminous-impregnated felt strip to form the contraction joint through the full slab depth.

Placement of the cast-in-place concrete includes planned considerations for consolidation, finishing, and curing. Finished repairs are ready for opening to traffic in 6-8 h. The darker-colored repair concrete can be observed in Figure 10. Most of the repairs do not exceed 8 ft in length and present an

Figure 7. Typical core section from patch in which internal vibration was used.



acceptable appearance from a moving vehicle.

Performance

Thirteen initial repairs of the type shown in Figure 10 have performed satisfactorily for more than four years. The functioning of the restored transverse joints and the proper tie-in at the construction joints between the repair concrete and existing pavement slabs were studied. Movements in several of the restored contraction joints and construction joints were instrumented by using gage plugs and monitored to determine whether they were functioning as they should. Deviations from expected behavior would have served as an early warning that the repair procedure needed modification. Movements of the restored contraction joints average approximately 10 times those in the construction joints at each of the repairs. The movement of the construction joints compared well with that of original construction joints on similar pavements. The tightness of the construction joints can be attributed to the careful tie-in of reinforcement mesh between the old pavement and the repair concrete. The vertical section of a core drilled down through the construction joint revealed the overlapped transverse bars of the mesh as seen in Figure 11. The excellent potential for load transfer can be seen in the mechanical interlock between concretes in that figure. The favorable irregular face in the old pavement was created by the concrete-removal technique described earlier, and good bonding was achieved at the construction-joint interface by adequate consolidation following the application of a bonding slurry.

Although, as indicated above, the cast-in-place restoration has performed well, it has fallen into disfavor because of its expense, which is primarily associated with restoring the dowel assembly. For this reason, cast-in-place replacement, as discussed below, is the most frequently used full-depth repair procedure. Bid experience has shown that the former

Figure 8. Shear-bond test results for four bonding systems.

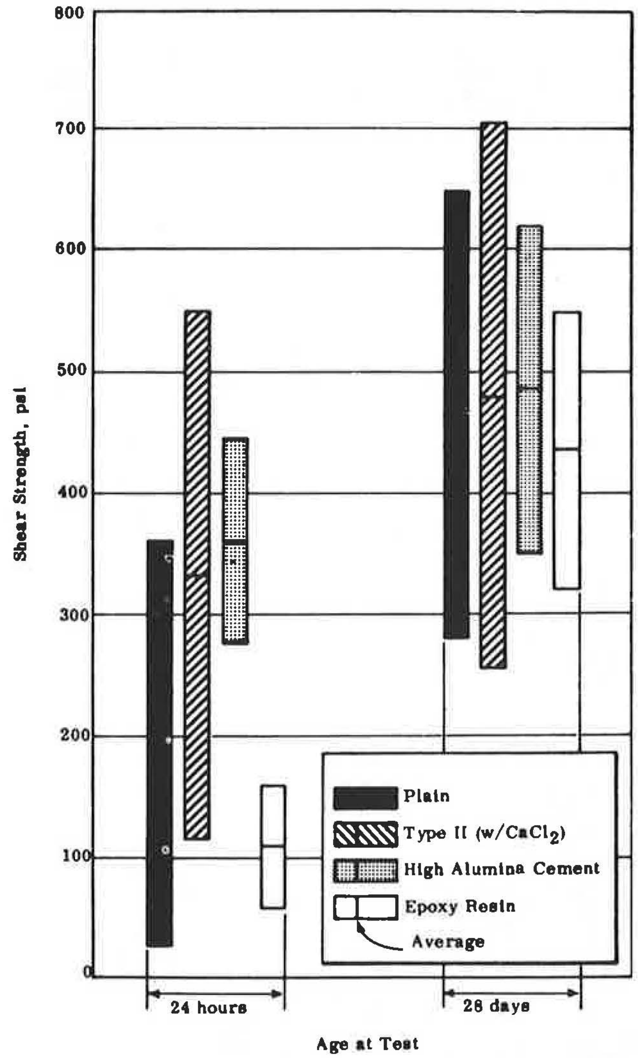
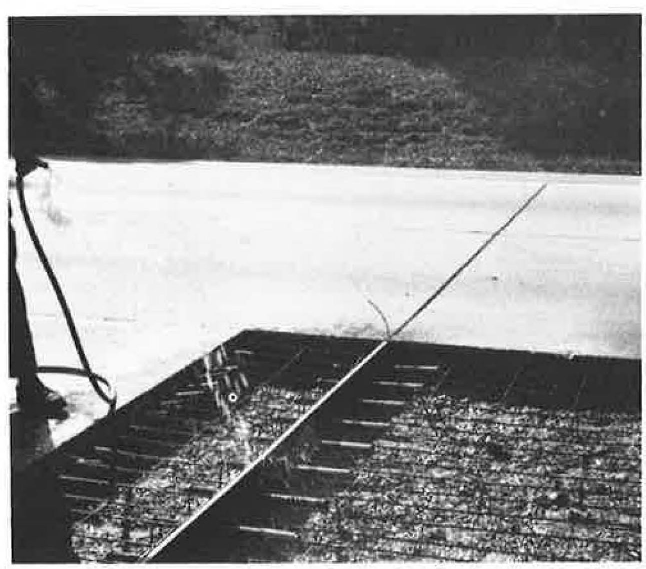


Figure 9. Precut steel-fabric reinforcement and prefabricated dowel assembly in place.



is about 50 percent more costly than the latter.

Cast-in-Place Replacement

The cast-in-place replacement procedure used for most full-depth repairs at transverse PCC pavement joints is depicted in Figure 12. This approach has become commonly known as the inverted-T method.

Load Transfer

It can be observed in Figure 12 that the traffic loadings for this repair situation are transmitted directly from the existing pavement slabs on either end of the repair area to the repair concrete through the inclusion of 6-in undercuts in the subbase material. The volume of the undercuts is filled along with the volume created by removal of the deteriorated concrete and approximately 6 in of the subbase material. The resultant monolithic mass of repair concrete serves to carry traffic loadings directly on its surface and from the adjacent slabs.

In some instances, in which cement-stabilized subbases underlie the pavement to be repaired, no undercut is feasible and repairs have been placed at full depth without the reestablishment of load transfer. In these cases, a repair that is a minimum of 6 ft long is provided on the assumption that the more-massive repair will be less subject to displacement under wheel loads. Surprisingly, the problems associated with this type of repair are those of upheaval due to pumping action of the adjacent slabs rather than those of depression under load. A contract now being developed will use a nonvertical saw cut on either side to provide approximately a 1-in undercut of adjoining slabs so those slabs will tend to rest on the repair concrete.

Concrete Removal

The 2-ft minimum length for concrete removal on either side of the existing transverse joint indi-

Figure 10. Hardened cast-in-place restoration made by using HAC concrete.

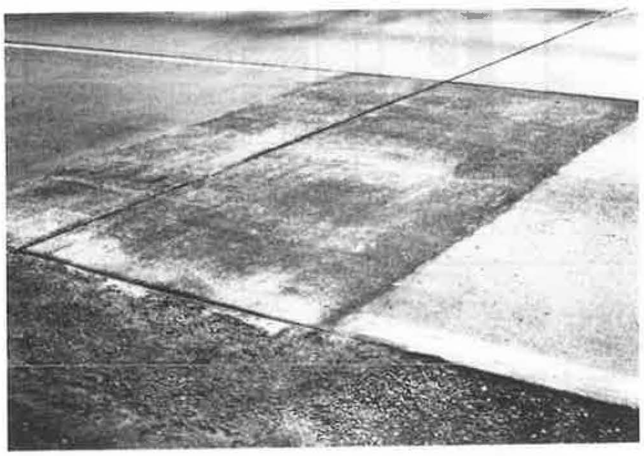


Figure 11. Core from construction joint between HAC repair concrete (left) and old pavement (right).

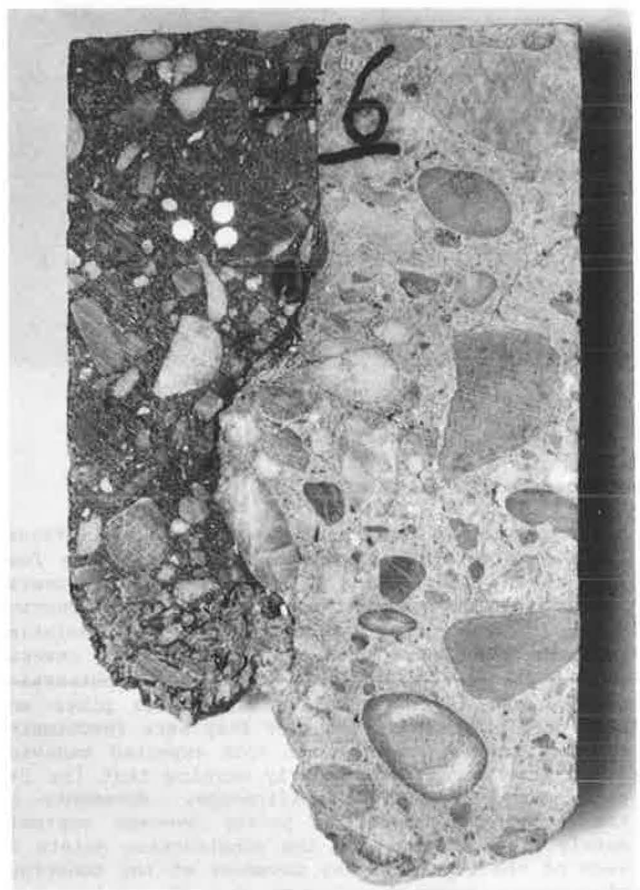
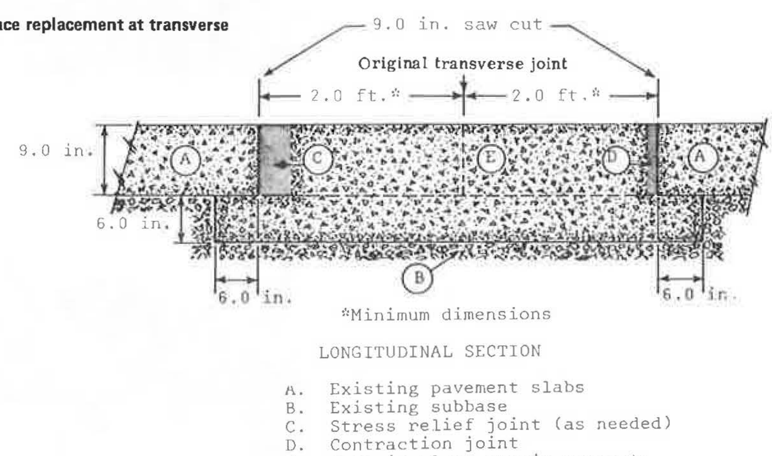


Figure 12. Typical details of cast-in-place replacement at transverse joint.



cated in Figure 12 is based on field experience. It has been observed that subsurface fractures of the joint face that may not be visually detected from the pavement surface can extend more than 1 ft into the slab before they intersect the subbase interface (3). It is therefore appropriate to designate minimum limits for pavement removal for this and other procedures for full-depth pavement repairs.

