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Assessing Pavement Maintenance Needs

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Contents

Foreword

Authors of papers published in this Record provide information on four themes:

- Evaluation of the usefulness of specific instruments for measuring certain attributes of pavements,
- Evaluation of the effectiveness of selected maintenance procedures,
- Mathematical models that relate pavement attributes and other quantifiable variables to maintenance needs and priority evaluations made by local area supervisors, and
- Computer programs that can be used to coordinate plans at central headquarters or to evaluate competing alternatives.

A survey of a jointed, reinforced concrete pavement with ground-penetrating radar indicated that the equipment provides a nondestructive technique that can be used at a minimum rate of 5 lane-miles of pavement per hour to detect voids deeper than 1/8 in. and that the information derived from the survey can be useful in developing a sound and cost-effective slab stabilization operation through the proper placement of grout holes. This work confirmed previous work that demonstrated successful use of the Dynaflect instrument to test for voids. Use of the Dynaflect revealed that blanket undersealing was actually detrimental at times and that, to be effective, undersealing must be done on a selective basis only at locations of known voids.

A study of concrete pavement rehabilitation techniques (pavement grinding, improved full-depth patch design, and procedures) indicated that the cost of these techniques is only about 50 percent of that of resurfacing for 10 years of service. And a study of rutting of asphalt concrete pavement overlays on continuously reinforced concrete pavements in Texas indicated that overlay thickness was an important predictor of rutting in overlays.

A mathematical model developed to assess highway maintenance needs and establish rehabilitation threshold levels proved reliable for 86 percent of the study locations in establishing an optimum threshold value to use in the selection of a repair technique. In another paper a procedure is described that can be used to estimate routine maintenance work loads by highway section during a coming year or season. The procedure is based on a periodic survey of highway unit foremen and subsequent use of a set of quantity standards developed by relating the foremen's subjective ratings of road conditions to objective measurements of distress in the field. Subjective ratings are then transformed into expected work loads.

Two of the papers are about the development of microcomputer data bases for use at different maintenance management levels. The first suggests use of combined data from condition survey information and roughness measurements to coordinate programming and for use by the central office to assist subdistricts in setting priorities for routine maintenance activities. The other paper is on the development and implementation of the PRDS-1 computer program that uses a system approach to economic and structural evaluation of rehabilitation alternatives.

Use of Ground-Penetrating Radar for Detecting Voids Under a Jointed Concrete Pavement

Gerardo G. Clemeña, Michael M. Sprinkel, and Robert R. Long, Jr.

A survey of a jointed, reinforced concrete pavement with ground-penetrating radar indicated that radar provides a nondestructive inspection technique that can be used at a minimum rate of 5 lane-miles of pavement per hour with only minimal interference with traffic. The coring of some slabs and subsequent use of a devised water test revealed that the radar was effective in detecting voids deeper than 1/8 in. but considerably less effective in spotting shallow voids. The overall accuracy was approximately 68 percent, which indicates that the sensitivity of the equipment needs to be improved. The location component used with the radar unit showed insufficient accuracy. A regression analysis of the recorded quantities of grout used daily in subsealing portions of the pavement versus the total linear feet of voids detected under the slabs grouted each day yielded only a 51 percent correlation. However, the regression was found to be significant at a 95 percent probability level. It is believed that if the width and depth of each void can be conveniently estimated so that the extent of voids can be expressed in terms of volume instead of length alone, an even more successful method of estimating grout quantities would be available. It has been shown that information derived from a radar survey can be useful in developing a sound and cost-effective slab stabilization operation in which grout holes are properly placed.

Each year an increasing number of miles of concrete pavement become in need of maintenance and repair. Because the formation of voids beneath concrete slabs is a major cause of pavement failure, there is an urgent need for a rapid, nondestructive inspection technique for detecting such voids before failures occur. The value of knowing the location and extent of voids when planning for slab stabilization is immeasurable.

Recent studies *(1–5)* have shown that cavities under concrete sidewalks, runways, and approach slabs to bridges can easily be detected with ground-penetrating radar (GPR). Consequently, there is a trend toward increasing use of this technique in the inspection of concrete pavements. However, with concrete pavements, failures can occur when the gap (or void) between a slab and the subbase is only 0.125 in. deep, and the ability of GPR to consistently provide accurate indications of this type of void had not been evaluated and reported.

GPR was used to survey a 14.5-mi section of Interstate 81 in southwestern Virginia for the purpose of *(a)* evaluating the accuracy of GPR in detecting voids, *(b)* determining if the radar could be used as a reliable method for estimating the quantities of grout needed to fill the voids, and *(c)* examining other possible uses of the technique.

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PRINCIPLE OF GPR

The GPR systems used generally have been of the short-pulse type. This type operates on the principle of inducing a single pulse from a transmitter, then abruptly ceasing transmission for a short interval (typically on the order of microseconds) during which reflected signals return to a receiver. In all recent studies, the transducers (i.e., the combined transmitter and receiver) used have had a pulse width of approximately 1 nanosecond (nsec). These transducers operate at relatively high frequencies (typically 1 GHz) to provide both sufficient penetration (approximately 3 ft) and the best available resolution. This is in contrast with the low-frequency transducers that are more suitable for geological and similar surveys because they yield relatively greater penetration albeit poorer resolution.

When a pulse of electromagnetic energy is directed into a concrete pavement (Figure 1), a portion (P1) is reflected back to the transducer at the air-concrete boundary, which is the first boundary between two media having highly contrasting dielectric properties (Table 1). The remaining energy propagates through the concrete until it strikes another boundary, which would be the concrete-base boundary, where another

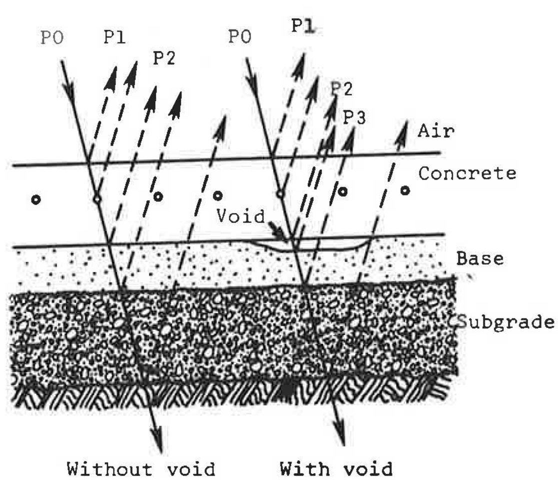

FIGURE 1 Propagation of microwave pulses through a concrete pavement without and with a void under the concrete slab.

TABLE 1 DIELECTRIC CONSTANTS OF MATERIALS RELEVANT TO PAVEMENTS

Material	Relative Dielectric Constant (ε)
Air	1
Concrete	6–8
Sand (dry)	4–6
Granite (dry)	5
Limestone (dry)	7–9
Dolomite	6–8
Soil	3–12
Water	81

portion (P2) is reflected back. The portion not reflected penetrates through the layer of base material and other subsequent layers of materials and repeats the reflection-and-penetration processes until the original energy is completely dissipated. (The maximum penetration would depend on the moisture content of the materials below the concrete slab.)

As is shown in Figure 1, when a void exists below the concrete slab, an additional reflection (P3) is caused by the additional air-base (void-base) boundary. In addition, the P2 reflection would become stronger (i.e., have a larger amplitude) than that where no void exists because of a change in the nature of the boundary (i.e., from concrete-base to concrete-air or void, and a corresponding increase in the reflectivity at the boundary. These recognizable changes in the reflection pattern of a pavement, as shown in Figure 2, make possible the detection of voids beneath the concrete slab.

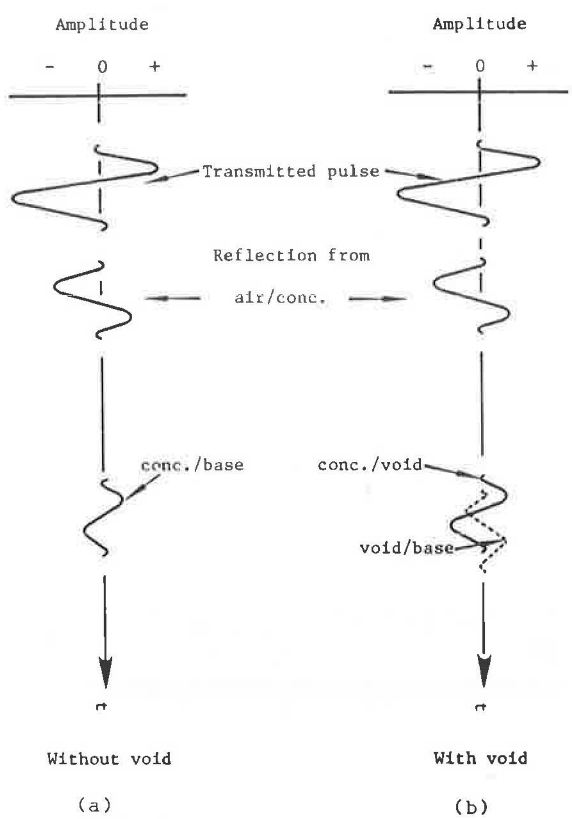

FIGURE 2 Microwave reflection profile for concrete pavement without and with a void.

The amplitude *(A)* and polarity of the energy reflected from a boundary are governed by

$$\rho = A/A_o = (\sqrt{\varepsilon_1} - \sqrt{\varepsilon_2})/(\sqrt{\varepsilon_1} + \sqrt{\varepsilon_2}) \tag{1}$$

where

ρ = reflection coefficient at the boundary,
A_o = amplitude of the incident energy,
ε_1 = relative dielectric constant of medium (or material) 1, and
ε_2 = relative dielectric constant of medium (or material) 2.

In accordance with this equation, the surface of the concrete pavement (i.e., the air-concrete boundary) would have a reflectivity of negative value because concrete has a dielectric constant greater than that of air, as indicated in Table 1. Therefore the energy reflected from the surface of the pavement would have a negative polarity with respect to that of the transmitted pulse in Figure 2*a* and an absolute amplitude proportional to A_o and ρ air-concrete.

The polarity and amplitude of the reflection P2, at the concrete-base boundary, would depend on the base material used. In the section of I-81 surveyed the base material was a 2-in. sand leveling course over a 6-in. layer of crushed stone (either limestone or dolomite). If the sand is dry during the survey, P2 will likely be positive and have an amplitude only about one-tenth that of P1, as shown in Figure 2*a*. However, if the sand is moist, its dielectric constant could be considerably greater than that of concrete, so that P2 would become negative and stronger.

Where there is a void beneath the concrete slab, the concrete-base boundary is, of course, replaced by a concrete-void and then a void-sand boundary. In this case reflection P2 would remain positive, although its amplitude would become larger, maybe about one-third that of P1. In addition to this change, an additional reflection (P3) would occur and assume a negative polarity of considerable amplitude (Figure 2*b*).

The difference in the times at which any two successive reflections reach the receiver would depend on the thickness and the dielectric constant (ε) of the material between the reflection boundaries; that is,

$$t = 2D\sqrt{\varepsilon}/(11.81) \tag{2}$$

where

t = pulse arrival time, or separation from a preceding reflection, in nanoseconds, and
D = thickness, in inches.

According to this relationship, reflections P1 and P2 would arrive at the receiver about 3.3 nsec apart, assuming an average slab thickness of 9 in. This is considerably more than the minimum 1 nsec (the pulse width that characterizes the transducer) needed to prevent their overlapping and interfering with each other.

Because the relative dielectric constant of air is 1, according to Equation 2 reflection P3 from the bottom of a void would be behind P2 in proportion to the depth of the void; that is,

$t = D/5.9$ (3)

where D is the depth, in inches, of the void. It was suspected that the voids beneath the pavement slabs on I-81 would probably be no deeper than a few eighths of an inch and that P3 and P2 thus would be separated by no more than 0.06 nsec. This implies that P3 would often overlap P2 and, therefore, affect its amplitude, as is shown in Figure 2*b*. Again, it is by observing these changes in the reflection corresponding to the bottom of the concrete slab that voids can be detected.

During a survey the electromagnetic pulse is repeatedly transmitted through the pavement (at a rate of 50 kHz for one radar system) while the radar system travels slowly (5 to 10 mph) over the pavement. This creates a stream of radar reflection profiles that contain information on the pavement being surveyed.

RADAR SYSTEM FOR SURVEY OF PAVEMENTS

The radar survey of I-81 was contracted to Gulf Applied Research of Marietta, Georgia (mention of a company does not constitute endorsements by the authors). The system used, as diagrammed in Figure 3, has a radar unit and a location reference unit integrated with a wide-angle video unit that records pavement surface conditions for possible correlation with subsurface conditions derived from radar data. The radar unit operated two antennas simultaneously, which provided coverage over two survey paths, each approximately 18 in. wide, in a traffic lane in a single pass.

The system was also equipped with a reference marking unit that automatically sprayed a paint mark on the pavement at 1,000-ft intervals during the survey. (This capability has since been improved to apply the marking at intervals as short as 10 ft.)

SURVEY PROCEDURE

The antennas were set up 5 ft apart. Then the survey vehicle was driven over each lane on a course such that Antenna 1 was approximately 3 ft from the edge of the pavement shoulder or the longitudinal joint (Figure 4). The survey of the entire 14.5-mi section of this four-lane highway, which translates to 58.0 lane-miles of jointed concrete pavement, was accomplished with five runs made in approximately 12 hr. Only minimal traffic control was necessary because any lane closure necessary for setting up antennas or extracting cores for verification was quite brief.

RESULTS AND DISCUSSION

Microwave Reflection Profiles of Concrete Pavement

Figure 5 shows a pair of microwave reflection profiles for 950 ft of pavement recorded on magnetic tape during the survey and later transferred to a strip chart recorder. The top and bottom profiles correspond to the 3- and 8-ft wheelpaths, respectively. (The horizontal scale in each pattern corresponds to the traveled distance from a starting point, and the vertical scale corresponds to the arrival time of the reflection at the antenna receiver or to the depth of the various materials when the dielectric constants involved are known.)

In the portion of pavement shown in Figure 5, numerous

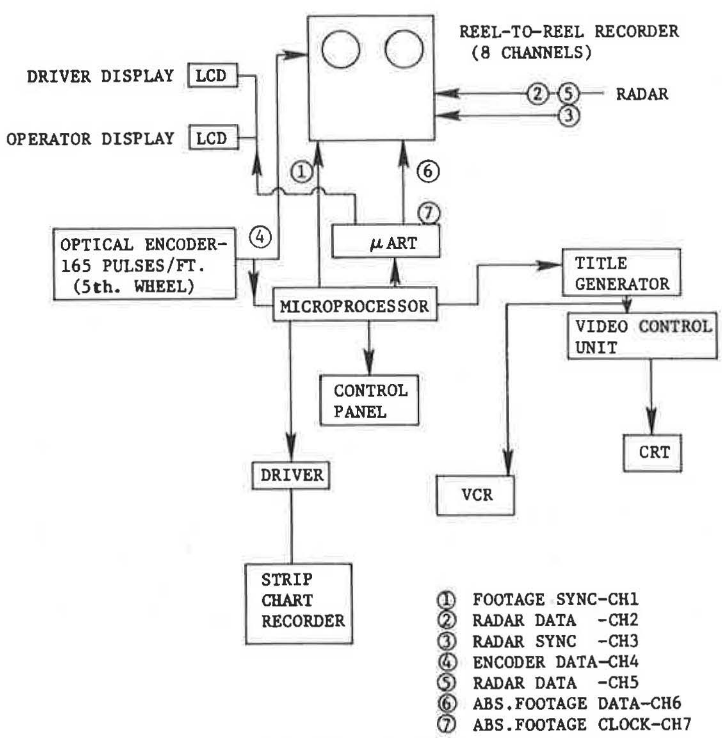

FIGURE 3 Diagram of Gulf Applied Radar's RODAR unit.

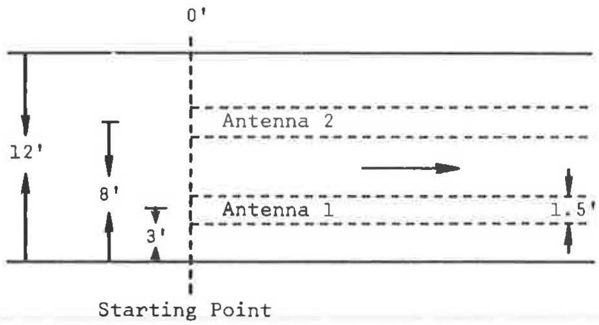

FIGURE 4 Antennae paths on each lane.

voids of various sizes were evident. Each was manifested by a pair of relatively intense white-then-black bands at the concrete-base boundary caused by reflection P3 and its interaction with P2, as discussed earlier. The transverse joints, which were 61.5 ft apart, were readily recognizable as peaks at the top of a reflection profile, especially the upper one. Careful examination of this set of reflection profiles would also show that practically all of the voids were downstream of the joints with respect to the direction of traffic flow, which is from left to right of the chart. This is particularly noteworthy because it provides a cross-sectional "picture" of what occurs under the concrete slab and around a joint or crack. [This picture appeared to support the view of experts in pavement rehabilitation that during pumping there is a movement of particles counter to the direction of traffic across a joint that often results in a buildup of loose materials under the slab upstream of the joint (i.e., the approach slab) while some fine materials are pumped out, through the joint or crack, from under the slab downstream of the joint (i.e., the leave slab) to create a void there.]

Although not all joints showed up readily in the reflection patterns recorded, joints with severely damaged sealant were quite distinct. This was also true of severely faulted joints or cracks. Although not shown, bridges had a typical reflection pattern distinct from that of the pavement.

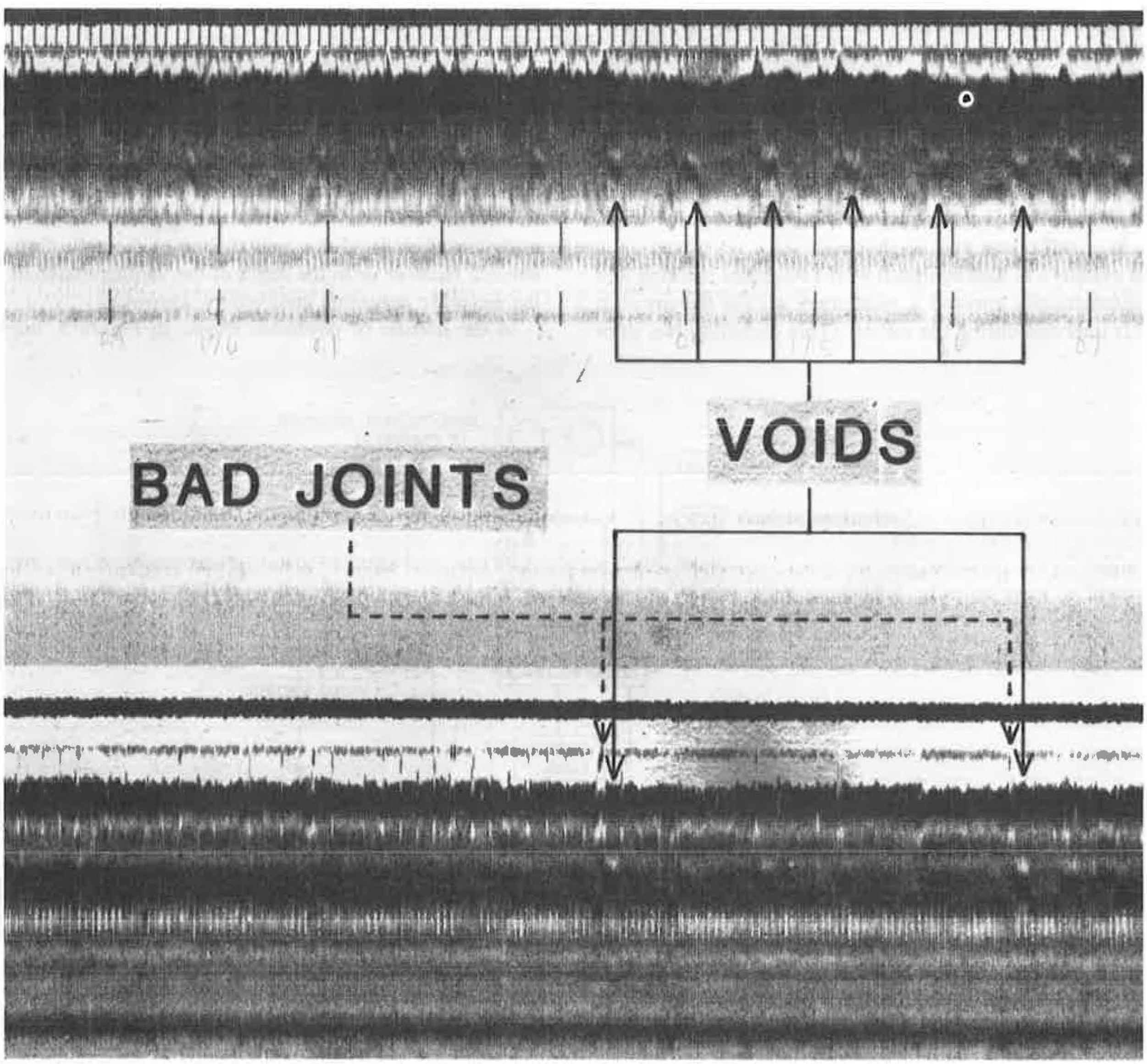

FIGURE 5 Reflection profile for a 950-ft section in the northbound travel lane of I-81 in Botetourt County, Virginia.

Detection and Location of Voids

Briefly stated, the detection and location of a void involves two steps: *(a)* sensing the reflection associated with the void by the radar unit and *(b)* location of the void, with respect to the entire pavement, by the location reference unit. Each step can contribute an error that affects the overall accuracy of a pavement survey.

Error from the Location Reference Unit

To determine the reliability of the location reference unit, the distances (to the nearest foot) between consecutive reference markers in a travel lane were measured. Each distance was then compared with the intended distance (or interval) of 1,000 ft.

The results, shown in Figure 6, indicated that the measured distances were consistently greater than 1,000 ft. The errors, which assumed a normal distribution, ranged from 0 to 10 ft and averaged 4 ft. These errors may have been caused by improper tire pressure in the fifth wheel and pulse counts missed by the optical encoder. This second source of error can be minimized by using an optical encoder that counts more than the 25 pulses per foot produced by the encoder used in this survey. Use of such an encoder would make occasional missed pulses less significant. It has also been suggested that mounting the fifth wheel at the center at either the front or the rear of the survey vehicle instead of at the side would minimize error.

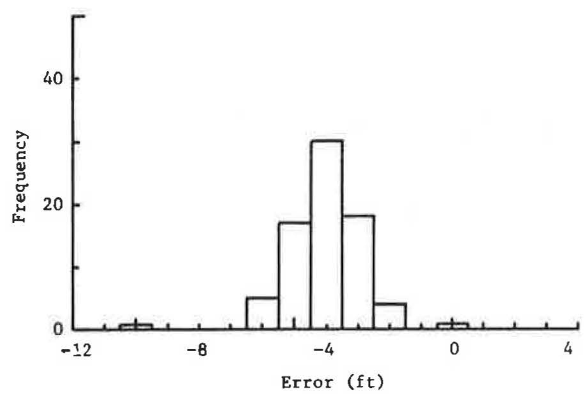

FIGURE 6 Observed errors in the location reference unit.

If sealing the detected voids is planned, reference marks with errors of such magnitude as those obtained in the study could not be used with confidence by maintenance crews. Fortunately, for jointed pavements such as the section of I-81 surveyed, this problem can be remedied by *(a)* associating each void with the slab that is above it and *(b)* locating this void in terms of its distance from the upstream joint or the beginning of the slab. This distance is then determined by subtracting the position of the upstream joint from that of the void (both positions are measured with the location reference unit).

Using a shorter spacing, say 50 or 100 ft, between the reference marks may render error less significant. For continuously reinforced concrete pavements, using a short spacing is the only recourse because the remedial procedure for jointed pavements is not applicable.

Error Associated with the Radar Unit

The characteristic reflection pattern of most voids is readily recognizable in a reflection profile (Figure 5). However, the reflections for very shallow or small (in area) voids may be so small in magnitude that they cannot be detected by the radar unit; or, if detected, they may not all be interpreted as voids by the user. The latter situation is likely to occur when the reflections received by the antenna are presented on graphs or a video monitor as bands of varying gray tones. The voids that are either undetected or misinterpreted contribute to the errors that affect the overall accuracy of the radar technique, which will be discussed in the next subsection.

Overall Accuracy of Radar in Detecting Voids

Cores were taken from some slabs to verify the presence of radar-detected voids. If the core dropped more than 1/8 in., it was obvious that a deep void was present. If the core dropped 1/8 in. or less, the hole was inspected for grout. If grout was found, it was obvious that the void had been filled with grout between the time the radar survey was conducted and the time the core was taken. If no grout was present, the hole was filled with water, and the quantity of water was measured. The volume of the void was then estimated by subtracting from the total volume of water used, the volume of water required to fill the hole in the slab and the volume that drained through the subbase material. As will be explained later, a volume of ≤ 0.005 ft^3 was not considered to be indicative of a void. If the water test did not indicate a void, the void condition was designated "uncertain."

The method used to determine the quantity of water that drained through the subbase material was based on the assumption that the flow through the subbase was at a constant rate. For a constant flow rate the volume of water that flowed through the subbase was assumed to be equal to the volume of water in the core hole in the slab, which was approximately 9 in. deep, multiplied by the time required to fill the subbase, void, and core hole and divided by the time required for the water to drain from the hole in the slab. Typically, from 10 to 30 sec were required to fill the subbase, void, and core hole in the slab, and 5 min were required for the water to drain from the core hole. An evaluation of the permeability of the subbase revealed that when the subbase consisted of a 2-in. layer of sand over material with a California bearing ratio (CBR) of 30, the permeability coefficient was approximately 10^{-3} cm/sec. Therefore the error in estimating the flow through the subbase was negligible because of the small amount of water that flowed through the subbase during the test. In areas where there was no sand over the CBR-30 material, the permeability coefficient was approximately 10^{-1} cm/sec. For this condition if the CBR-30 material was not saturated before the water test was run, it could absorb as much as 0.3 lb of water, which amounts to a volume of 0.005 ft^3. All but one void identified by the water test had a volume in excess of 0.005 ft^3. Therefore the test method was reliable for the project conditions.

Table 2 gives the void condition at the locations that were cored. The detailed radar analysis included corrections for errors due to the location reference unit and interpretation of

TABLE 2 VOID CONDITIONS DETERMINED BY CORING AND RADAR

Condition Based on Coring		Condition from Detailed Radar Analysis	
Condition	No. of Cores	Voids	No Voids
Uncertain	28	10	18
Shallow	39	21	18
Deep	29	26	3
Grout	7	4	3

data. It can be seen that 103 cores were taken. Unfortunately, at 28 locations there was uncertainty as to whether there was a void. At only 10 of these locations did the radar indicate that a void existed. A void condition could be determined by coring at 75 locations: a shallow void was found at 39 locations, a deep void at 29, and grout at 7. It is obvious from these data that the radar survey had a high rate of success when the voids were deep. Twenty-six (or 90 percent) of the 29 deep voids were located by the radar. On the other hand, the radar identified only about half (21 of 39) of the shallow voids. Thus, as would be expected, the survey showed that radar is more likely to miss small or shallow voids than large or deep voids. It can be seen that, overall, radar found confirmed voids 68 percent (51 of 75) of the time.

Estimation of Quantity of Grout

Estimating the quantities of grout needed for a stabilization operation has been quite difficult. Reported attempts to correlate initial pavement deflections and the volume of grout pumped have not been successful *(6)*. As a last recourse, various historical averages of quantities of grout per grout hole have been used, even though this approach lacks any scientific basis.

It is the belief of the authors that the radar survey technique has the potential to provide a sounder basis for estimating grout quantities from the extent of detected voids. Consider that, theoretically, the total quantity of grout used (say, in a day) should be equal to the total volume of all of the voids beneath the slabs grouted; that is,

$$G = \sum V_i \tag{4}$$

where

G = total quantity of grout in cubic feet,
V = volume of an individual void in cubic feet, and
i = 1, 2, 3, . . . nth voids.