The pavement-removal technique described was also used in this procedure. Following removal of the subbase material, no further preparation of the area was necessary since the cast-in-place concrete could conform to irregularities in the subbase.

Repair Concrete and Joints

The concrete mixtures described earlier for cast-in-place restoration are used in this method of repair. Transverse joints are treated as if they were contraction joints on new construction. Essentially, this means that they are resawed and sealed by using a preformed seal or a good-quality poured sealant that has the appropriate shape factor.

Performance

Repairs of this type number more than 1000 and have been in service for up to five years. In some contracts, most are in excellent shape; in others, up to 50 percent have failed, sometimes in less than six months. Most failures have been attributable to one of the following causes:

1. Use of HAC concrete, which becomes excessively hot and results in shrinkage cracks that later spall;
2. Inadequate consolidation in the undercut area, which triggers early failure of the load transfer so that adjoining slabs become depressed and sometimes fail;
3. Failure to underseal the adjoining pavement so that it fails outside the limits of the repair; and
4. Poor quality control of concrete, particularly with mobile mixers in which water control is by judgment or slump only and is highly variable.

CONCLUSION

In conclusion, it may be stated that Virginia experience has shown that durable repairs to jointed concrete pavements can be achieved if the engineering requirements of the repair and the engineering characteristics of repair materials are properly accommodated in the chosen repair procedure. Accordingly, most failures have been seen to occur when the above factors did not receive due consideration or when quality control of repair activities was lacking. Such failures are enormously expensive because of the costs for traffic control associated with repairs. For this reason, it behooves the highway engineer to ensure that repairs are carefully planned and that the best inspection and quality control possible be provided on repair activities.

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REFERENCES

1. H.R. Cedergren, J.A. Arman, and K.H. O'Brien. Development of Guidelines for the Design of Subsurface Drainage Systems for Highway Structural Sections. Office of Research, FHWA, 1973.
2. H.W. Giffin. Transverse Joints in the Design of Heavy Duty Concrete Pavements. Proc., HRB, Vol. 23, 1943, pp. 56-74.
3. S.S. Tyson and K.H. McGhee. Deterioration of Jointed Portland Cement Concrete Pavements. Virginia Highway and Transportation Research Council, Charlottesville, Nov. 1975.
4. S.S. Tyson. Full-Depth Repair of Jointed PCC Pavements: Cast-in-Place and Precast Procedures. Virginia Highway and Transportation Research Council, Charlottesville, March 1976.
5. S.S. Tyson. Partial-Depth Repair of Jointed PCC Pavements: Cast-in-Place and Precast Procedures. Virginia Highway and Transportation Research Council, Charlottesville, Jan. 1977.

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Highway Pavement Repairs by Using Polymer Concrete

ALVIN H. MEYER, B. FRANK McCULLOUGH, AND DAVID W. FOWLER

As traffic, particularly truck traffic, has increased on the primary highway system, the need for rapid repair methods has increased. Polymer concrete (PC) has been used effectively for rapid repair of portland cement concrete pavements, both jointed and continuously reinforced. Basic formulations for PC are presented and both user-formulated and prepackaged systems are described. Methodology for the repair of cracks, joints, spalls, and punchouts is illustrated. The results of several PC repairs are presented. Deflection measurements that illustrate the restoration of structural integrity, which means a prolonged pavement life, are given.

Many high-volume highways in the United States were constructed by using portland cement concrete (PCC) as the pavement surface. Now that traffic and allowable axle loads have increased, many of these facilities are approaching the limits of their design life, which usually means increased maintenance. The need for rapid, permanent types of repairs has led to the development of polymer concrete (PC) as a repair material.

The use of PC is not new to highway repairs. It

Figure 1. Loading arrangement for testing flexural bond strength of PC to PCC.

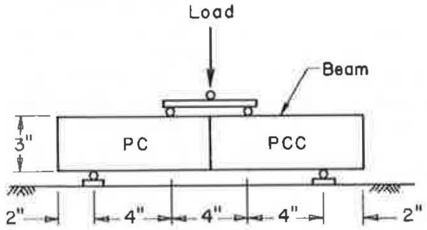
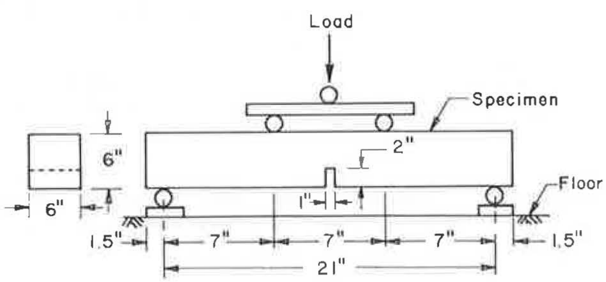


Figure 2. Crack chaser in operation.

Figure 3. Loading arrangement for testing the concrete beams.



has been used for several years as a patching material for highways and bridges. We attempt to summarize the types of repairs that have been made and some of the research that has led to the use of PC for those repairs.

All the PC described in this paper has a base of methylmethacrylate (MMA) for reasons discussed in the next section. Two methods of producing PC are used: prepackaged, which includes all the commercially available systems in which the user mixes the materials in the proportions supplied; and user-formulated, in which the polymer used is formulated by the user to satisfy the conditions of placement.

NOMENCLATURE AND DEFINITIONS

Synthetic polymers constitute a broad class of materials that includes polyesters, nylons, and epoxies that have been chemically transformed into high-molecular-weight chemical structures from simpler units called monomers. Several types of chemical reactions can be used to cause polymerization of monomers into polymers. Each class requires carefully selected formulations of monomers, additives, and reaction conditions to achieve the desired result. A number of these polymer systems have been successfully employed as substitutes for PCC for binding aggregates together to produce PC. Acrylic polymers formed primarily from MMA offer distinct advantages for PC systems for rapid repair of PCC pavements and bridges.

This application of PC places stringent requirements on both the process of polymerization and the properties of the resulting polymer. In addition, the interaction of the polymer with the selected aggregate system to form a good bond is another important requirement, which can be adversely affected by extraneous matter, such as water.

The most important process requirement is the rate of polymerization or the time for curing the monomer into a solid polymer. The resulting polymer must develop adequate mechanical properties to meet the requirements for the composite PC.

The main mechanical property of the polymer that is of interest is the ultimate strength, which is defined as the maximum stress that the material will sustain. For polymers, this characteristic is conventionally measured in tension because most polymer failures occur in this mode. For many purposes, ductility of the polymer is also a critical issue in its applications. For this work, measures of polymer ductility are primarily based on the magnitude of deformation at failure in tension tests. Many of the above-mentioned characteristics are basic to the selection of MMA as the base monomer; however, each characteristic can be significantly affected by the formulation employed in the formation of the polymer.

The remainder of this paper makes extensive use of simple abbreviations of the chemical names of the various ingredients in the formulation of polymers. These abbreviations and the standard chemical names are given below:

Abbreviation	Chemical Name
Monomers	
MMA	Methylmethacrylate
BA	Butylacrylate
EHMA	2-Ethylhexylmethacrylate
IDMA	Isodecylmethacrylate
AA	Acrylic acid
MAA	Methacrylic acid
HPMA	Hydroxypropylmethacrylate
Initiator agents	
BP (also BPO or BzP)	Benzoyl peroxide
DMPT (also DMT)	Dimethylparatoluidine
DMA	Dimethylaniline
Cross-linking agents	
TMPTMA	Trimethylolpropane trimethacrylate
TTEGDA	Tetraethylene glycol diacrylate
Plasticizers	
DOP	Dioctyl phthalate
TCP	Tricresyl phosphate

The polymerization of MMA proceeds by a free-radical process, which requires a source of free radicals. This may be accomplished by a number of methods; however, the most useful for these purposes is the addition of a chemical initiator.

Several types of initiators are known and employed, but peroxides are the most useful for PC applications since their decomposition into free radicals can be catalyzed (accelerated or promoted) by the addition of another suitable chemical. Thus, the initiator system is made up of a peroxide and a promoter, whose type and proportion have dramatic

Figure 4. Repair no. 6.

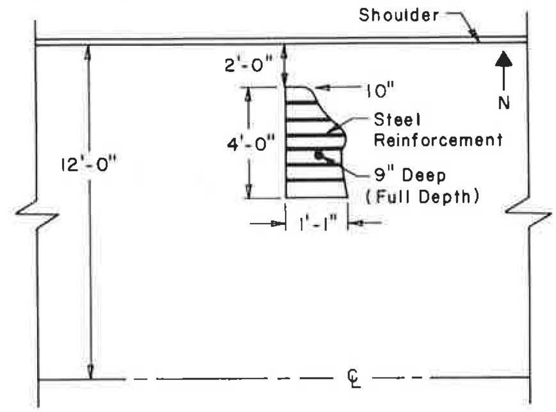


Figure 5. Repair no. 10.