It is conceivable that voids can assume different shapes or that, in a simple case, a majority could assume the shape of a rectangle, a triangle, or a combination of both. To simplify this discussion, assume that each void is practically a rectangle that can be defined by a width (W_i), a length (L_i), and a depth (D_i). The total volume could then be expressed so that Equation 4 becomes

$$G = \sum W_i L_i D_i \tag{5}$$

If the three independent variables can be measured by radar with reasonable accuracy, it should be possible to arrive at a reasonably accurate estimate of the quantity of grout needed to fill the voids.

Unfortunately, the radar used had two limitations that prevented the adoption of this rigorous approach. First, the radar gave limited areal coverage. This limitation is apparent in Figure 7, which shows an actual void that had been filled with grout then exposed by carefully removing the entire concrete slab. In this example, only about 40 percent of the void, the area directly below the coverage of one of the two antennas, would be detected; and only its length could be defined or measured. (This limitation can be easily eliminated by building a radar system with the capability of using three, or even four, antennas simultaneously. With such a system, both L_i and W_i could be estimated with better accuracy.)

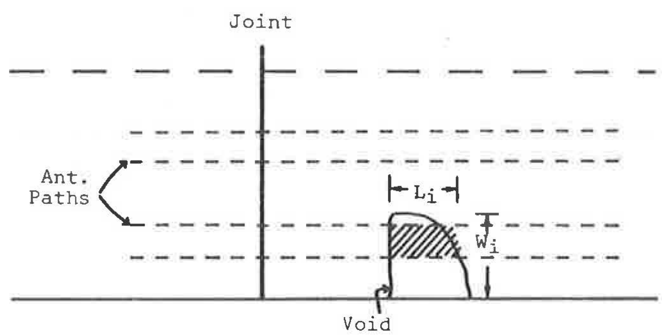

FIGURE 7 Limitation on coverage by radar; only the shaded portion of this void would be detected and defined by radar.

The second limitation concerned the determination of depth. As discussed earlier, most voids are no deeper than a few eighths of an inch, so the reflection from the bottom of a void (P3) would be overlapped by the reflection from the bottom of the slab (P2). This means that it would be extremely difficult, if not impossible, to estimate the time separation between these reflections so that the depth of a void could be calculated from Equation 3. This problem can be remedied by using the existence of a linear relationship between the amplitude of P3 and the void depth. This approach, however, also entails difficulty because it requires that a reasonably good correlation between these two variables be established by coring the pavement above a sufficient number of properly selected voids of various depths.

Because there was not sufficient information on void width and depth to make possible the use of the rigorous approach outlined in Equation 5, a simpler version was attempted.

Correlation of Grout Quantity with Length of Void

For a simpler version of Equation 5, it was assumed *(a)* that the depths of all voids were practically uniform (i.e., constant) and *(b)* that the widths of all voids also were practically constant. It is obvious that the first assumption would be more acceptable than the second. Nevertheless, with these assumptions, Equation 5 becomes

$$G = WD \sum L_i = k \sum L_i \qquad (6)$$

To determine the utility of this approach, the daily total quantities of grout pumped under various sections of pavement during 22 days of slab stabilization operations were measured. These daily total quantities were then correlated with the total length of radar-detected voids (in linear feet) underneath the slabs that were subsealed each day. (The pattern of grout holes used is shown in Figure 8. During the actual pumping of grout, the contractor and inspector used a modified Benkelman beam type of device to inspect and pumping was ceased when the grout appeared in adjacent holes or joints, to avoid raising any slab above grade.)

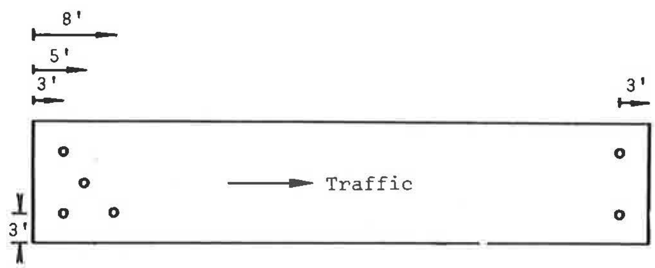

FIGURE 8 Pattern of grout holes used in slab stabilization.

As Figure 9 shows, there was only a 51 percent correlation between the lengths of the voids detected by radar and the quantities of grout used. It appeared that the correlation suffered at low void lengths, as is evident at the low end of the regression line.

The following points should be noted:

1. Any void in the area between the two antenna paths (Figure 5) obviously would not be detected and therefore was not included in the estimation of the total extent of voids, even though the void would be grouted.
2. Some of the detected voids were a considerable distance from joints, so they would be missed by the grout holes and not be filled.
3. There was a strong belief that more grout than was needed was pumped under some slabs, even though the construction crew took care not to lift the slabs.

Although the correlation was relatively poor, statistical tests indicated that the regression line was significant at the 95 percent probability level. This indicates that the rigorous approach to the estimation of grout quantities expressed in Equation 5 would be even more successful and should be tried as soon as the necessary three- or four-antenna radar units become available.

Other Uses of Radar Survey

Checking the Effectiveness of a Grouting Operation

So far the discussion has dealt only with the use of radar for detecting and locating voids under pavement slabs before pave-

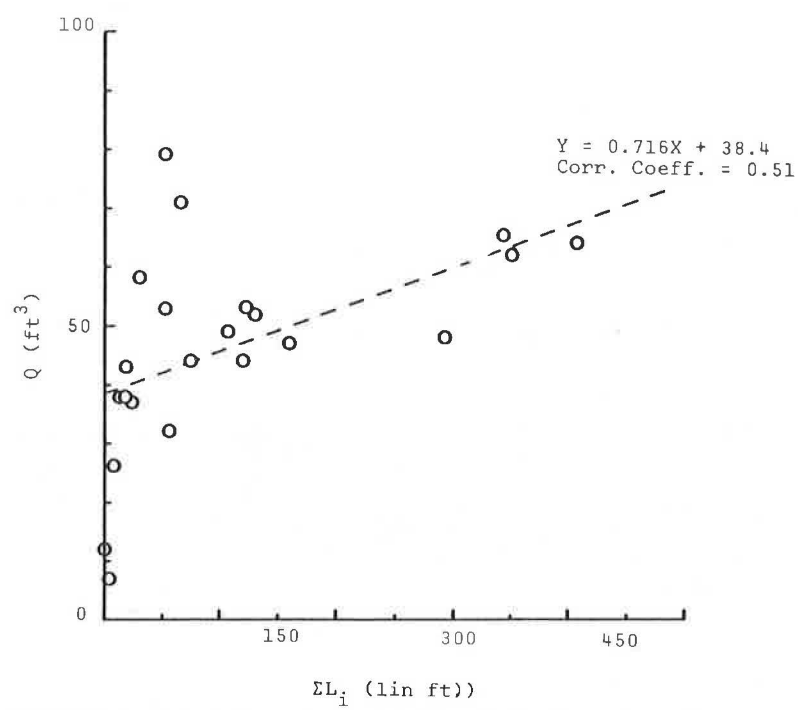

FIGURE 9 Correlation between total length of voids and grout quantity.

ment rehabilitation and restoration. Radar, however, can also be used after the repair to check whether the grouting operation was effective.

Figure 10 shows the reflection profile for a portion of the pavement that was covered in the radar survey and also happened to have been grouted during a preceding maintenance season. As is evident in this example, there was a sufficient difference between the dielectric properties of the grout and those of the base material for the presence of grout to be detected. More important, the profile also shows signs of voids in both antenna paths. It is possible that the voids were missed in the grouting operation or formed subsequently. There were also two joints that were likely to be defective; one happened to be next to the voids, and the other was only one slab (61.5 ft) away.

Designing Effective Patterns of Grout Holes

At the beginning of a slab stabilization project, it is common to select a pattern for the grout holes based on engineering judgment, and, depending on the results obtained during the course of the project, this may be changed. The information that radar provides on the location of voids can be useful in this selection process. To illustrate, consider some of the radar results obtained for the southbound lanes of I-81 that have yet to be rehabilitated. For some 13,000-ft sections of the southbound travel lane, where most voids were located, Figure 11 shows that, in general, a majority of the voids were centered around the joints for both wheelpaths. At the 3-ft wheelpath, 88 percent of the voids were centered within 7 ft downstream of a joint, 4 percent were centered within 2 ft upstream, and only 8

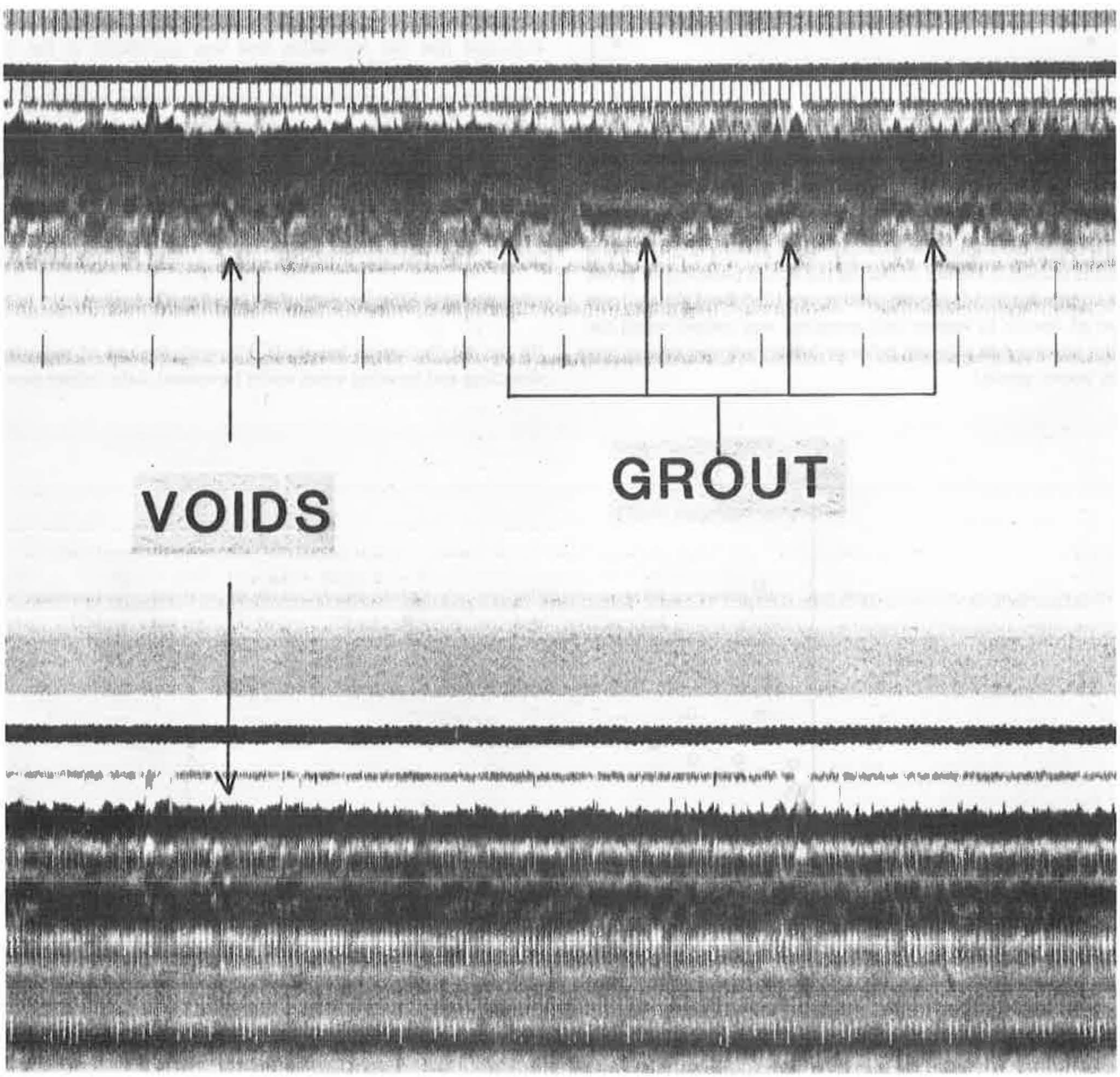

FIGURE 10 Reflection profile for a portion of a grouted concrete pavement showing grout and voids.

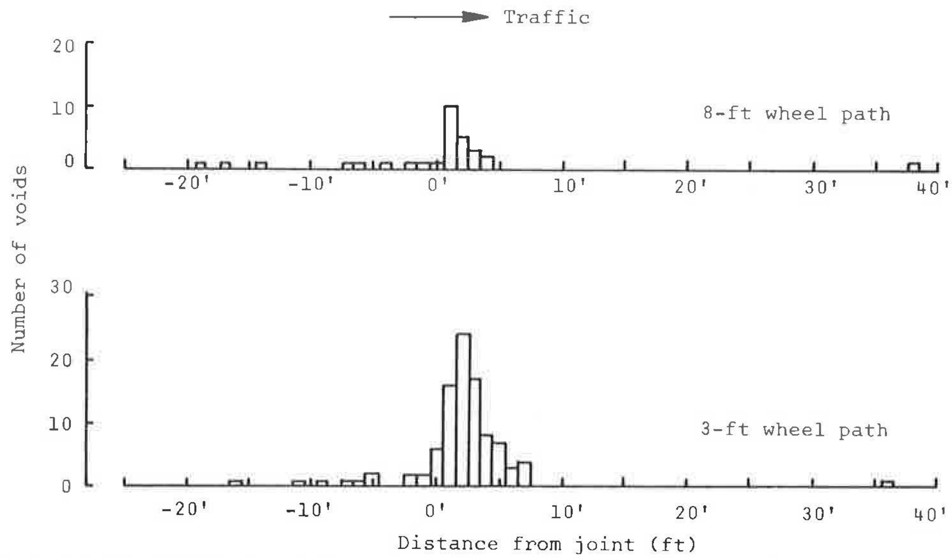

FIGURE 11 Distribution of the centers of the voids detected by radar in the southbound travel lane of I-81.

percent were relatively far from a joint. At the 8-ft wheelpath, 66 percent of the voids were centered within 4 ft downstream of a joint, 13 percent were centered within 2 ft upstream, and 21 percent were relatively far from a joint.

A further analysis can provide clues on how effective a specific hole pattern would be if used in filling these voids. Consider the use of the six-hole pattern shown in Figure 8 and assume that if a grout hole falls no farther than 1 ft from either end of a void, the void would be effectively filled by grout pumped from that hole. Also assume that the two top holes in this pattern coincide with the 8-ft wheelpath and that the three bottom holes coincide with the 3-ft wheelpath, thus allowing for some reasonable drifts around these intended wheelpaths during the survey. On the basis of these assumptions, the relative effectiveness of the grout holes, except the middle one, can be assessed.

Figure 12 shows the number of a particular grout hole that would coincide with a void, based on the analysis of 103 slabs. In the 8-ft wheelpath only 5 of the No. 1 holes and 19 of the No. 2 holes would hit or coincide with a void. The highest frequency of a hole coinciding with a void would occur at the No. 4 hole in the 3-ft wheelpath. At this location 76 of the 103 holes would coincide with a void. Also, note that, between the grout holes at each end of the slabs, 7 voids in the 8-ft wheelpath and 17 voids in the 3-ft wheelpath would not receive grout. It is obvious that, at a cost of $8.40 per hole, which equates to $63,000 for the 14.5 lane-miles, it would have been cost-effective to prepare the grouting contract on the basis of the results of a radar survey that cost only $6,000 for 14.5 lane-miles.

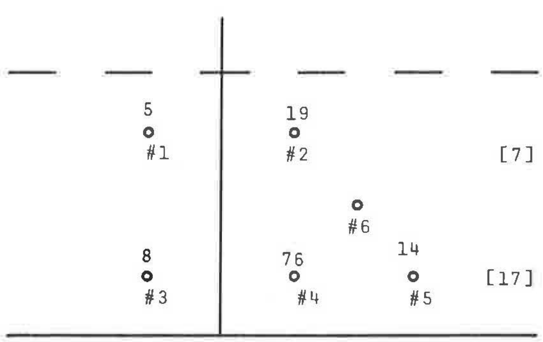

FIGURE 12 Number of each of the six standard grout holes that coincides with a void for 103 slabs; the bracketed numbers represent the number of voids in their respective vicinities that would be missed by the grouting operation.

Miscellaneous Uses

Other interesting information can be derived from the radar survey conducted on I-81. An example is shown in Figure 13, which shows the size distribution of voids detected by radar in the two southbound lanes. These log-normal distributions indicate that there were 10 times more voids in the travel lane than in the passing lane. (Similar types of distributions were observed for the northbound lanes, except that in the travel lane there were only four times more voids than in the passing lane.) Such a disparity between the extent of voids under a travel lane compared with a passing lane has also been inferred from reported deflection measurements made on concrete pavements. This disparity arose because travel lanes, in general, carry more traffic and more heavy truck traffic than do passing lanes.

Although not investigated in this survey, radar may also be useful in detecting poor drainage in a subbase, which is another cause of pavement distress.

CONCLUSIONS

On the basis of the preceding discussion, the following conclusions can be drawn:

1. A complete radar survey can be carried out at a minimum rate of 5 lane-miles of pavement per hour with minimal interruption of traffic.

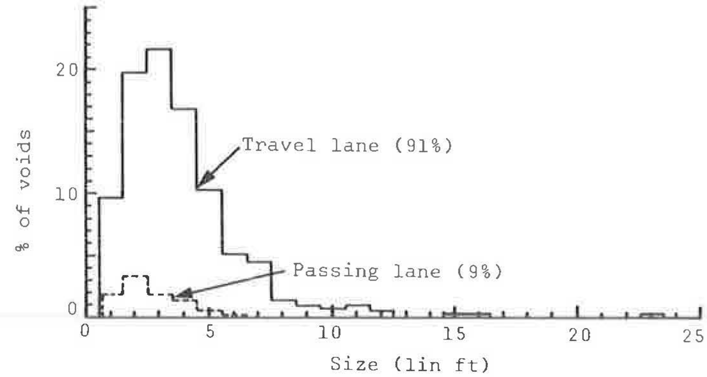

FIGURE 13 Size distribution of voids detected under southbound lanes of I-81.

2. The location reference unit used by the contractor in conjunction with his radar unit did not provide sufficient accuracy, which should probably be no less than ±1 ft.

3. Compared with data corrected for errors in the location reference unit and the interpretation of radar, cores taken from the slabs showed that the radar found known voids 68 percent of the time. As expected, radar found deep voids (>1/8 in.) 90 percent of the time and shallower voids only 54 percent of the time.

4. Despite this deficiency, GPR can already be used as a rapid nondestructive tool for surveying concrete pavement for underlying voids.

5. A regression analysis of data on daily grout quantities versus total linear feet of detected voids yielded a less than desirable degree of correlation (51 percent). This finding indicates that grout quantities cannot be estimated with reasonable accuracy from linear feet of voids detected.

6. However, the regression was statistically significant at the 95 percent probability level.

7. Radar surveys can be useful not only in detecting and locating voids before planning the stabilization of a concrete pavement but also for checking on the effectiveness of completed stabilization.

8. Radar surveys also provide information that is useful in deciding where to pump the grout for a cost-effective slab stabilization operation.

ACKNOWLEDGMENT

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Void Detection and Rigid Pavement Undersealing in Indiana: A Comprehensive Approach

ROGER A. MUTTI, JOSEPH J. SUDOL, AND BRADLEY W. LOVE

The Indiana Department of Highways (IDOH) has been undersealing concrete pavements with bituminous materials since the late 1940s. Most early rigid pavements were constructed directly on soil subgrades and were subject to severe pumping. Thus early undersealing operations involved treating entire sections of roadway. As pavement designs improved, severe pumping became less prevalent and a method of identifying only those areas that required undersealing became necessary. The approach taken by IDOH personnel was global in nature. Because it was impractical to locate and treat specific voids, a method was developed to identify and treat the most severely distressed areas. The method of void detection presented herein uses Dynaflect deflections measured at regular (100-ft) intervals within each contract section. Decision criteria based on midslab deflections are established for each contract; Sensor 5 is the primary indicator variable. Because decision criteria are obtained independently for each contract section, the method is applicable to both jointed and continuously reinforced concrete sections and to previously overlaid sections. When the areas that require undersealing have been identified, all cracks and joints within each area are treated. The procedure involves carefully monitoring slab motion during material injection with a sensitive deflection gauge developed specifically for that purpose. Furthermore, injection time limits are observed to minimize material losses due to blowouts. Data are presented that demonstrate both the validity of the void detection method and the joint deflection improvements that can be expected from the undersealing procedure. The economic feasibility of the method is discussed in terms of the savings that have been realized since the implementation of the method.

The Indiana Department of Highways (IDOH) began undersealing concrete pavements with bituminous material in the late 1940s. Most of the early rigid pavements in Indiana were constructed directly on top of compacted soil subgrade and were subject to severe pumping under load. Thus the earliest undersealing operations involved treating entire sections of roadway. In some instances, as much as 4 gal of asphaltic materials were injected per square yard of pavement. The earliest specifications also included provisions for second treatments where, in the opinion of the engineer, the first treatment was insufficient or unsatisfactory. Reliable procedures for locating voids were not especially critical because pumping was visually apparent at nearly all joints and cracks.

As more information became available and design procedures improved, the state began constructing its concrete pavements over granular subbases. The presence of this select subbase material greatly reduced pavement pumping; however, problems associated with slab instability remained, and bituminous undersealing continued to be an important part of the overall rehabilitation-overlay procedure. In the years that followed, studies were conducted to determine the most effective and economical methods of injecting the undersealing materials. The method evolved from pumping through several holes at each joint or crack to injection at a higher pressure through a single hole placed at midlane, 3 feet from and on the leave side of each joint or crack. In the absence of a reliable void detection method, typical undersealing contracts called for undersealing every joint and crack; as much as 40 gal of material were pumped into each hole. The large quantity of material was attributed to the presence of large voids. However, evidence of pumping of the newer pavements was not sufficient to justify these quantities; furthermore, no large voids could actually be identified. In an attempt to reduce the overall cost of undersealing operations, studies were initiated in the 1970s to establish a method of locating voids or areas of poor support.

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UNDERSEALING PROCEDURE

A comprehensive testing and undersealing procedure was developed by personnel of the Division of Research and Training of the IDOH and was implemented on a statewide basis in 1980. The method involves both Dynaflect deflection testing of each contract section to determine undersealing requirements and the detailed specification of undersealing procedures to be followed. The method is applicable, with minor alterations, to both jointed and continuously reinforced concrete (CRC) pavements. Because decision criteria are established for each contract section, the method is also valid for the evaluation of previously overlaid pavements. The steps of the Indiana method are outlined in the following subsections.

Step 1—Stationing

Each highway section scheduled for Dynaflect testing is "stationed" by the IDOH Construction Division. Large station markers visible from the test vehicle are requested.

Step 2—Equipment and Calibration

The state maintains a fleet of three Dynaflect testing machines. Operation is conducted in strict accordance with manufacturer's recommendations to minimize measurement variations both within and between machines. Furthermore, periodic correlation studies are conducted at the division's test road in West Lafayette, Indiana, to determine intermachine variation. Although factory calibration establishes the 1,000-lb peak-to-peak force output, the 8-Hz operating frequency is calibrated at regular maintenance intervals. The operator calibrates the five geophone displacement transducers before testing each day.

Step 3—Testing

Dynaflect testing is conducted at approximately 100-ft intervals in the outer wheelpath, about 3 ft from the shoulder edge of the pavement. Jointed pavement deflections are obtained by spotting the Dynaflect force wheels adjacent to and on the leave side of the joint or crack nearest each station (Figures 1 and 2). Typical jointed pavements were constructed with 40-ft joint spacing, but nearly all have cracked in two or three places so actual testing locations may occur as much as 15 ft on either side of the station markers. CRC pavements are tested at each station regardless of crack location.

Step 4—Decision Criteria

Although all five sensor readings are recorded, just two Dynaflect deflection values are used for the identification of the pavement areas that require undersealing. Numerous investigators have concluded that the displacement at the first sensor, commonly referred to as *DMD*, gives the best indication of pavement strength and most report using the difference between the fourth and fifth sensors *(BCI)* as an indication of support conditions. Majidzadeh *(1)* suggested using either the *BCI* or the Sensor 5 (W_5) value for this purpose. Experience in Indiana has shown the W_5-value, rather than the *BCI*, to be most sensitive to pavement support conditions. In reporting the results of tests in which voids were artificially created beneath a 9-in. pavement at both crack and center slab locations, Mutti *(2)* also concluded that W_5 is more sensitive to pavement support than is the *BCI*-value.

FIGURE 1 Dynaflect positioned at a typical crack in a jointed pavement.

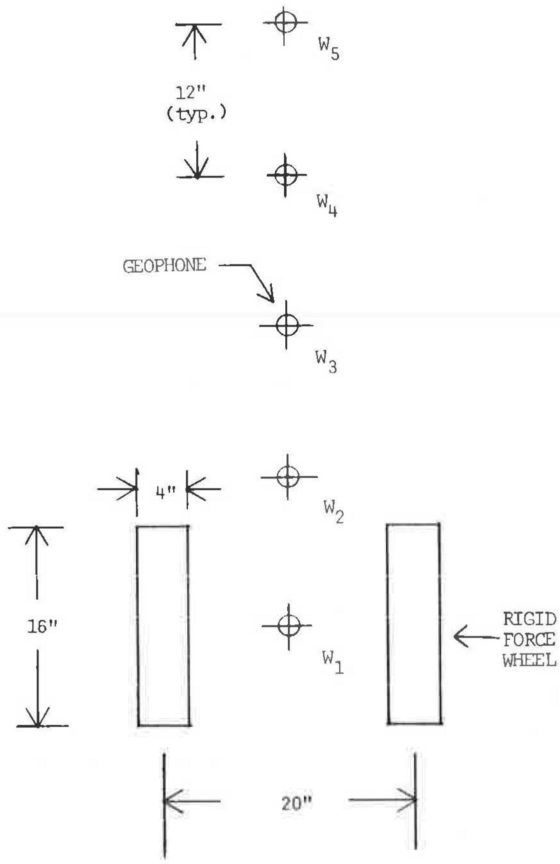

FIGURE 2 Dynaflect load and sensor arrangement.

Early in the development of the Indiana method, the magnitude of W_5 was the only decision criterion used. However, as more data were accumulated it appeared that undersealing areas where W_5 was marginal and *DMD* was low did not give satisfactory results. In cases in which borderline W_5-values were observed, pavement strength, as determined by *DMD* magnitude, showed little improvement if the initial *DMD*-value was less than approximately 0.60 mil.

Because of the inherent variability of in situ pavement properties, a value for the W_5 decision variable is established for each contract section. The determination of a suitable value is based on the premise that each slab must be fully supported at some point, and two possible locations that most often provide minimum deflection results have been experimentally identified. These are either along the outer wheelpath midway between joints or cracks, or both, (center-slab) or at the geometric middle of the slab (midslab). Continuously testing along the outer wheelpath is more convenient and center-slab results have been found to provide satisfactory W_5-values for most pavements.

Approximately 50 center or midslab readings are taken within each 5- to 8-mi contract section, and the average value of W_5 is used as the threshold limit for undersealing. Occasionally, relatively high W_5-values are obtained at some center or midslab test locations. These abnormal values are not included in calculating the average for the section, nor are the locations undersealed. Pumping has not been observed at these locations, and experience has shown that undersealing center or midslab areas almost always increases joint or crack deflections, or both.

The primary decision variable used for CRC pavement screening is the W_5-value. Threshold values are obtained by testing areas known to be performing "satisfactorily" at 1- or 2-ft intervals or from center or midslab deflection measurements taken on uncracked slabs at least 8 ft in length. Satisfactory performance is determined both visually and by reference to measured deflections. Although *DMD*-values may be used in identifying "satisfactory" control sections, this variable is, in general, not used to determine CRC undersealing requirements.