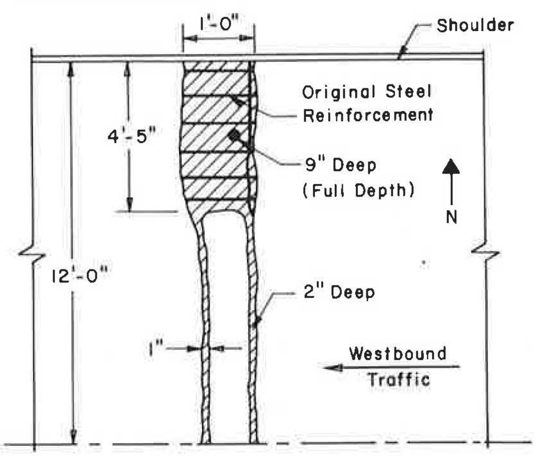


Figure 6. Joint repair no. 4.

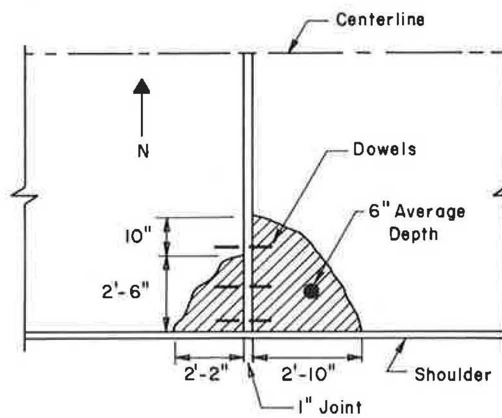
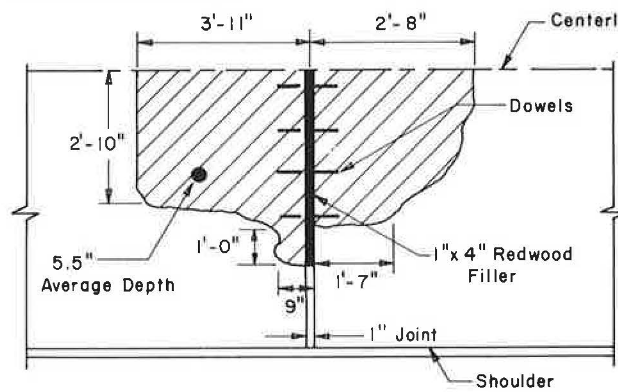


Figure 7. Joint repair no. 7.



effects on the rate of polymerization or curing and the resulting properties of the polymer. This work has employed BP and DMPT, respectively, as peroxide and promoter. Other choices could have been made or will be made in subsequent work; however, this system has proved sufficiently versatile and effective.

Polymer technology frequently uses mixtures of monomers to form copolymers since this provides an effective means of tailoring the polymerization process and the polymer to meet specific needs. The polymers mentioned thus far are linear in their molecular structure and may be likened to a train (a polymer chain) formed from boxcars, flatcars, etc. (monomers and comonomers).

Chemical monomers that possess multiple functionality can be added to introduce branching and subsequently cross-linking between polymer chains. These materials are known as cross-linking agents and the most typical one employed in PC formulations is TMPTMA. These materials dramatically alter the polymerization rate and polymer properties.

Sometimes nonreactive plasticizers are incorporated into polymer formulations to add ductility to the resulting material. Plasticizers are used in prepackaged PC formulations but have not been employed in the user-formulated formulations used here. A variety of other chemical ingredients may be added to the formulation as needed to alter workability or behavior.

These materials and concepts form an arsenal of tools for problem solving that can be employed in any PC application.

PHYSICAL PROPERTIES OF PC

The physical properties of prepackaged and user-formulated PC are very similar. Typical ranges of values are given below:

Physical Property	Range (psi)
Compressive strength	5000-10 000
Modulus of elasticity	2-4 x 10^6
Splitting tensile strength	800-1800
Flexural strength (modulus of rupture)	1500-2500

The flexural bond strength of PC to PCC is equal to the flexural strength of the PCC when the bond surface is clean and dry. To test this property, beams were prepared in which half the length was PCC (Figure 1). In all cases that used recommended procedures to develop bond, the failure occurred in the PCC. When the prepackaged systems are used, a primer coat is necessary to ensure proper bond strength.

The bond strength of PC to steel reinforcement is about three times that developed by PCC. In other words, if 12 bar diameters are required for a PCC application, the same bond strength can be generated with 4 bar diameters when PC is used.

CRACK REPAIR

In many PCC pavement failures, the formation of unwanted cracks precedes the failure. In some instances, if the cracks could be bonded together, the service life of the pavement could be extended.

Figure 8. Deflection between cracks for a sound pavement.

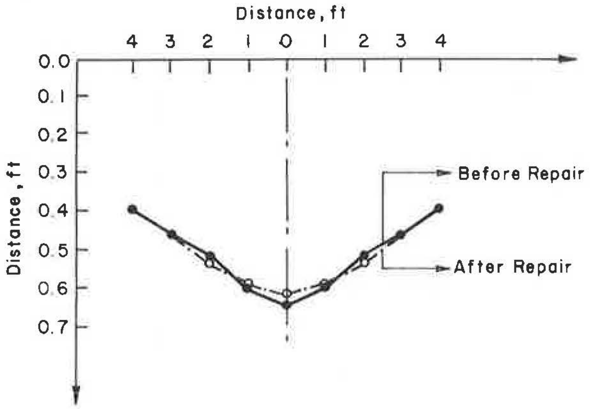
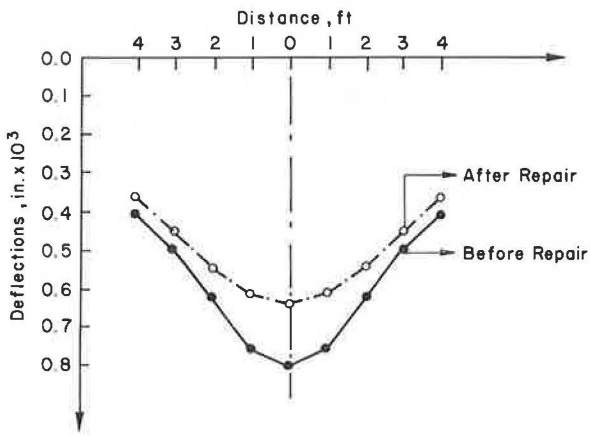


Figure 9. Deflections at upstream edge of repair no. 10.



A test was developed to simulate the crack formed in the concrete pavement by using a crack chaser. A crack chaser is a wheel-mounted piston-driven bit for following cracks in concrete and opening them up uniformly for remedial work. The crack chaser has a special tungsten-carbide-tipped, four-point cross face and side-cutting bits that have a standard diameter of 7/8 in. The standard bit drills holes about 2 in deep and 1 in wide. Figure 2 shows the crack chaser.

Five 6 x 24-in concrete beams were cast that had a groove 2 in deep and 1 in wide at the center. The beams were taken out of the forms after one day, and the sides of the groove were roughened by using a chisel to obtain a rough interface. The beams were moist-cured for one week and then dried in the oven at 250°F for three days.

The beams were broken in half with third-point loading [the groove was on the tension side (Figure 3)], and the loads were recorded. Next, two sheet-metal forms were fixed on the two sides of the groove with latex caulking material. After the caulking had hardened, monomer was poured into the groove to a depth of about 1/4 in to make sure that the crack was filled. The groove was then filled with aggregate and more monomer was added to fully saturate the aggregate. A mixture of 10 parts MMA powder to 90 parts sand was then sprinkled on top. After about 24 h, the sheet metal was removed. The beams were loaded in the same manner and orientation as in the first case.

In all cases, more than 50 percent structural integrity of the beam was restored. Thus, repairing the crack gave the beam more than 50 percent of the flexural strength of an uncracked beam. In most cases the gain was more than 80 percent.

Figure 10. Deflections at downstream edge of repair no. 10.

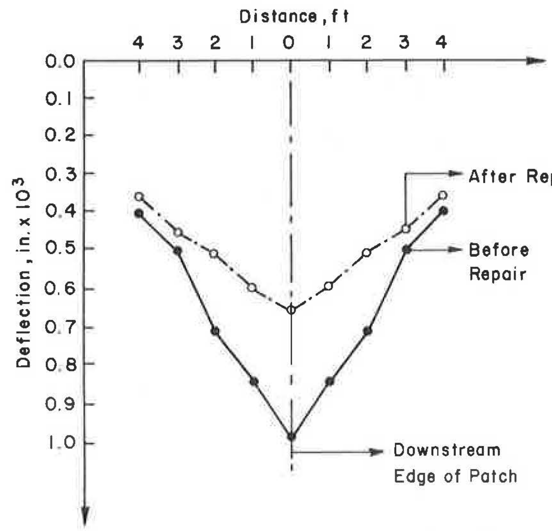
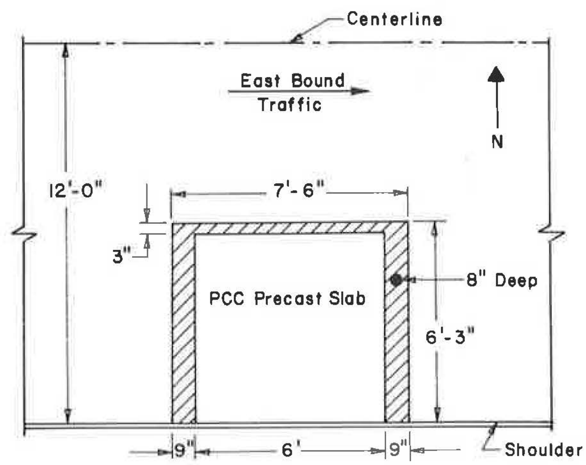


Figure 11. Installation for repair of punchout areas in CRCP.



Repair Procedures for User-Formulated System

Preparation of Repair Area

The repair area should be cleaned of deteriorated or delaminated concrete. The concrete surfaces should be dry and the surface should cool to about 120°F or less prior to making the repair. Joints through which the monomer could leak should be sealed by using caulking compound. If the repair is full depth, the base material usually has sufficient moisture to prevent excessive leakage into the base. If the base appears dry, it can be sprayed with water just before the repair is made. The monomer is lighter than the water, so the water does not come to the surface of the repair.

Making the Repair

The monomers, cross-linking agents, and promoters can be premixed a few hours or a day before use in the field. The initiator is then added when the repair is made.

The monomer can be mixed with the aggregate in one of two ways. The simplest consists of wetting the concrete surface by using the monomer system, placing the mixed, dry aggregate in the repair area, and screeding to a level surface. The monomer is then poured or sprinkled over the aggregate until it is fully saturated. The surface is screeded again to a reasonably smooth finish. Sand can be sprinkled on the surface to provide a smoother surface for trowelling. When added to the surface, MMA powder in the proportion of 10 parts MMA to 90 parts sand tends to form a surface skin in a few minutes, which minimizes evaporation of the monomer. If MMA powder is not used, it is recommended that the repair area be covered with a polyethylene sheet during curing.

If the repair is full depth, it is desirable to place the aggregate to about one-half the depth of the hole, add monomer, add the remaining aggregate, and add the required monomer. In all cases, it is recommended that the aggregates be vibrated or tamped to reduce honeycombing in the PC.

The second method involves mixing the aggregate and monomer in a concrete mixer or a wheelbarrow before the matrix is placed in a hole. The aggregate (coarse and fine) is placed in the mixer or wheelbarrow and an amount of monomer system equal to about 12 percent by weight of the aggregate is added and mixed in for 2 or 3 min. The mix is then placed in the repair hole. Finishing proceeds as in the first method. The mixer or the wheelbarrow and tools can be cleaned by using monomer without the initiator or by washing with water.

It may be necessary to add more monomer to the repair when either of the two methods is used. The aggregate should be kept saturated until curing begins. As the monomer begins to cure, heat is generated. The PC usually cures in 30 min to 1 h. When the repair has cooled enough to permit safely placing one's hand on the surface, the repair is usually ready for traffic.

Safety and Handling

The chemicals used should be handled with reasonable care. Some of them are flammable and toxic, although no more so than gasoline or kerosine. They should be kept in a cool, shaded, well-ventilated area. Some of the chemicals, especially TTEGDA, cause irritation when they come into contact with the skin. Gloves, rubber boots, and goggles are recommended when the materials are used. DMPT and BP should never come into contact with each other in concentrated forms because an explosion may occur. The use of BA should be limited, especially indoors, because of its strong odor.

Field Applications

Numerous repairs have been made in Texas on both continuously reinforced concrete pavement (CRCP) and jointed pavement by using PC. The repairs include spalls, punchouts, joints, and cracks. Both PC systems have been used with essentially the same effectiveness. Figures 4 through 7 are schematics that illustrate the typical types of repairs.

Typically, four to eight repairs are identified in a section of pavement to use the same traffic control for all repairs. Once the repair area has been prepared, PC requires 45 min to 1 h for placement and curing. Once PC has cured, the repair can be opened to traffic.

In addition to usual monitoring of the repairs, deflection measurements were made before and after the repair. The deflections were measured by using the Dynaflect.

Figures 8-10 illustrate typical results of the PC repairs. Figure 8 shows deflections for a sound pavement. Figures 9 and 10 show the change in deflections before and after repair. Comparing the deflections shows that some, if not complete, structural integrity is restored to the pavement. In other words, the life of the pavement is extended. No definitive results are available to indicate the length of the extension.

In addition, PC has been used with precast PCC panels to repair larger punchout areas in CRCP and is being tested for full-depth runway repair. The precast repairs will be reported in a separate paper and hence no details are provided here except Figure 11, which illustrates the type of installation.

SUMMARY

PC can be used to make rapid repairs to areas of highway or runway pavements that show distress. The use of PC can restore some measure of structural integrity to the pavement and prolong pavement life. PC is expensive; it ranges from $250 per cubic yard for some user-formulated materials to $1800 per cubic yard for some prepackaged materials. However, when user-delay costs are included, the use of PC can be justified for most pavements in the primary highway system.

ACKNOWLEDGMENT

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Pressure Grouting of Concrete Pavements

JOHN DEL VAL

A brief overview of current practice in concrete pavement jacking and in grout subsealing of concrete pavements is presented. One of the major causes of concrete pavement failure is the loss of support caused by the pumping action beneath the pavement. Early detection of this condition and prompt filling of the voids created will prevent early deterioration of the pavement. Topics discussed are materials and their necessary physical properties, equipment requirements, and methods and the current state of the art.

The preservation and extension of the useful life of concrete pavements is becoming of great importance today. Although little understood or used, the techniques of cement grout subsealing and concrete pavement jacking offer proven help to achieve longer life and better rideability of concrete pavements.

One of the major causes of pavement failure is the loss of support caused by the pumping action of the pavement. Early detection of this condition and prompt filling of the voids created will stop premature deterioration of the pavement.

When pavement and bridge-approach settlements occur, they can be raised back to their correct grade within specification tolerances by using the proper materials, methods, and equipment. Early correction of these problems will inhibit damage to the concrete caused by the loss of uniform support.

Pavement jacking, sometimes called mud jacking, is the raising and leveling of concrete pavements that have settled by the injection of grout under the pavement and the use of hydraulic pressure to raise the pavement to its correct grade. Although pavement jacking has been in use for many years, the measure of success has always been rather low. However, with the materials available today, better equipment, and improved techniques, it is possible to guarantee 100 percent performance in this pavement maintenance work.

The following discussion covers the items that must be considered in developing a maintenance program for pavement jacking and the writing of a specification to contract the work.

MAINTENANCE

Settlement of pavement usually occurs during the first years after construction. Typically one may observe these occurrences at cut-and-fill transitions, backfill at culverts, and bridge-approach panels. The presence of water is generally an important factor in the failure of the subgrade.

Early correction of these problems is desirable both to satisfy the users and to extend the useful life of the pavement. As the settlement occurs, portions of the pavement are unsupported. The concrete has substantial ability to flex and bend in this unsupported condition, but eventually heavy and repeated loadings will cause the slab to break. Even badly broken panels can be successfully jacked to grade, but the structural and riding qualities can never be fully restored.

When repairs to be done are evaluated, one should look for contributing factors that should also be corrected. These may be drainage problems, clogged bridge drains that cause water to flood out over approach panels, unsealed open joints and cracks, open edge joints at shoulders, lack of proper drainage in roadside ditches, etc.

The pavement-jacking slurry will assist in correcting some of these problems since it will fill and seal many of the cracks and joints from beneath. It has been observed that even hairline cracks are filled.

SPECIAL BIDDING REQUIREMENTS

The specifications should refer to the contracting agency's standard specifications, air- and water-pollution requirements in the area in which the work is being performed, and traffic control requirements according to the Federal Uniform Traffic Control Manual or appropriate state manual if applicable.

Since pavement jacking is an art rather than a rigorous science, the achievement of specification tolerances in the finished work is dependent on the skill and expertise of the contractor and the workers. The bidding contractor must be able to show substantial work of this character and scope completed satisfactorily during recent years. The agency cannot allow the learning process to take place on public pavements that must be in service daily and risk overjacked pavements and damaged and broken panels from use of incorrect techniques that can be repaired only by closing the roadway for removal and replacement of the pavement. Experience is best gained in noncritical locations such as floors in industrial and warehouse buildings and parking lots.

ESTIMATING QUANTITIES

In order to estimate the quantity of grout required to jack a pavement settlement to grade, the approximate volume of the visible settlement must be computed and then 20-25 percent added for voids under the pavement. The same procedure is used for bridge-approach panels; 50-60 percent is added for the large voids that generally occur. Additional adjustments must be made for any other voids such as inadequate backfill at culverts.

Any additional location may greatly underrun or overrun these estimates, but when a number of locations are averaged, they will usually balance out.

MATERIALS

The original jacking slurries were simply a silty loam soil that was pumped under the pavement. It was soon discovered that these early mud slurries were not very satisfactory. It took many days for the water to drain out or evaporate from the slurries, and they were constantly subject to erosion by water. Some additional stability was given to these mud grouts by the addition of small percentages of cement, but the problems still persisted. Attempts were then made to do the jacking by using better-controlled materials such as sand-and-cement grouts. These grouts presented additional problems in that the angularity of the particles generally found in most sands made it difficult for it to flow into narrow spaces, and the sand frequently went out of suspension under pressure. Bentonite and other materials were added in an attempt to keep the sand in suspension; the result was lower grout strength. Cement-and-pozzolan slurries have a definite advantage over other materials, and their use is highly recommended.

The history of pozzolanic materials dates back to the days of ancient Rome, but the use of these materials in grout slurries is a fairly recent development. The family of pozzolans includes natural pozzolans such as volcanic ash, diatomaceous earth, and artificial pozzolans (fly ash) produced by the combustion of coal. These materials are classified under ASTM C618. The use and performance of these materials in grout slurries is not to be confused with their use as a cement replacement in structural concrete.

Several properties of pozzolans are responsible for improved pavement-jacking characteristics. These include particle size and shape, gradation, and pozzolanic activity.

Figure 1. Pavement jacking by using conventional grout.