Step 5—Material Quantity Estimates

An average crack and joint frequency is determined for estimating purposes during Dynaflect testing of jointed pavements; typical frequencies range from five to eight joints and cracks per station. Bituminous material quantities are estimated on the basis of the single hole treatment at the rate of 15 gal per hole. CRC pavements that require undersealing are treated along the lane centerline at 8-ft intervals with 10 gal per hole.

Step 6—Undersealing

Undersealing is performed at considerable pressure (60 to 90 psi) to ensure uniform material distribution beneath the slab, and, as a result, the pavement starts to rise as soon as pumping begins. Slab motion is monitored with a sensitive deflection gauge developed at the Research and Training Center specifically for this purpose; use of the gauge is shown in Figure 3. Maximum slab uplift values of 1/4 in. for jointed and 1/8 in. for CRC pavements are specified in each undersealing contract. The difference in allowable motion for the two pavements is because jointed sections tend to settle somewhat when pumping stops whereas CRC pavements tend to rise slightly when pumping begins in the next hole. In either case, total injection time is limited regardless of pavement rise to minimize material loss due to blowouts through joints or cracks or at the shoulder. The time limits are 15 sec for jointed and 12 sec for CRC pavements *(3)*.

The effectiveness of the current single-hole injection method has been verified at numerous locations. Uniform distribution of the undersealing material can be observed whenever subsequent joint or crack repairs require slab removal. A uniform seam of the bituminous material has also been observed where edge drains have been installed along previously undersealed pavements *(4)*. Such a seam of material is visible in the photograph in Figure 4.

Step 7—Safety

Dynaflect testing is a slow-moving and potentially dangerous operation, particularly on high-volume roads. Thus appropriate safety precautions and traffic control measures are required to protect both the traveling public and testing personnel. A typical testing crew consists of the Dynaflect operator and two additional employees who follow in vehicles equipped with

FIGURE 3 Deflection gauge in use during undersealing of a section of CRC pavement.

arrow boards and signs for traffic control. Daily production rates for the crew average about 8 lane-miles.

The undersealing operation is also potentially hazardous, so contractors who perform the work are required to comply with all state signage and traffic control specifications. Additional safety precautions are recommended because of the potential

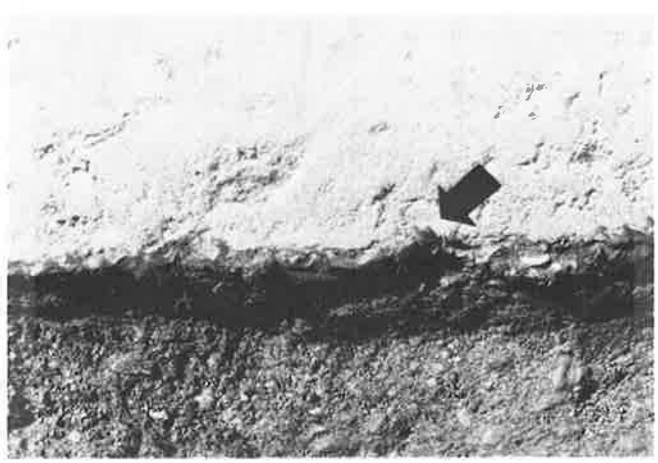

FIGURE 4 Seam of bituminous underseal exposed during installation of edge drain.

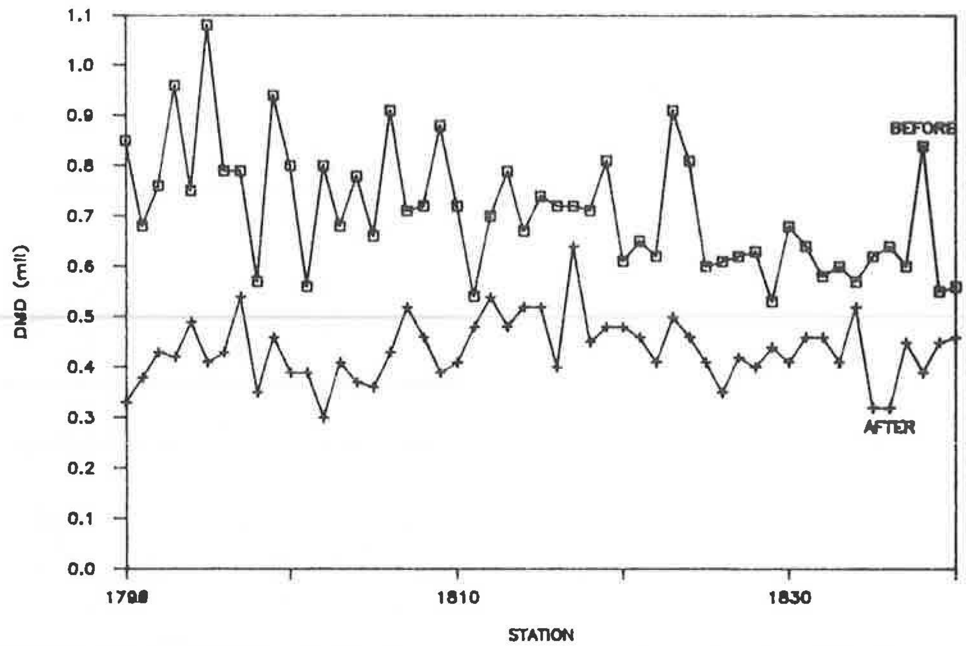

FIGURE 5 1983 IDOH void detection survey results: maximum Dynaflect deflections versus station before and after undersealing on US-30 between IN-23 and Queen Road (undersealing contract R-13899A).

hazard associated with handling the hot asphaltic material under high pressure.

RESULTS OF 1983 UNDERSEALING EFFORTS

Twelve jointed concrete pavement sections that had been undersealed during the 1983 construction season were retested with the Dynaflect to determine the extent of deflection improvement *(5)*. Graphic comparisons of the *DMD* data obtained before and after the undersealing of subsections in four of the twelve contracts are shown in Figures 5–8. Figures 2–7 show data representative of areas that showed substantial improvement after undersealing, and Figure 8 is a reminder of the statistical nature of the problem: the section from Station 200 to Station 246 showed little improvement after underseal-

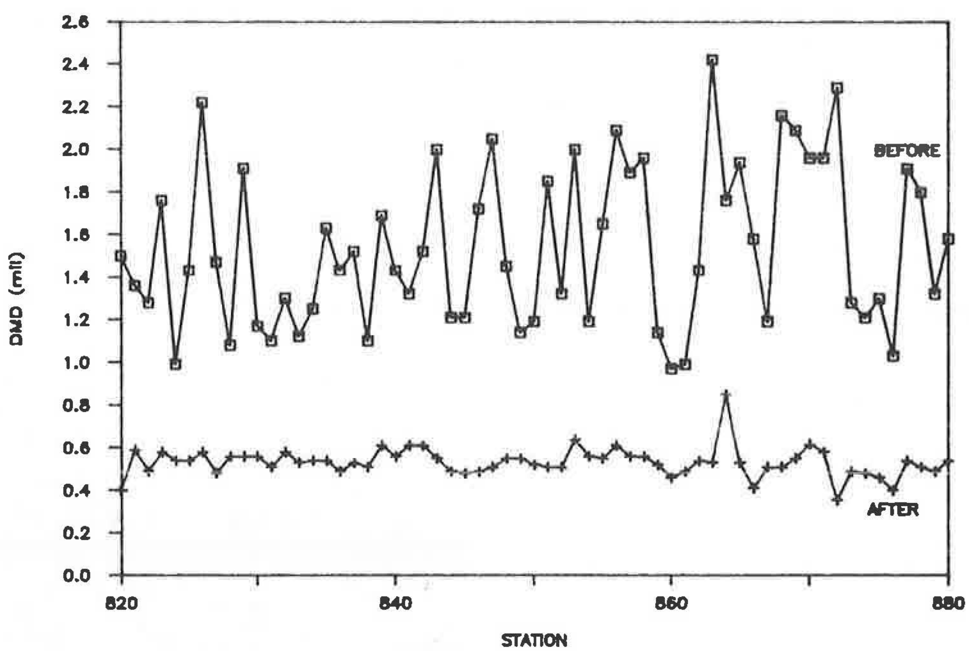

FIGURE 6 1983 IDOH void detection survey results: maximum Dynaflect deflections versus station before and after undersealing on I-69 from 2.3 mi east of IN-238 to 1.38 mi northeast of IN-38 (undersealing contract R-13947).

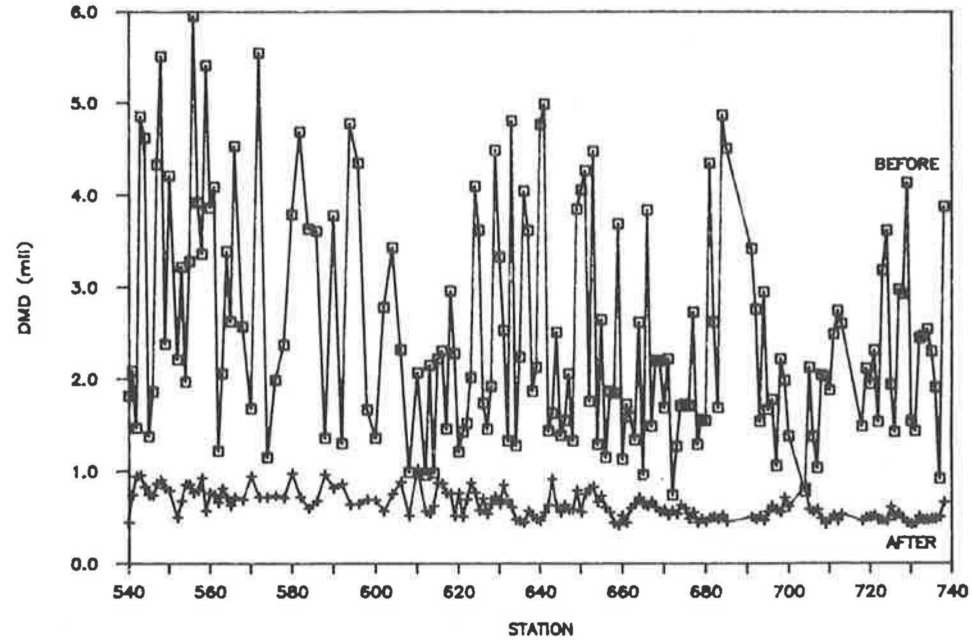

FIGURE 7 1983 IDOH void detection survey results: maximum Dynaflect deflections versus station before and after undersealing on I-69 from IN-18 to IN-124 (undersealing contract R-13773).

ing. In general, the most obvious features exhibited in these figures are the reduction in magnitude and variability of *DMD* deflections and the evidence of greater improvement in areas of higher initial *DMD*-values. A summary of mean *DMD* and standard deviation values determined for each of the twelve data sets is given in Table 1.

Figure 9, a comparison of the percentage improvement in *DMD* deflections versus initial *DMD* magnitudes, was prepared from the data obtained from all 12 sections. The percentage improvement was defined as

$$\text{Percentage improvement} = 100 * (DMD_i - DMD_a)/DMD_i$$

where DMD_i is the magnitude before undersealing and DMD_a is the magnitude after undersealing. It is evident from an inspection of Figure 9 that, in the majority of cases, an

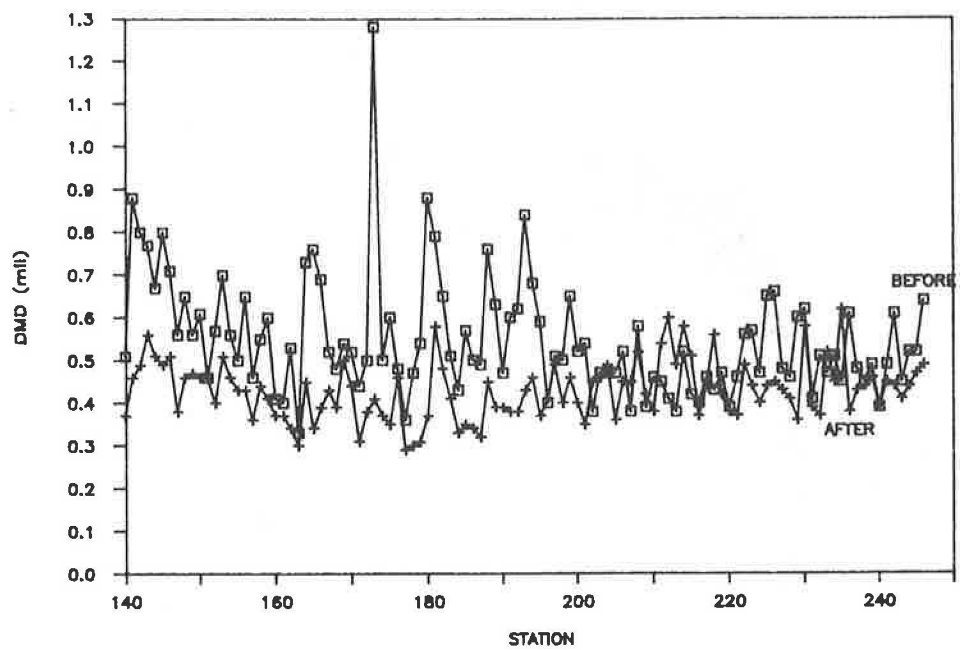

FIGURE 8 1983 IDOH void detection survey results: maximum Dynaflect deflections versus station before and after undersealing on I-69 from the West Fork of the White River to IN-332 (undersealing contract R-13903).

TABLE 1 SUMMARY OF 1983 DYNAFLECT DEFLECTION RESULTS

	Before Undersealing			After Undersealing		
Contract	Mean *DMD* (mil)	Standard Deviation (mil)	Coefficient of Variation (%)	Mean *DMD* (mil)	Standard Deviation (mil)	Coefficient of Variation (%)
R-13724	1.05	0.56	53	0.44	0.10	23
R-13724A	1.06	0.31	29	0.57	0.07	12
R-13773	2.51	1.21	48	0.64	0.15	23
R-13860	0.70	0.27	39	0.43	0.11	26
R-13899	0.62	0.11	18	0.49	0.07	14
R-13899A	0.71	0.12	17	0.43	0.07	16
R-13902	0.64	0.17	27	0.40	0.11	28
R-13903	0.55	0.14	25	0.43	0.07	16
R-13944	0.74	0.17	23	0.45	0.08	18
R-13947	1.54	0.38	25	0.54	0.06	11
R-13947A	1.17	0.31	26	0.52	0.06	12
R-13948	1.03	0.20	19	0.49	0.09	18

improvement of from 25 to 65 percent may be expected after undersealing. It is also evident that the effect of undersealing areas with DMD_i-values below about 0.50 or 0.60 mil is greatly reduced.

ECONOMIC BENEFITS

Use of the comprehensive Indiana testing and undersealing method has resulted in substantial savings by assuring more efficient allocation of the state's pavement rehabilitation resources. The result has been that, since implementation of the method, many additional miles of pavement are undersealed annually for the same or fewer relative dollars. These savings accrue from a reduction of both the number of joints and cracks treated and the volume of material injected at each location. Depending on the test results, undersealing requirements may vary from 30 to 100 percent of the joints and cracks in a contract section.

A summary of material savings realized on the 28 contract sections tested and undersealed during the 1985 construction season is given in Table 2. Based on a material cost of $300 per ton, the reduced material requirements represent a savings of approximately $12.2 million.

SUMMARY

The Indiana method, which is based on a Dynaflect deflection survey and a controlled undersealing procedure, has been

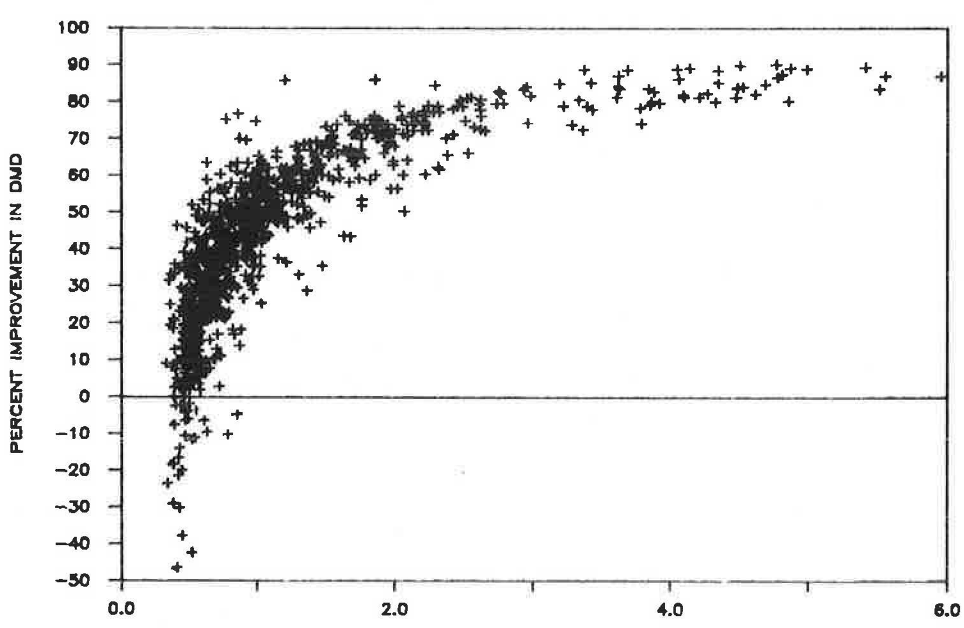

FIGURE 9 1983 IDOH void detection survey results: percentage improvement in *DMD* as a function of initial *DMD*.

TABLE 2 COMPARISON OF 1985 UNDERSEALING MATERIAL REQUIREMENTS WITH ESTIMATED REQUIREMENTS OF THE PRE-1980 UNDERSEALING METHOD, ADAPTED FROM LOVE *(4)*

Highway	Contract	Location	Comprehensive Method (tons)	Pre-1980 Method (tons)	Savings (tons)
US-31	R-15366	3.6 mi E of US-31 to IN-19	398.3	1,393.2	994.9
I-465	R-15414	I-565 to Fall Creek Pkway	754	6,907.6	6,153.6
US-41	—[a]	Margaret Ave. to Maple Ave.	150	703.7	553.7
US-30	—[a]	4.1 mi E of US-24 to OH line	141	1,632.7	1,491.7
US-31	—[a]	1 mi S of IN-25 to IN-110	0	2,199.1	2,199.1
IN-49	—[a]	Toll road to 3.9 mi N	293.7	837.2	543.5
US-24	—[a]	US-31 to IN-13	255	1,046.8	791.8
I-70	—[a]	Emerson to Shadeland	0	450.8	450.8
IN-3	R-15309	3.05 mi S of IN-244 to US-52	325.2	961.3	636.1
IN-9	R-15157	IN-14 to IN-205	0	527.8	527.8
I-65	R-15423	Greenwood to I-465	0	1,231.5	1,231.5
US-40	—[a]	Centerville to 0.73 mi W of US-27	38.7	726.4	688.6
IN-9	—[a]	12.23 mi S of US-24 to 6 mi S of US-24	194	1,029.6	835.6
I-65	—[a]	IN-56 to US-50	648.4	3,430.3	2,781.9
I-65	—[a]	Ohio River to IN-160	89	3,195.4	3,106.4
US-31	—[a]	Mills to Southern	48.7	129.9	81.2
US-52	R-14721	US-52 to Stockwell Rd.	929	2,059.2	1,130.2
US-30	R-15500	IN-109 to I-69	515	2,885.5	2,370.5
IN-67	R-15502	0.5 mi N of IN-239 to IN-144	252	2,532.3	2,280.3
I-70	R-15320	I-465 to Harding St.	237	1,561.9	1,324.9
I-69	R-15245	I-465 to Sand Creek	96.5	1,955.5	1,859
US-24	R-15141	US-24 bypass Fort Wayne	212	1,166.2	954.2
I-65	R-15544	College Ave. to North Western	138	1,539.4	1,401.4
IN-64	R-15313	US-231 to IN-145	230.5	1,276.2	1,045.7
US-35	R-15314	2.01 mi S of IN-18 to IN-18	45	169.9	124.9
I-465	R-15527	1.88 mi W of US-231 to 1.22 mi W of US-431	16.2	218.8	202.6
IN-3	—[a]	I-69 to Decalb Co. Line	256.4	1,935.2	1,678.8
I-465	—[a]	56th St. to I-65 S	121.5	3,443.6	3,322.1
Total			6,384.2	47,147.0	40,762.8

[a]Contract numbers not available at time of recording.

shown to be both practical and economical. The method is applicable to both virgin and overlaid jointed and CRC pavements with only minor procedural modifications and has been used successfully throughout the state to reduce the cost of undersealing unstable concrete pavements.

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Development of a Procedure for Assessing Routine Maintenance Needs of Highways

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In this paper is described a procedure that can be used to estimate routine maintenance work loads by highway section for a coming year or season. Although the approach can also be extended for use in maintenance budget planning, the primary area of application of the proposed procedure is in determining the amount of maintenance work that is to be undertaken on what highway sections within a subdistrict subject to the constraint of a given maintenance budget. The procedure is based on periodic surveys of highway distress by unit foremen and subsequent use of a set of quantity standards, termed "present quantity standards." These standards were developed by relating the foremen's subjective ratings of road conditions to objective field measurements of distress and subsequently transforming the subjective ratings to expected work loads. A statistical regression analysis was used to develop the necessary relationships. The field data were collected from 18 maintenance units in Indiana.

One of the most important functions of a maintenance management system is to estimate the amount of maintenance work to be performed on various highway sections within a maintenance unit during a coming year or season. For the state highway system in Indiana, the budgeting for routine maintenance work is established primarily by subdistrict foremen on the basis of historical quantity standards and judgment *(1)*. The procedure, used in most states, is based on Roy Jorgensen's work in the 1960s *(2,3)*. However, this historical-empirical approach may not provide an assessment of actual needs by specific highway sections for use in scheduling activities in the field.

PROPOSED SYSTEM

In the present study a system is proposed for assessing routine maintenance work load on the basis of a condition survey of roadways by unit foremen. It is believed that the proposed system can provide a tool to assist in the assessment of work loads by highway section. The proposed procedure can have several added benefits. Subdistricts and districts will be able to have systematically gathered and uniformly defined maintenance needs data. Maintenance management at all levels can thus have another tool to check the maintenance levels of service throughout the state and to help keep maintenance policies consistent.

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Maintenance Management Systems

The present version of the maintenance management systems in most states is based primarily on the development of appropriate standards. These standards are then used to control and plan various maintenance activities:

1. Quality standards are used to represent maintenance levels of service.
2. Quantity standards are the means by which inventory units are converted into work load. For example, if a certain network has 10 mi of bituminous road, multiplying this by the quantity standard for shallow patching, such as 2 tons per mile of bituminous road, will lead to the expected amount of shallow patching: 20 tons. Quantity standards are developed primarily from historical data as well as from input from the unit foremen. These standards are averages of past requirements per unit of inventory for each maintenance activity.
3. Performance standards help to translate expected work load per activity to man-hours, material, and dollars per activity. They provide the average requirement of manpower and materials to accomplish one unit of a maintenance activity. Thus, when the work load per activity is known, these quantities can be multiplied by their respective performance standards to arrive at the requirements for labor and materials.

The Indiana Department of Highways (IDOH) *Management System Procedures Manual (4)* and the *Field Operations Handbook for Foremen (5)* provide a good insight into the maintenance management system in use in Indiana. The procedure is based on the three sets of standards described earlier.

Condition Evaluation Procedures

Present condition survey procedures were developed mainly for pavement management systems, and they are directed to decisions regarding rehabilitation needs. However, in the present study it was necessary to develop a survey procedure that could identify conditions that trigger routine maintenance needs. The proposed procedure is to have unit foremen conduct a visual condition survey on a periodic basis.

DEVELOPMENT OF THE PROPOSED APPROACH AND DESIGN OF EXPERIMENT

Development of the Condition Survey Form

A simple survey form was developed on the basis of current procedures and consultation with the unit foremen and sub-

TABLE 1 ROUTINE MAINTENANCE ACTIVITIES INCLUDED IN THE STUDY

Pavement		Unpaved Shoulders		Drainage	
No.	Activity	No.	Activity	No.	Activity
201	Shallow patching	210	Spot repair of unpaved shoulders	231	Clean and reshape ditches
202	Deep patching	211	Blading shoulders	234	Motor patrol ditching
203	Premix leveling	212	Clipping unpaved shoulders		
204	Full-width shoulder seal	213	Reconditioning unpaved shoulders		
205	Seal coating				
206	Sealing longitudinal cracks and joints				
207	Sealing cracks				

district personnel. The selection of maintenance activities and types of distress to be included in the survey procedure was based on maintenance personnel's opinion and information available in the literature on highway maintenance management. Table 1 gives the list of maintenance activities included in the study, and the types of highway distress considered in the survey are given in Table 2.

TABLE 2 TYPES OF HIGHWAY DISTRESS INCLUDED IN THE SURVEY

Flexible Pavements	Rigid Pavements
Blowups	Blowups
Bumps	Bumps
Depressions	Condition of longitudinal joints
Ditch condition	Condition of transverse joints
Linear cracks	Ditch condition
Potholes	Linear cracks
Raveling	Potholes
Rutting	Raveling in bituminous shoulder
Shoulder buildup	Shoulder buildup
Shoulder drop-off	Shoulder drop-off
Shoulder potholes	Shoulder potholes
Surface failures	Spalling
	Surface failures

Design of Experiment

The proposed approach was tested in the field to determine its validity and accuracy as well as to check whether the survey form represented actual typical conditions of the roadways. The work elements included

1. Collection of information on the physical condition of the highway by means of a visual inspection by unit foremen. The type of visual inspection was the same as that currently used by the IDOH. The units were selected in a stratified random sampling. The unit foremen were asked to generate two types of data: a subjective opinion about the degree of several deficiency conditions in the roadway stretch being analyzed and an estimate of the expected amount of work in the selected maintenance activities currently needed, based on the condition of the roadway.
2. Objective measurements of different deficiency conditions by the research team on the same highway stretches surveyed by the unit foremen.
3. Statistical correlation and analysis of the data collected in Steps 1 and 2.
4. Development of the criteria that would relate the unit foremen's evaluation of a deficiency condition to a certain level of routine maintenance activity.
5. Analysis of the variability of the subjective opinions about the roadway condition. This analysis can then assist in improving the consistency of future maintenance decisions.

The forms used requested information on the roadway condition and estimated maintenance needs. Foremen were required to estimate the work load so that the information could be used to analyze the validity of the proposed approach. It is not proposed to use this part of the survey form during actual implementation of the procedure.

Statistical Selection of Maintenance Units Surveyed

A stratified random sampling scheme was used in the study. A stratified random scheme is a restricted randomization design in which the experimental units are first sorted into homogeneous groups or blocks and then the required number of experimental units is randomly selected within each group *(6)*.

The northern, central, and southern parts of the state of Indiana were considered blocks from which the units to be surveyed were selected. This made it possible to take into account variations in climate and regional maintenance practices during the analysis of the validity of the proposed approach. Three subdistricts were randomly selected in each of these three regions. Within each of these subdistricts, two randomly selected maintenance units were surveyed. This made it possible to analyze the variations associated with both unit foremen and subdistricts when assessing the accuracy of the proposed condition survey method. A total of 18 maintenance units were included in the study. The survey covered asphalt and concrete highways in both the Interstate and the state highway systems. A total of 965 lane-miles were surveyed. The forms used to conduct the foremen's survey are shown in Figures 1 and 2.

Objective Measurement of Highway Distress

The highway stretches surveyed by the unit foremen were also surveyed by the research team and the types of highway distress observed were physically measured. This measurement took place within no more than 2 days of the foremen's survey. Every highway stretch that a foreman had evaluated was subsequently evaluated by objectively measuring its distress.