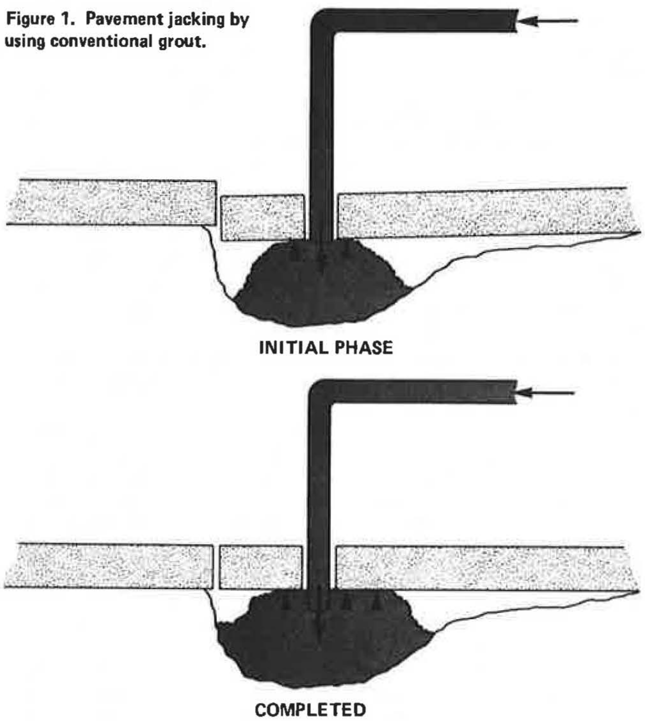
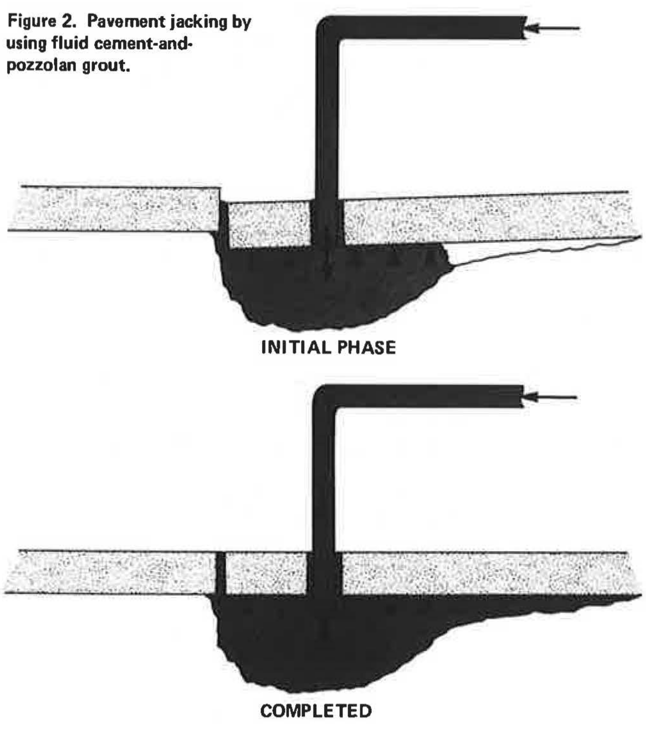


Figure 2. Pavement jacking by using fluid cement-and-pozzolan grout.



The fineness of the pozzolan particles and their predominantly spherical shape enble grout-incorporating pozzolans to be more easily pumped than those that contain only cement or cement and sand or other mineral fillers (Figures 1 and 2). The spherical shape results in a ball-bearing effect, which enhances the flow properties of the grout. Partial replacement of either cement or sand by using pozzolans improves permeability and injection penetration by keeping the grout in suspension and thus reducing sedimentation.

During the jacking process, as the pavement is being lifted new voids are obviously being created that must be promptly filled in order to maintain a uniform hydraulic pressure on all parts of the slab. In addition, permanent support is required to protect the pavement from excessive flexure and breaking. The pozzolanic grouts have the unique ability to fulfill these needs.

Although pozzolan particles are mainly the size of silt, they also contain a small but effective amount of clay-sized particles. These clay-sized particles provide sufficient grading to reduce segregation during pumping and injection; less voids and increased durability result.

The pozzolanic characteristics of these materials in combination with lime produce a stable cementitious material. Since the hydration of cement produces lime, additional cementition results when pozzolans are mixed with cement. This reaction produces a more-effective bond than that developed between sand and cements in weak cement grouts.

The sole purpose of the grout is to transfer the vehicular wheel loads from the rigid pavement to the underlying base or subgrade. The engineering properties that the grout must possess are incompressibility, nonsolubility, and nonerodibility. The grout must be confined in situ and cannot be displaced laterally after it has lost fluidity. It does not require flexural strength. The actual in-place yield of these grouts will be as much as 10 percent greater than that of other grouts when dry volume is compared, because of the fine particle size. This creates an additional economy in their use.

MIX DESIGN

A typical mix design for pavement jacking is as follows: one part (by volume) of portland cement type 1 or type 2, three parts (by volume) of pozzolan (natural or artificial), and water to achieve the required fluidity. Type 3 (high early) cement may be specified if there is a need for early strength.

Because of variations in pozzolans from differing sources, the contractor should be required to supply test reports from a laboratory that has competency in this field that show chemical and physical properties of the material suggested for use as well as compressive-strength tests (one-day, three-day, seven-day), flow-cone times, shrinkage and expansion observed, and time of initial set.

The pozzolans should meet the requirements of ASTM C618. Should the material not meet these requirements, the contractor may be allowed through testing to show that the proposed pozzolan will meet or exceed the qualities needed for the project.

Some of the western (class-C) ashes are sufficiently reactive that reduction or elimination of the cement component may be considered.

The consistency of these slurries is not measurable by using conventional slump-cone testing techniques. Water content is determined by the flow-cone method (U.S. Army Corps of Engineers, CRD-C 79-77), and for pavement-jacking slurries it should be in the range of 16-26 s. A very fluid mix (flow-cone range 9-15 s) may be used for the first few minutes of the jacking operation if needed to penetrate very dense materials and permit pumping of the jacking slurry. These ranges will allow for differing field conditions, and the flow-cone tests should be made twice a day. Even with high water content, this material will show little or no bleed water. The purpose of slab jacking is to replace lost subgrade and subbase support for the overlying rigid pavement. The strength required is limited to that of the minimum subgrade strength that underlies the pavement.

The determination of initial set time of the grout in the laboratory tests is useful in comparing various mixes. Generally, the Gilmour needle test is used and, on some occasions, the Corps of Engineers test methods (CRD-C 82-76) are used. However, none of these current test methods address the fact that cement-and-pozzolan grouts at normal temperatures will lose their fluidity within approximately 20 min after placement. Since they are virtually always in a totally confined environment, they are therefore capable of supporting substantial loads long before the set time of one and one-half to two hours that would be indicated by the generally accepted testing methods. Reapplication of vehicular traffic immediately after the completion of jacking has shown no pumping or displacement of the in situ grout.

Various additives may be specified to achieve required goals. Laboratory tests show widely differing reactions from the same additive when it is combined with pozzolans from different sources, and various brands of similar admixtures will react differently when combined with the same pozzolan. Each must be tested and evaluated in the laboratory prior to final approval by the contracting agency. They may consist of water-reducing agents, fluidifiers, superplasticizers, expanding agents to offset the shrinkage sometimes found with volcanic ashes, calcium chloride to accelerate set, etc.

EQUIPMENT

As a minimum, the following equipment will be needed:

1. A grout plant that has the ability to accurately measure and batch the dry materials and water. The dry materials should be packaged in uniform-volume sacks or measured by weight if they are in bulk. Water should be metered or weighed. To achieve a true colloidal mix that will stay in suspension and resist dilution by free water, it is necessary to use a high-speed colloidal mixer that operates in a range of 800-2000 rpm. Placement of the grout is done by a positive-displacement cement-injection pump.
2. A water tanker that has a pressure pump and adequate capacity for the day's production.
3. An air compressor and rock drills or a mechanical hole-drilling device capable of drilling holes of the required number and diameter (usually 2 in).
4. Necessary hoses, valving, and valve manifolds to control pressure and volume; pressure gauges and gauge protectors; expanding packers for the grout injection; wood plugs; hole-washing tools; drill steel and bits; material transport and handling equipment; and service trucks.
5. Traffic-control equipment as required.

TECHNIQUES

Hole-drilling patterns for pavement jacking should be determined in the field by the contractor based on conditions such as location of joints, cracks, and subgrade conditions. Extra holes may be required during the progress of the jacking to apply additional pressure in a particular area. Because the hole drilling is determined by the contractor, it should not be a paid item and will be considered incidental to the jacking. Holes may be washed by using water or blown by using air to create a small cavity from which the grout slurry can then spread. The injection holes should be drilled through treated base so this material may be lifted with the pavement.

The jacking is then begun by the injection of the grout slurry. Pressure should be observed and will be of the order of 75-200 psi. In some conditions, pressures as high as 300 psi may be required for short periods. When the pavement structure is bonded to the subgrade, brief pressure increases to 600 psi may be tolerated. Pressures this high or higher cannot be sustained without damage to the pavement. At bridge-approach panels, care must be exercised to prevent lifting the slab off the seat.

String lines with blocks should be continually monitored for pavement movement. Vertical offsets between panels and lanes should not be tolerated nor should overjacking. Final grade on jointed pavements and bridge-approach panels should be 0.03 ft plus or minus from the string line, and on continuously reinforced pavements it should be 0.02 ft plus or minus from the string line. In field conditions most of the work can usually be brought to within the thickness of the string. Any lesser tolerances will exhibit poor riding qualities.

On occasion, difficult situations may arise that need special consideration. In the case of bridge panels that are down 5 in or more, the tolerances should be increased to 0.04 ft. This is because a panel so far down is usually wedged or bound. Sawing full depth at the joints will generally relieve this problem, in which case extra payment or a bid item for sawing should be set up.

Cracks in the pavement that emanate from the drill holes are an indication of overpressure and poor grouting techniques. Consideration should be given to the imposition of penalties in the event of this occurrence.

Some small quantities of grout will be wasted on the pavement from the insertion and removal of packers and leaks at panel joints and along the edge joints. Uncontrolled flow and waste that go into the ditches should not be paid for.

WORK BY OTHERS

Work done by others may include asphalt shoulder adjustments, removal of overlays, joint and spall repairs, etc., or they may be included as separate bid items in the contract.