DISTRICT________ HIGHWAY S US IS No:______

SUBDISTRICT________ FROM________

UNIT NO.________ TO________

DATE________ TRAFFIC LOW MED HIGH

DIRECTION N S E W

ASPHALT PAVEMENTS						
TRAFFIC LANES AND PAVED SHOULDERS						
M	S	F	N	SLIGHT	POTHOLES	SHALLOW PATCHING .... tons
M	S	F	N	MODERATE		
M	S	F	N	SEVERE		
M	S	F	N	SLIGHT	CRACKS	CRACK SEALING .... gals
M	S	F	N	MODERATE		
M	S	F	N	SEVERE		FULL WIDTH SHOULDER SEAL ...... ft. miles
M	S	F	N	SLIGHT	RAVELING	
M	S	F	N	MODERATE		SEAL COATING ...... lane miles
M	S	F	N	SEVERE		
M	S	F	N	BLOW UPS, BUMPS AND SURFACE FAILURES		DEEP PATCHING .... tons
M	S	F	N			
M	S	F	N			
M	S	F	N	SLIGHT	RUTTING, DIPS	LEVELING .... tons
M	S	F	N	MODERATE		
M	S	F	N	SEVERE		
UNPAVED SHOULDERS						
M	S	F	N	SLIGHT	BUILD-UP	CLIPPING .... shldr. miles
M	S	F	N	MODERATE		
M	S	F	N	SEVERE		
M	S	F	N	SLIGHT	POTHOLES	SPOT REPAIR (210) .... tons of agg.
M	S	F	N	MODERATE		
M	S	F	N	SEVERE		
M	S	F	N	SLIGHT	DROP-OFF	BLADING .... shldr. miles
M	S	F	N	MODERATE		RECONDTING .... shldr. miles
M	S	F	N	SEVERE		
DRAINAGE						
P	F	G			DITCHES	DITCHING (231) ...... linear ft
						MOTOR PATROL DITCHING (234) .... ditch miles

FIGURE 1 Asphalt pavement condition survey form used by the foremen in the study.

Because the measurement took place soon after the foremen's survey, the possibility of occurrence of any changes in the highway condition between the two evaluations was minimized. The form used to record the physical measurements of distress is shown in Figure 3.

ANALYSIS OF VALIDITY OF PROPOSED APPROACH

The subjective condition rating data were converted into a numerical scale so that quantitative statistical analysis methods could be used. A point estimation technique was applied for the conversion of the subjective category scale used during the field survey to a 0 to 10 numerical scale.

To analyze the data gathered, regression analyses were performed. Table 3 gives a summary of the results obtained. It shows the significance of the proposed approach in explaining the variability of maintenance work load for eight of the nine maintenance activities considered. The lack of significance in the case of Sealing Longitudinal Cracks and Joints can be attributed to the small sample size.

It can be seen in Table 3 that maintenance subdistricts showed a significant influence on the estimation of the work load of Shallow Patching, Crack Sealing, and Premix Leveling at a level of significance of 0.05. Individual estimator's influences were found significant in assessing the needs for Spot Repair of Unpaved Shoulders, Blading Unpaved Shoulders, and Cleaning and Reshaping Ditches. These results suggest that the amount of work in Spot Repair of Unpaved Shoulders, Blading Unpaved Shoulders, and Cleaning and Reshaping Ditches is particularly influenced by the personal judgment of unit foremen, whereas the amounts of Shallow Patching, Crack Sealing, and Premix Leveling are more subject to regional differences in maintenance materials, practices, or standards. The influences of subdistricts and foremen should be further studied in order to achieve consistency in maintenance needs assessment.

DISTRICT________ HIGHWAY [S | US | IS] No. ________

SUBDISTRICT________ FROM________

UNIT NO.________ TO________

DATE________ TRAFFIC [LOW | MED | HIGH]

DIRECTION [N | S | E | W]

CONCRETE PAVEMENTS			
TRAFFIC LANES AND PAVED SHOULDERS			
M S F N	SLIGHT	POTHOLES	SHALLOW PATCHING .... tons
M S F N	MODERATE		
M S F N	SEVERE		
M S F N	BLOW UPS, BUMPS AND SURFACE FAILURES		DEEP PATCHING .... tons
M S F N			
M S F N			
P F G	LONGITUD. JOINTS		SEALING LONG. CRACKS & JOINTS ..... linear miles of cracks & joints
P F G	TRANSVERSE JOINTS		CRACK SEALING .... gals.
M S F N	SLIGHT	CRACKS	
M S F N	MODERATE		FULL WIDTH SHOULDER SEAL ...... ft miles
M S F N	SEVERE		
M S F N	RAVELING IN BITUMINOUS SHLDR		
UNPAVED SHOULDERS			
M S F N	SLIGHT	BUILD–UP	CLIPPING .... shldr. miles
M S F N	MODERATE		
M S F N	SEVERE		
M S F N	SLIGHT	POTHOLES	SPOT REPAIR .... tons of agg.
M S F N	MODERATE		
M S F N	SEVERE		
M S F N	SLIGHT	DROP–OFF	BLADING .... shldr. miles
M S F N	MODERATE		RECONDTNG .... shldr. miles
M S F N	SEVERE		
DRAINAGE			
P F G	DITCHES		DITCHING (231) ...... linear ft
			MOTOR PATROL DITCHING (234) .... ditch miles

FIGURE 2 Concrete pavement condition survey form used by the foremen in the study.

Work Load and Subjective Evaluation of Distress

A set of regression analyses was performed to relate the routine maintenance work load to the subjective evaluation of distress by unit foremen. The purposes of these analyses were

1. To develop models that could be used to estimate routine maintenance work loads on the basis of subjective evaluation of roadway distress,
2. To form the basis of the calculation of "present" quantity standards, and
3. To learn how much of the variability of estimated maintenance work loads can be explained by the foremen's survey.

These points were addressed by a stepwise regression procedure that gives "best" models for each of the analyzed maintenance activities. The following model was adopted:

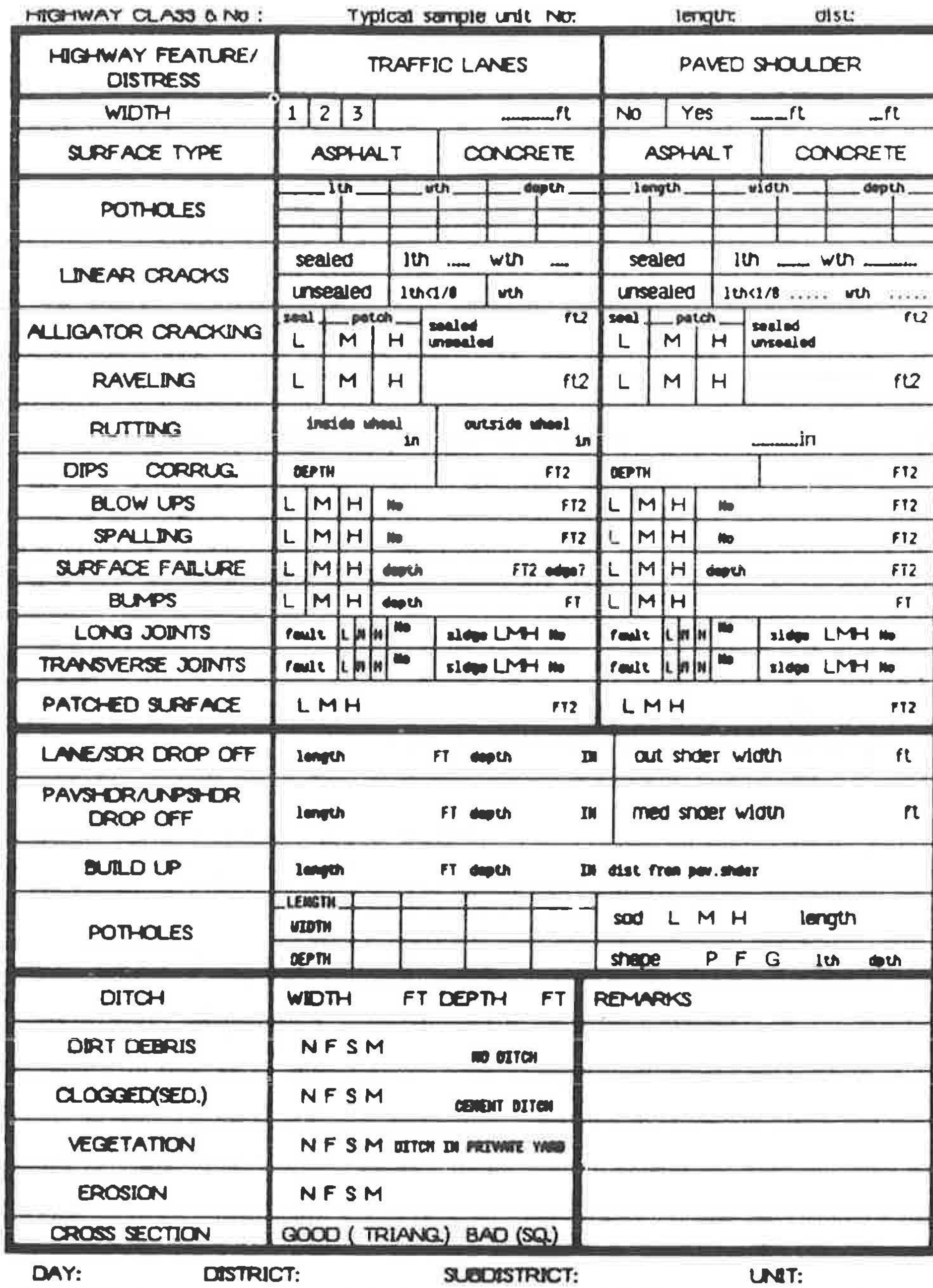
$$y_i = a + \sum_{j=1}^{n_i} b_j X_{ij} \quad (1)$$

where

y_i = square root of expected work load per activity per lane-mile, shoulder-mile, or ditch-mile;
a = constant;
b_j = regression parameters, $j = 1,2, \ldots, n_i$; and
X_{ij} = subjectively rated distress (pothole frequency, pothole size, etc).

The variables listed in Table 4 were included in Equation 1 in the process of developing models to predict work load per activity. The "best" models arrived at are given in Table 5.

The values of the coefficients of determination (R^2) represent the proportion of the variability of estimated work loads that can be explained by foremen's evaluation of distress. Except



HIGHWAY CLASS & No : Typical sample unit No: length: dist:

HIGHWAY FEATURE/ DISTRESS	TRAFFIC LANES	PAVED SHOULDER
WIDTH	1 2 3 ____ft	No Yes ____ft __ft
SURFACE TYPE	ASPHALT CONCRETE	ASPHALT CONCRETE
POTHOLES	lth wth depth	length width depth
LINEAR CRACKS	sealed lth ___ wth ___ / unsealed lth<1/8 wth	sealed lth ___ wth ___ / unsealed lth<1/8 ..... wth .....
ALLIGATOR CRACKING	seal patch sealed unsealed ft2 L M H	seal patch sealed unsealed ft2 L M H
RAVELING	L M H ft2	L M H ft2
RUTTING	inside wheel in / outside wheel in	____in
DIPS CORRUG.	DEPTH FT2	DEPTH FT2
BLOW UPS	L M H No FT2	L M H No FT2
SPALLING	L M H No FT2	L M H No FT2
SURFACE FAILURE	L M H depth FT2 edge?	L M H depth FT2
BUMPS	L M H depth FT	L M H FT
LONG JOINTS	fault L M H No sldge LMH No	fault L M H No sldge LMH No
TRANSVERSE JOINTS	fault L M H No sldge LMH No	fault L M H No sldge LMH No
PATCHED SURFACE	L M H FT2	L M H FT2
LANE/SDR DROP OFF	length FT depth IN	out shder width ft
PAVSHDR/UNPSHDR DROP OFF	length FT depth IN	med shder width ft
BUILD UP	length FT depth IN dist from pav.shder	
POTHOLES	LENGTH / WIDTH / DEPTH	sod L M H length / shape P F G lth dpth
DITCH	WIDTH FT DEPTH FT	REMARKS
DIRT DEBRIS	N F S M NO DITCH	
CLOGGED(SED.)	N F S M CEMENT DITCH	
VEGETATION	N F S M DITCH IN PRIVATE YARD	
EROSION	N F S M	
CROSS SECTION	GOOD (TRIANG.) BAD (SQ.)	

DAY: DISTRICT: SUBDISTRICT: UNIT:

FIGURE 3 Form used to record typical distress during field measurements.

for Crack Sealing, Premix Leveling, and Blading Shoulders, the R^2-values are reasonable. Some factors that might have lowered the R^2-values obtained are *(a)* the lack of full understanding on the part of some foremen of the meaning of some types of distress, such as raveling, when rating the roads; *(b)* the lack of consistency in the speed at which the foremen evaluated the roads (10 to 55 mph); *(c)* some foremen might have rated the extent of certain types of distress influenced by "nontypical" spots rather than on the basis of the overall extent of those types of distress over the highway stretches; *(d)* maintenance standards for certain activities may be based on usage and experience rather than on established maintenance levels of service (for example, unpaved shoulders may be clipped once every few years instead of being clipped whenever the buildup is greater than a determined height); *(e)* some of the types of distress evaluated trigger two or more maintenance options; for example, bumps may trigger either "Bumps Burning" or "Deep Patching," depending on severity; and *(f)* altogether different maintenance activities may be triggered by a given extent of a particular type of distress (for example, raveling can trigger sealing, patching, or major maintenance, depending on the extent and severity of raveling). It is believed that many of these items can be improved by training foremen and that the resulting future R^2-values can thus be increased.