MEASUREMENT AND PAYMENT

A single unit of measurement [cubic feet (dry measure) of grout placed into or under the pavement structure] can be used, or it may be expanded into items that cover mobilization and traffic control. The basic grout item will include all labor, materials, and equipment; incidental items; additives; hole drilling; etc. This will simplify field management of the project. To bid this work on a square-yard basis is not advisable, since the contractor will then hedge the bid because of unknown grout volume.

Publication of this paper sponsored by Committee on Pavement Maintenance.

Choosing Cost-Effective Maintenance

G.J. CHONG, W.A. PHANG, AND F.W. JEWER

A project is described that was conducted under the Pavement Maintenance Strategies Task Force of the Ontario Ministry of Transportation and Communications. Its objectives were (a) to develop guidelines for corrective pavement maintenance to be used by maintenance patrol staff that suggest a method for evaluating pavement distress and the appropriate cost-effective and practical maintenance treatment alternative and (b) to conduct a pilot test of the developed guide to verify the procedures to be carried out in full later. The guidelines were developed from existing standards for pavement maintenance quality and related management systems data combined with the judgment of individuals experienced in the fields of pavement design and evaluation, construction, and maintenance. Emphasis was on the collection of subjective performance data on various maintenance treatments through personal interviews with experienced maintenance personnel. This material was incorporated into a working copy of the pavement maintenance guidelines. The pilot test was based on the working copy and a combination of an audiovisual presentation and individual instruction of selected maintenance patrol staff. This was followed by a return visit to patrols for interviews to obtain comments on the usefulness, ease of use, and validity of action levels described in the guidelines. The pilot test was conducted in 14 patrols in the five regions of the province. The results obtained confirmed that the working copy of the pavement maintenance guidelines could, with some minor changes, be adopted for full use in the province.

As the television commercial unfolds, the richly dressed elderly lady with the chauffeur and the Mercedes-Benz in the background stresses the point that her enviable life-style was achieved through shrewd investment in material goods that are of lasting value. Furthermore, she maintains her affluent life-style by protecting her investments against undue depreciation through a program of cost-effective maintenance. She has chosen Speedy Muffler King to do the corrective maintenance necessary on her car's muffler because of their lifetime guarantee. Her message is simple: Protect your investment by timely maintenance at the cheapest cost or suffer the consequence of reduced resources. This message, a perfectly sound management practice at any time, is particularly fitting in this time of high inflation and fiscal restraint. The extensive highway system in Ontario, like the lady's fine car, has been a very sound investment because benefits accrued to the public have been worth many times the money spent. The system represents billions of dollars of public funds and is a valuable asset that must be protected against unwarranted deterioration that can affect the useful service life as well as the quality of service offered the public. In addition, emphasis now is not only to protect but to prolong the pavement service life, because continuing budgetary constraints will surely result in the curtailment and postponement of capital expenditures for the highway system.

In order to achieve the twin objectives of protecting and prolonging the service life of the pavement in the system, a maintenance program that promotes timely maintenance and fosters cost-effectiveness is needed. In Ontario, a systematic approach to the total maintenance program has recently been developed that tries to incorporate these needs. The two principal features of the approach are these:

1. Pavement quality standards for the maintenance management system are being revised to introduce more-uniform methods of deficiency identification and to foster corrective actions that are estimated to be most cost-effective, i.e., corrective maintenance; and
2. Complementing these deficiency-corrective actions are the courses of action that anticipate the occurrence of deficiencies or are intended to retard progression of defects, i.e., preventive maintenance.

The corrective-maintenance measures are carried out to protect the integrity and safety requirements of the pavement system, and the development and implementation of this corrective-maintenance management process is the subject of this paper. The preventive-maintenance feature of the systematic approach deals with those actions that stave off the inevitable consequences of age and traffic. Development in this area is described elsewhere (1).

OBJECTIVES

The basic objectives of this study were as follows:

1. The development of a corrective-maintenance guide to be used in identifying the most suitable type of maintenance treatment for any given pavement condition and the most appropriate timing of the application,
2. The development of a process for evaluating the cost-effectiveness of various maintenance treatments, and
3. Pilot testing to confirm the systematic management process of corrective maintenance represented by this guide as the proper and effective methodology for full implementation.

APPROACH

The chief aim of the guide is to help patrol staff, patrol supervisors, and district maintenance staff to identify the most cost-effective maintenance treatment for any given pavement distress problem. Therefore, this guide must include four basic elements:

1. A simple and functional system of identifying and describing the distress problem,
2. A list of appropriate alternative solutions to correct the problem,
3. A standardized methodology for cost-effectiveness comparison to generate the most desirable solution, and
4. A standard for quantity and quality of work output for each possible solution.

There previously existed approved ministry methods to identify and describe various kinds of pavement distress that were used in assessing pavement condition ratings (2,3) as well as maintenance quality and performance standards (4). These methods and standards served as the basis for formulating this guide.

The general approach used for the project can be described as follows:

1. Use the existing methods and standards to formulate a functional but comprehensive format that contains the four basic elements outlined above and obtain estimates of life of treatments by interviewing experienced maintenance staff,
2. Verify the functional practicality of this format through personal interviews of experienced personnel,
3. Formulate a working guide for pilot testing and verification, and
4. Assess the pilot-test results to establish

the final product, the publication Pavement Maintenance Guidelines: Distresses, Maintenance Alternatives and Performance Standards (5).

FORMULATING THE GUIDE

The aim is to provide a practical guide for our district maintenance field staff to use in their day-to-day operations. Use of this guide does not require our field staff to produce an inventory of pavement distress. It is not a full-blown pavement management system, but it is an important part of our overall pavement maintenance strategies.

Although it is not practical to reproduce the guide here, a typical example will enable the reader to comprehend the structure of the guide--treatment of alligator cracking.

Figure 1. Alligator cracking (extract from pavement maintenance guide).

Description:	Cracks form a network of multi-sided (polygon) blocks resembling the skin of an alligator. The block size can range from 5 to 10 cm to about 50 cm. The alligatored area may or may not be accompanied by distortion in the form of depression, and may occur anywhere on the pavement surface.	
Possible causes:	1. Insufficient pavement strength. 2. Poor base drainage and stiff or brittle asphalt mix at cold temperature.	
Severity:	**Class.**	**Guidelines** (Base on appearance and surface distortion)
	Slight	Alligator pattern established with corners of polygon blocks fracturing
	Moderate	Alligator pattern established with spalling of polygon blocks
	Severe	Polygon blocks begin to lift; may or may not involve potholes.
Density:	*Local:*	Less than 30% of pavement surface affected; distress spotted over localized areas only.
	General:	More than 30% of pavement surface affected; distress spotted evenly over entire length of pavement section.

Pavement Condition Survey Scheme

The first requirement of the guide is a system of identification and description of various kinds of pavement distress. This system must be simple and direct because it is not the function of the district maintenance field staff to trace the performance history of the highway pavements or to evaluate the pavement performance periodically to satisfy a data bank. For field staff, it is only important that they provide detailed descriptions of the extent of occurrence (density) and severity of any specific distress so that an appropriate maintenance treatment can be chosen.

Therefore, the simple scheme is based on work descriptions and action-oriented classification scales because (a) word descriptions are on a value scale in common everyday use, (b) the surveyor assesses and describes conditions in familiar terms, (c) the surveyor is only concerned about what action to take in view of the density and severity of the distress, and (d) the word descriptions help directly in identifying appropriate alternative maintenance treatments.

The terms "local" and "general" are used to describe the density of any pavement distress as follows: "local" means that less than 30 percent of the pavement surface area is affected by distress and that distress is spotted over localized areas only; "general" means that more than 30 percent of the pavement area is affected by distress and that the distress is spotted evenly over the entire length of the pavement section.

Eight types of distress for flexible pavements and 10 types of distress for rigid pavements are included in the guide. The extract from the guide shown in Figure 1 illustrates a typical format used

Table 1. Example of maintenance treatment alternatives: treatments for alligator cracking.

Evaluation			Maintenance Function Classification[a]		
Severity	Density	Recommended Maintenance Treatment Alternative	Routine Patrol	Nonpatrol	Expected Effective Life (years)
Slight	Local	No action			
	General	No action but monitor closely for future development			
Moderate	Local	Spray patch	1004		1
		Cold-mix patch	1001		1
		Hot-mix patch	1001 1002	1002	4
		Hot-mix patch for multilanes		1002	4
	General	Same as above but notify district office of situation and maintain close monitoring			
Severe	Local	Cold-mix patch	1001		0.5
		Hot-mix patch	1001 1002	1002	3
		Excavate; granular and hot-mix patch	1002	1002	7
		Improve drainage (additional)	-[b]	-[b]	
	General	Hot-mix patch for highways with AADT < 2000 and notify district office	1002		2
		Mulch for AADT < 2000		1014	3
		Granular lift and surface treatment for highways with AADT < 2000		1017	4
		Hot-mix patch over selected areas and notify district office for further action for highways with AADT > 2000	1002	1002	3

Note: AADT = average annual daily traffic.

[a]Numbers listed correspond to maintenance performance standards. [b]Contract work only.

Figure 2. Example of maintenance performance standard.

M-1002
MACHINE PATCHING
PLACING AND SPREADING

DESCRIPTION			
The machine placing and spreading of premixed asphaltic materials (hot or cold mix) to repair major surface defects such as depressions, bumps and other pavement defects. Includes preparation of patching area and compaction.			
ROAD TYPE HOT MIX MULTI-LANE	HOT MIX 2-3 LANE	MULCH 2-LANE	ALTERNATIVE METHOD FOR ALL ROAD TYPES
CREW SIZE 1-Grader Opr. 1-Roller Opr. 2-Shovellers/Rakers 1 to 5-Drivers (Asphalt Supply) 1 to 2-Drivers (Traffic Cont.)	1-Grader Opr. 1-Roller Opr. 2-Shovellers/Rakers 1 to 5-Drivers (Asphalt Supply) 1 to 3-Flagmen (Traffic Cont.)	1-Grader Opr. 1-Roller Opr. 2-Shovellers/Rakers 1 to 5-Drivers (Asphalt Supply) 1 to 3-Flagmen (Traffic Cont.)	2-Screed Oprs. 1-Roller Opr. 2-Shovellers/Rakers 1 to 5-Drivers (Asphalt Supply) 1 to 2-Drivers or 1 to 3-Flagmen (Traffic Cont.)
EQUIPMENT 1-Crew Carrier 1-Grader 1-Roller 1 to 5-Dump Trucks 1 or 2-Sign Trucks or Sign Trailers	1-Crew Carrier 1-Grader 1-Roller 1 to 5-Dump Trucks	1-Crew Carrier 1-Grader 1-Roller 1 to 5-Dump Trucks	1-Crew Carrier 1-Tailgate Spreader 1-Roller 1 to 5 Dump Trucks 1 to 2 Sign Trucks/ Sign Trailers
MATERIALS Hot or Cold Mix	Hot or Cold Mix	Cold Mix	Hot Mix
ACCOMPLISHMENT Tonnes	Tonnes	Tonnes	Tonnes
MANHOURS PER ACCOMPLISHMENT 0.7 - 2	0.7 - 2	0.7 - 2	
ACCOMPLISHMENT PER DAY 105 - 35	105 - 35	105 - 35	

to describe a pavement distress, in this case alligator cracking.

Maintenance Treatment Alternatives

Maintenance treatment alternatives run from simple spray patching to full-width machine-laid hot-mix patching and even to full-width single-course hot-mix resurfacing. However, in spite of the many maintenance treatments available, there is a more-limited selection suitable for correcting any particular distress condition. This is because the appropriateness of the treatment depends on the distress, the stage to which it has progressed, the timing of the treatment, the class of road and type of traffic, the traffic volume, the availability of materials, equipment, and funds, and so forth. Nevertheless, it is useful to have a uniform approach to the evaluation and description of pavement distress that is coordinated with corresponding sets of suitable maintenance alternatives from which effective solutions may be chosen.

Table 1 illustrates how appropriate maintenance treatments are identified based on the evaluation of severity and density of the distress (columns 1 and 2, Table 1). If, for example, moderate local alligator cracking is the condition, the alternatives suggested are spray patching, cold-mix patching, hot-mix patching, or hot-mix patching for multilanes.

Maintenance Treatment Performance Standard

Maintenance treatment alternatives can vary from work that can be done by patrol and equipment crews to work that needs specialized equipment and skilled personnel and to full-scale contract work. In general, routine corrective and small-scale preventive-maintenance work is usually handled by patrol-sized crews. Capability is generally dependent on the size of the job and on equipment and personnel available.

For these routine corrective and small-scale preventive-maintenance activities, performance standards are available to help the district maintenance staff plan, budget, and schedule their work by giving them necessary information to estimate costs, to estimate production quantities and material needs, to schedule workers and equipment for the job, and to provide the procedural steps that make up the maintenance method to ensure work quality.

For each of the alternative maintenance treatments listed in Table 1, there are maintenance performance standard numbers classified either as routine patrol or as nonpatrol. When the number listed is classed as "routine patrol", this means that the work is within the capability of average maintenance patrol forces. A listing under "nonpatrol" conveys the message that additional resources are needed and

Table 2. Example of labor cost chart for calculating unit cost of surface repairs.

Men in Crew	Hours Worked on Operation							
	1	2	3	4	5	6	7	8
1	8.05	16.10	24.15	32.20	40.25	48.30	56.35	64.40
2	16.10	32.20	48.30	64.40	80.50	96.60	112.70	128.80
3	24.15	48.30	72.45	96.60	120.75	144.90	169.05	193.20
4	32.20	64.40	96.60	128.80	161.00	193.20	225.40	257.60
5	40.25	80.50	120.75	161.00	201.25	241.50	281.75	322.00
6	48.30	96.60	144.20	193.20	241.50	289.80	338.10	386.40
7	56.35	112.70	169.05	225.40	281.75	338.10	304.45	450.80
8	64.40	128.80	193.20	257.60	322.00	386.40	450.80	515.20
9	72.45	144.90	217.35	289.80	362.25	434.70	507.15	579.60
10	80.50	161.00	241.50	322.00	402.50	483.00	563.50	644.00

Note: Crew cost based on $8.05/h standard provincial labor rate for 1979-1980.

Figure 3. Extract from pavement maintenance guide that shows method for machine patching.

Recommended Method

1. Set up safety devices and signs in accordance with "Traffic Control Manual for Highway Work Operations."
2. Remove any loose or broken sections of existing pavement extending into the good pavement about 30 cm. Clean up the patch area, removing all loose material and water.
3. Compact the subgrade, if necessary.
4. Apply a tack coat at each end of the patch and along the centreline joint when one lane is to be patched.
5. Place and spread the mix with a grader or a tailgate spreader.
6. Smooth and feather out the edges of the patch.
7. Be sure that enough mix material has been placed to obtain the proper grade and cross-fall.
8. Compact the patch area, rolling progressively from the high side toward the low side, overlapping a few centimetres on each pass.

Note: Joints at the ends of the patch should be spray patched at a later date.

Procedures for Placing and Spreading Hot or Cold Mix Asphalt (Machine Patching)

must be provided either by combining patrol forces or by contracting out the work.

Cost-Effectiveness

The mission of the maintenance staff is to select the most cost-effective maintenance treatment appropriate to the particular situation, to ascertain its practicality, and to carry out the maintenance.

Cost comparison of maintenance alternatives is best made by using their equivalent annual cost, which is based on the cost of doing the job and the expected life of the work accomplished. The performance standards provide the estimated cost of each

Table 3. Preidentified equipment listing and rental rates.

Description	MTC Equipment Code	Current (1979) Rental Rates ($/h)
Half-ton pickup	0003	3.00
Three-quarter-ton pickup	0006	3.10
Three-quarter-ton five-man cab	0006	3.10
Three-ton dump	0023	3.90
Four-ton dump	0023	4.05
Five-ton dump	0031	7.10
	0041	8.40
	0051	
	0052	9.05
Grader (100 hp)	0105	9.95
Portable roller	0558	8.70
	0562	5.45
	0564	14.70
Loader (1.25 yd^3)	0430	8.20
Asphalt tailgate spreader	0719	7.65
Maintenance kettle	0420	7.35
Rubber/asphalt kettle	0422	13.10
Concrete saw (router)	0630	9.80
Asphalt distributer	0892	17.45[a]
Pulvi-mixer	0461	22.90
	0462	18.55
Aggregate spreader	0721	28.35
Wobble-wheel roller	0559	13.65
Steel-wheel roller	0563	10.90
	0566	16.35
	0567	16.35
Broom-towed type	0186	15.90
Air compressor	0232	7.65
	0239	7.65
Jackhammer	0360	2.40
Sign-truck unit	0655	3.80
Sign trailer	0861	7.35

[a] Add to truck rate.

maintenance alternative chosen, which is the basic unit cost. That is,

(Personnel + equipment + materials)/(accomplishment per day) = unit cost.

To obtain the equivalent annual cost, which is the indicator used to evaluate cost-effectiveness, the unit cost is matched against the life expectancy of the maintenance treatment. That is,

Unit cost/(expected life of alternative in years) = equivalent annual cost.

Expected life is defined as the time in years (or months or even weeks) before the distress treated reappears and progresses to the condition that existed prior to the treatment. The expected life given for each maintenance alternative under a particular situation represents the median value of estimates obtained through interviews with experienced maintenance staff.

Figure 4. Cost calculation for machine patching: hot mix.

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

MAINTENANCE PRODUCTION UNIT COST AND ANNUAL COST

OPERATION

NUMBER 1002

ACTIVITY Hot Mix Patching

Alligator Cracking; Moderate, Local

CREW SIZE

POSITION	CLASS	No. MEN	RATE	HOURS /DAY	COST	% OF TOTAL
TOTAL		9	8.05	72	$579.60	

EQUIPMENT COMPLEMENT

TYPE	No. PCS.	RATE	HOURS /DAY	COST	% OF TOTAL
Crew carrier 3/4 T - 5M	1	$3.10	6	$ 18.60	
Dump truck 5 T	3	9.05	18	162.90	
Grader – 100 H.P.	1	9.95	6	59.70	
Roller	1	14.70	6	88.20	
TOTAL				$329.40	

MATERIAL

TYPE	QUANT. /DAY	UNIT	COST/ UNIT	COST	% OF TOTAL
	70	t	$2004	$1402.80	
TOTAL				$1402.80	
GRAND TOTAL COST PER DAY				$2311.80	

REMARKS

Section of Highway to be prepared 30 km.
Std. Planning Value Applied;
.8t/E2L km.
30 km x .8 t = 24 t Reqd.
24 t x $33.03 = $792.72

Expected life of repair four years

ACCOMPLISHMENT: PER DAY (APPROX.)	70 t	
PRODUCTION RATE: MAN-HOURS PER ACCOMPLISHMENT UNIT	1.029	ACCOMPLISHMENT / MAN-HOURS
UNIT COST: DOLLARS PER ACCOMPLISHMENT UNIT	$ 33.03	TOTAL COST / ACCOMPLISHMENT
ANNUAL COST: DOLLARS PER YEAR	$198.18	UNIT COST / EXPECTED LIFE

The last column in Table 1 gives four years as the life for hot-mix patching (performance standards 1001 and 1002) for correction of moderate local alligator cracking and one year for either cold-mix patching or spray patching. The details of the comparison of cost-effectiveness for these alternatives in this situation are given below.

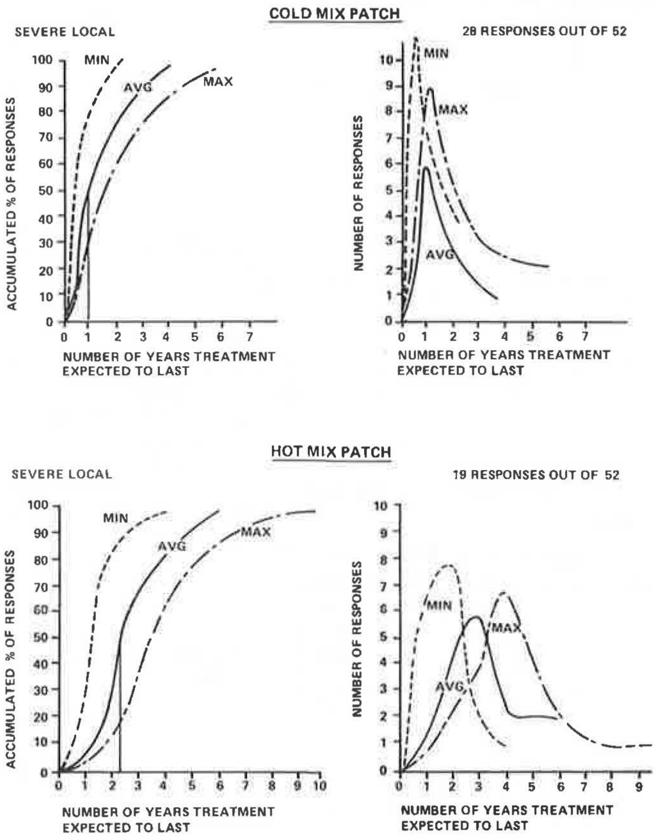
In determining the most cost-effective maintenance treatment available, the cost of every recommended alternative must be compared: personnel, equipment, and materials are the major components that determine the cost of each alternative. The following must be calculated:

1. The required crew size, equipment, and materials are stated in the maintenance performance standards (Figure 2). This standard (M-1002) suggests a crew of 6-12 workers. Equipment consists of one crew carrier, one grader or tailgate spreader, one roller, one to five dump trucks, and one or two sign trucks or trailers. The materials are cold or hot mix. The recommended method, illustrated by a photograph, is given in Figure 3.
2. Examples of a labor cost chart and equipment rental rates for calculating the unit cost of surface repairs (in dollars per hour) are provided in Tables 2 and 3. In hot-mix patching, a crew size of, say, nine workers will cost $579.60/day (Table 2). The cost of equipment rental is computed by summing the cost of equipment per day. The cost of a piece of equipment is dependent on rental rates and the number of hours it is used per day. As an example, one crew carrier used for 6 h/day at rental rates of $3.10/h will cost $18 60/day. The cost of materials varies according to tender or locality. The cost of these items is recorded on a form similar to that in Figure 4.
3. The total cost of maintenance per day is the total of the crew cost, equipment cost, and material cost per day.
4. The accomplishment per day is the result of processed materials per day.
5. The unit cost in dollars per accomplishment unit is calculated as follows:

Total cost/accomplishment = unit cost.

6. The annual cost in dollars per year is calculated from

Figure 5. Expected effective life of maintenance for distortion of severe local condition.



Unit cost/expected life = annual cost.

Similar cost calculations are also recorded for the alternative treatments of cold-mix patching and spray patching.

Following the completion of these calculations for each alternative, a cost comparison may be made and the most cost-effective alternative is made evident. The comparison in this case is in favor of machine patching by using hot mix. However, the urgency of maintenance; the availability of materials, equipment, and funds; and a variety of other factors should be considered before a decision is made as to maintenance treatment.

VERIFICATION OF FORMAT

A draft format of the pavement distress manifestations and maintenance treatment alternatives was prepared as described earlier. In this draft it was suggested that certain maintenance treatments and alternatives are suitable for various conditions of pavement distress (Table 1). The very significant item that was missing from the draft was the expected effective life of a maintenance treatment.

Methodology

This draft and a questionnaire were presented to a selected group of staff members within the ministry and responses were later solicited during individual interviews. The candidates interviewed were chosen for their present and past experiences in pavement maintenance. In all, more than 50 candidates were selected; each had had maintenance-related experience of more than 20 years, i.e., an aggregate of more than 1000 person years of experience. Information obtained was assembled and analyzed to establish the processes in the pavement maintenance guide, to verify the suitability of alternative treatments, and to provide the estimates of expected effective life for each treatment and condition.

Questionnaire Results

Responses to the questionnaires indicated that, in general, the format presented is efficient, understandable, and simple for use as a management tool in the hands of the field maintenance staff. Overall, only minor changes, additions, and deletions were required.

The missing expected effective life of the various maintenance treatments was provided by the experienced personnel. The expected life given for each maintenance treatment under a particular situation represents the median value of estimates provided by the subjects interviewed. An example of the analysis is given in Figure 5.

It should be clearly understood, however, that the expected life based on past experience is provided at this time only as a guide. An organized monitoring program will be undertaken to verify many of these estimates. However, when it is realized that for rigid pavements there were 32 treatment situations for which estimates of expected lifetimes were needed and that 102 estimates were needed for flexible pavements (a total of 134 estimated lifetimes), this verification task appears formidable.

The results were assembled and a working copy of the pavement maintenance guide was established for pilot testing.

WORKING COPY

The working copy consisted essentially of three sections. The first told the patrol worker how to use the guide to identify the extent and severity of any given type of distress, how to use the guide to identify suitable alternative maintenance treatments for the specific condition, how to calculate which of the alternatives is the most cost-effective (provided it is an activity within the worker's scope), and finally how to report the condition if the remedial measures are outside of the worker's capability.

The second section consisted of types of distress and maintenance alternatives for both flexible and rigid pavements. The types of distress for flexible pavements, which are adequately illustrated by means of photographs, are (a) raveling and streaking; (b) flushing; (c) slippery surface; (d) potholes; (e) rippling and shoving; (f) wheel-track rutting; (g) distortion (sagging, dishing, depression, settlement, bump and frost-related bump, and excessive crown); and (h) cracking (longitudinal and transverse, map, progressive edge and edge breaking, and alligator). The maintenance alternatives for these distresses are manual patching by using cold mix or hot mix, machine patching by using cold mix or hot mix, crack filling with or without routing, spray patching by using liquid asphalt, mulching pavement by using grader mixing or spreader mixing, treating the surface, cold planing, heater planing, burning and sealing, and hot-mix patching by using recycled asphaltic materials.

The types of distress for rigid pavements and their corresponding maintenance alternatives are also adequately illustrated in the guide. Types of distress for shoulder and surface drainage are also described and illustrated by photographs, but no treatments are given. This chapter is simply a reminder of the importance of shoulder and surface drainage as related to pavement performance service life.

The third and final section of the guidelines contains the performance standards. Each standard provides information on required crew size, equipment, materials, the unit of accomplishment, the person hours per unit of accomplishment, and the accomplishment per day. Further, the standard provides a step-by-step fully illustrated description of the method used to carry out the treatment.

PILOT TESTING

The working copy of the pavement maintenance guidelines, which incorporated all the recommended changes, was printed for distribution in the pilot study. The main objectives of the pilot study were to determine the practicality of the processes, the suitability of the treatments, and the success of the cost-effectiveness approach and to gauge the overall impression of the maintenance patrols to the greater recognition given to maintenance through these processes.

Approach

A small number (14) of maintenance patrols was selected in different geographical areas of the province (five regions) to provide equal representation. Instruction was provided through an audiovisual presentation to the patrol workers and patrol supervisory staff that described the proposed guide and its use. This was followed by a question-and-answer session. Use of this guide as a management tool was examined by the selected maintenance staffs. Last, the effectiveness of this guide was verified through questionnaires and individually conducted personal interviews.

Results of Pilot Testing

The results indicated that, in general, the guide in its existing working-copy format required only minor changes. Highlights of the questionnaire results are as follows:

1. The guide is easy to use and understand.
2. The guide provides uniformity in evaluation processes and management methodology.
3. The guide offers a simple and functional system of identifying and describing the distress problems.
4. The guide offers the appropriate maintenance alternatives for the types of distress identified.
5. The guide offers an easy-to-use methodology for comparing and choosing the most cost-effective maintenance alternative for the distress problem.
6. At the patrol and patrol-supervisory levels, there is a developing awareness of their responsibility for choosing practical and cost-effective treatments and a sure knowledge of their role in the task of pavement maintenance.

APPLICATION OF FINDINGS

The findings from the pilot testing project were assembled to effect the required changes in the working copy. The result is Pavement Maintenance Guidelines: Distresses, Maintenance Alternatives and Performance Standards (5). Full use of this guide by the field maintenance staff is being carried out by the ministry's maintenance branch.

FUTURE RESEARCH

Studies to provide performance data on maintenance alternatives are needed to verify the life estimates used in the guide that were obtained through personal interviews of experienced maintenance staff. Monitoring programs are also needed for work methodologies, materials, and equipment to improve maintenance techniques and thus lengthen the effective life of the treatment. New maintenance techniques and equipment and new materials that become available should be examined periodically in the future to extend the usefulness of the guidelines.

REFERENCES

1. W.A. Phang and W. Blum. Preventive Pavement Maintenance Concepts. Ontario Ministry of Transportation and Communications, Downsview, 1980.
2. G.J. Chong, W.A. Phang, and G.A. Wrong. Manual for Condition Rating of Flexible Pavements: Distress Manifestations. Ontario Ministry of Transportation and Communications, Downsview, Aug. 1975.
3. G.J. Chong, W.A. Phang, and G.A. Wrong. Manual for Condition Rating of Rigid Pavements: Distress Manifestations. Ontario Ministry of Transportation and Communications, Downsview, Aug. 1977.
4. Maintenance Manual Quality Standard, Volumes 1 and 3. Ontario Ministry of Transportation and Communications, Downsview, n.d.
5. G.J. Chong, F. Jewer, and K. Macey. Pavement Maintenance Guidelines: Distresses, Maintenance Alternatives and Performance Standards. Ontario Ministry of Transportation and Communications, Downsview, Rept. SP-001, 1980.

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