A note of caution should be given. The models developed in this section are statistical in nature. No mechanistic or cause-

TABLE 3 TESTS FOR THE SIGNIFICANCE OF THE APPROACH AND OF THE EFFECTS OF SUBDISTRICTS AND INDIVIDUAL ESTIMATORS

	Approach (related "assessed" distress)			Subdistrict Effect			Individual Estimator's Effect		
Maintenance Activity	Significant at $\alpha = 0.05$	F	α	Significant at $\alpha = 0.05$	F	α	Significant at $\alpha = 0.05$	F	α
Shallow patching	Yes	6.98603 (4,41)[a]	<0.001	Yes	2.9448 (8,50)	0.01–0.025	No	1.2666 (9,41)	>0.1
Crack sealing	Yes	4.6951 (4,41)	0.001–0.005	Yes	2.5729 (8,50)	0.01–0.025	No	1.7119 (9,41)	>0.1
Deep patching	Yes	2.9663 (7,38)	0.01–0.025	No	0.8495 (8,47)	>0.1	No	1.0688 (9,38)	>0.1
Premix leveling	Yes	2.9248 (3,32)	0.01–0.025	Yes	2.3576 (8,41)	0.025–0.05	No	1.7193 (9,32)	>0.1
Sealing longitudinal cracks and joints	No	49.3049[b] (3,1)	>0.1	No	3.5725 (4,2)	>0.1	No	4.3236 (1,1)	>0.1
Clipping unpaved shoulders	Yes	25.8952 (2,43)	<0.001	No	1.6044 (8,52)	>0.1	No	1.3799 (9,43)	>0.1
Spot repair of unpaved shoulders	Yes	5.9417 (4,41)	<0.001	No	1.9063 (8,50)	0.05–0.1	Yes (9,41)	2.4455 0.05	0.025–
Blading unpaved shoulders	Yes	4.2549 (4,41)	0.005–0.01	No	1.7162 (8,50)	>0.1	Yes	4.0648 (9,41)	0.001–0.005
Cleaning and reshaping ditches	Yes	26.7146 (1,44)	<0.001	No	1.4627 (8,53)	>0.1	Yes	3.782 (9,44)	0.001–0.005

[a]Degrees of freedom are in parentheses.
[b]Sample size is much smaller in this case and therefore the power of the tests is lower.

and-effect relationship between work load and "assessed" distress was established.

Analysis of the Field Survey Data

A regression of maintenance work load per activity on related measured distresses was done. The objective was to highlight major types of distress that need to be included in the survey form proposed for implementation. The extent of patched surface was found to be the only additional significant highway feature that contributed to the explanation of the variation in estimated needs of Premix Leveling.

Proposed Quantity Standards

On the basis of the models developed in this study "present" quantity standards (QS) were computed for various combinations of highway distress frequency and severity. As an illustration, the following example can be considered. The QS for Shallow Patching in roadways assessed as having "Many" "Slight" potholes was calculated using the model for Shallow Patching. In that model, expected shallow patching per lane-mile is a function of the assessed frequency (X_1) and severity of potholes (X_2). The model was solved with the numerical values associated with the categories "Many" and "Slight" potholes, 8.01 and 1.79, respectively. The resulting QS-value can thus be computed as 1.20 tons per lane-mile. Similar computations were done for other activities under various combinations of frequency and severity of distress. The resulting QS-values are shown in Figure 4.

The procedure proposed for use in estimating future routine maintenance work loads appears to be conceptually sound; it involves an assessment of maintenance needs based on present needs (evaluation of types of distress that trigger those needs) rather than past experience or arbitrary guesses.

PROPOSED PLAN FOR IMPLEMENTATION

The steps that could be followed to implement the proposed approach are outlined next.

1. Unit foremen would perform the condition survey in early fall and early spring each year. Condition data would be recorded for each highway stretch within the boundaries of a maintenance unit. One form should be filled out for each highway stretch. Figures 5 and 6 show the proposed forms for asphalt and concrete pavements. These forms are modified versions of the forms used in the study. Unlike the forms used in the study, the proposed forms include "patched area" as one of the distress indicators, and a three-category scale is used for the frequency of distress. The analysis conducted in the study indicated that these changes would improve the survey results.
2. Unit foremen would drive along the entire stretch of a roadway at a reduced speed of about 30 mph before making their ratings. It should be noted that the proposed survey was designed to be fast enough that an entire highway stretch could be surveyed without resorting to sampling sections so the foremen could base their judgment on the overall condition of

TABLE 4 VARIABLES CONSIDERED IN THE DEVELOPMENT OF REGRESSION MODELS

Maintenance Activity	Types of "Assessed" Distress Considered
Shallow patching	Frequency of potholes (X_1) Severity of potholes (X_2) Frequency of cracks (X_3) Severity of cracks (X_4)
Crack sealing	Frequency of cracks (X_3) Severity of cracks (X_4) Frequency of raveling (X_5) Severity of raveling (X_6)
Deep patching	Frequency of potholes (X_1) Severity of potholes (X_2) Frequency of cracks (X_3) Severity of cracks (X_4) Frequency of raveling (X_5) Severity of raveling (X_6) Frequency of bumps, blowups, and surface failures (X_7)
Premix leveling	Frequency of ruts and dips (X_8) Severity of ruts and dips (X_9) Frequency of bumps, blowups, and surface failures (X_7)
Sealing longitudinal cracks and joints	Frequency of cracks (X_3) Severity of cracks (X_4) Condition of longitudinal joints (X_{10})
Clipping unpaved shoulders	Frequency of buildups (X_{11}) Severity of buildups (X_{12})
Spot repair of unpaved shoulders	Frequency of potholes in unpaved shoulder (X_{13}) Severity of potholes in unpaved shoulder (X_{14}) Frequency of drop-off (X_{15}) Severity of drop-off (X_{16})
Blading shoulders	Frequency of potholes in unpaved shoulder (X_{13}) Severity of potholes in unpaved shoulder (X_{14}) Frequency of drop-off (X_{15}) Severity of drop-off (X_{16})
Cleaning and reshaping ditches	Condition of roadside ditches (X_{17})

the stretch. Only one combination of frequency and severity of a particular deficiency condition should be selected. For example, if a unit foreman thinks that there is extensive cracking of low severity in a highway stretch, he will mark the cell corresponding to "Many" "Slight" cracks.

3. An estimation of maintenance work load for each activity and for each highway stretch can be made by matching the condition data recorded on the forms shown in Figures 5 and 6 during the spring survey with the appropriate "present" quantity standards shown in Figure 4. These quantity standards are functions of the "assessed" levels of frequency and severity of distress. For example, when a stretch has "Many" "Moderate" potholes, 2.05 tons of Shallow Patching for each lane-mile of the stretch would be considered. By multiplying the corresponding "present" quantity standards by the number of lane-miles, shoulder-miles, or ditch-miles of the highway stretch, various maintenance work loads for each highway stretch would be obtained. The maintenance needs for any maintenance unit, subdistrict, district, or the state can be computed by adding the needs for each road stretch within that area. The estimated work loads by highway sections can then be used to determine the actual work loads within a budget constraint.

4. The aggregation of the evaluation data for each maintenance subdistrict would provide a periodic indication of the overall condition of the highways within the subdistrict. These data can be used to check the effectiveness of different maintenance policies related to field work.

SUMMARY AND CONCLUSIONS

The principal objective of this study was to develop an approach that could be used primarily to determine how much of a routine maintenance activity is to be performed on a highway section during a given time period subject to a given budgetary constraint. This approach is based on the subjective rating of highway distress by maintenance unit foremen. Rou-

TABLE 5 MODELS FOR ESTIMATION OF WORK LOAD

Maintenance Activity	Best-Suited Models (estimated regression coefficient)	R^2 (%)
Shallow patching	$y'^a = 0.157 + 0.09253\ X_1 + 0.10865\ X_2$	37.15
Crack sealing	$y' = 5.261 + 1.03834\ X_4$	21.46
Deep patching	$y' = -\ 0.362 + 0.11716\ X_1 + 0.15267\ X_7$	30.66
Premix leveling	$y' = -\ 0.187 + 0.46177\ X_8$	16.21
Sealing longitudinal cracks and joints	No significant model was developed because of lack of sufficient sample size	–
Clipping unpaved shoulders	$y' = -\ 0.067 + 0.06746\ X_{11} + 0.05793\ X_{12}$	55.43
Spot repair of unpaved shoulders	$y' = -\ 0.004 + 0.21536\ X_{13} + 0.26212\ X_{16}$	31.30
Blading shoulders	$y' = 0.239 + 0.08648\ X_{13}$	12.71
Cleaning and reshaping ditches	$y' = 34.845 - 4.26425\ X_{17}$	47.98

NOTE: The variables $X_1, X_2, \ldots, X_{17}$ are defined in Table 4.

[a] y' = y transformed = square root of expected work load in 6 months per lane-mile, shoulder-mile, or ditch-mile.

Premix Leveling

(Tons per Lane Mile)

"Assessed" Frequency of Ruts and Dips

N	0.02
S	2.50
M	12.30

Crack Sealing

(Gallons per Lane Mile)

"Assessed" Severity of Cracks

Sl	7.0
Mo	10.0
Se	14.0

Clipping Unpaved Shdrs.

(Shdr. Miles per Shdr. Mile)

"Assessed" Severity of Buildups	"Assessed" Frequency of Buildups: N	S	M
Sl	0.01	0.10	0.33
Mo	0.07	0.25	0.50
Se	0.20	0.45	0.90

Deep Patching

(tons per Lane Mile)

"Assessed" Bumps, Blow-Ups and Surface Failure Frequency	"Assessed" Pothole Frequency: N	S	M
N	0.0	0.04	0.50
S	0.10	0.50	1.30
M	0.90	1.70	3.25

Spot Repair Unpaved Shdrs.

(Tons per Shdr. Mile)

"Assessed" Severity of Dropoff	"Assessed" Frequency of Potholes in Unpaved Shdr: N	S	M
Sl	0.40	1.70	4.80
Mo	2.00	4.45	9.10
Se	5.10	8.60	14.70

Blading Shdrs.

(Shdr. Miles per Shdr. Mile)

"Assessed" Frequency of Potholes in Unpaved Shdrs

N	0.10
S	0.30
M	0.90

Shallow Patching

(Tons per Lane Mile)

"Assessed" Pothold Severity	"Assessed" Pothole Frequency: N	S	M
Sl	0.20	0.50	1.20
Mo	0.60	1.10	2.10
Se	1.20	1.90	3.10

Clean and Reshape Ditches

(Ft per Ditch Mile)

"Assessed" Condition of Roadside Ditch

P	693.0
F	190.0
G	2.0

FIGURE 4 Proposed present quality standards.

DISTRICT________ HIGHWAY S | US | IS No.________
SUBDISTRICT________ FROM________
UNIT NO.________ TO________
DATE________ TRAFFIC LOW | MED | HIGH
DIRECTION N | S | E | W

ASPHALT PAVEMENTS				
TRAFFIC LANES AND PAVED SHOULDERS				
M	S	N	SLIGHT	POTHOLES
M	S	N	MODERATE	
M	S	N	SEVERE	
M	S	N	SLIGHT	CRACKS
M	S	N	MODERATE	
M	S	N	SEVERE	
M	S	N	SLIGHT	RAVELING
M	S	N	MODERATE	
M	S	N	SEVERE	
M	S	N	BLOW UPS, BUMPS AND SURFACE FAILURES	
M	S	N	SLIGHT	RUTTING, DIPS
M	S	N	MODERATE	
M	S	N	SEVERE	
M	S	N	SLIGHT	PATCHED SURFACE
M	S	N	MODERATE	
M	S	N	SEVERE	
UNPAVED SHOULDERS				
M	S	N	SLIGHT	BUILD-UP
M	S	N	MODERATE	
M	S	N	SEVERE	
M	S	N	SLIGHT	POTHOLES
M	S	N	MODERATE	
M	S	N	SEVERE	
M	S	N	SLIGHT	DROP-OFF
M	S	N	MODERATE	
M	S	N	SEVERE	
DRAINAGE				
P	F	G	DITCHES	

FIGURE 5 Asphalt pavement form proposed for implementation.

DISTRICT________ HIGHWAY S | US | IS No.________
SUBDISTRICT________ FROM________
UNIT NO.________ TO________
DATE________ TRAFFIC LOW | MED | HIGH
DIRECTION N | S | E | W

CONCRETE PAVEMENTS				
TRAFFIC LANES AND PAVED SHOULDERS				
M	S	N	SLIGHT	POTHOLES
M	S	N	MODERATE	
M	S	N	SEVERE	
M	S	N	BLOW UPS, SPALLING, BUMPS AND SURFACE FAILURES	
P	F	G	LONGITUD. JOINTS	
P	F	G	TRANSVERSE JOINTS	
M	S	N	SLIGHT	CRACKS
M	S	N	MODERATE	
M	S	N	SEVERE	
M	S	N	RAVELING IN BITUMINOUS SHLDR	
UNPAVED SHOULDERS				
M	S	N	SLIGHT	BUILD-UP
M	S	N	MODERATE	
M	S	N	SEVERE	
M	S	N	SLIGHT	POTHOLES
M	S	N	MODERATE	
M	S	N	SEVERE	
M	S	N	SLIGHT	DROP-OFF
M	S	N	MODERATE	
M	S	N	SEVERE	
DRAINAGE				
P	F	G	DITCHES	

FIGURE 6 Concrete pavement form proposed for implementation.

tine maintenance needs are connected to their immediate cause, highway deficiencies. It is envisioned that the implementation of this approach would give a more structured approach to maintenance planning because estimation of maintenance needs would be based on present needs rather than historical averages or arbitrary guesses.

This study developed both the methodology to perform the proposed foremen's surveys and the criteria to relate the subjective data obtained to certain levels of routine maintenance activities. Regression analyses allowed the development of estimation models for expected work load based on foremen's subjective evaluation of distress. Finally, the concept of "present" quantity standards was introduced. It should be noted, however, that before the procedure can be implemented, further work is necessary to establish increased consistency in foremen's evaluation of distress conditions and the subsequent estimation of work loads.

The use of this approach can provide decision makers with information and tools to monitor the condition of the highway network. This can help not only to assess maintenance needs but also to check the efficiency and quality of maintenance field work.

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Development of Mathematical Models to Assess Highway Maintenance Needs and Establish Rehabilitation Threshold Levels

PAUL E. THEBERGE

Recent developments in methods of managing pavement investments have emphasized the importance of communication between the various subsystem components of a pavement management system. Historically, the maintenance element has been difficult to integrate. A systematic and objective means of assessing maintenance needs would improve the likelihood that funds would be optimally expended. This study was undertaken to examine the mathematical relationship between a variety of pavement attributes, and other quantifiable variables, on the one hand, and maintenance needs and priority evaluations made by district area supervisors on the other. A secondary objective was to establish threshold levels for preventive maintenance, capital maintenance, and rehabilitation. Descriptions, which conform to the Maine Department of Transportation's operations, were included in order to categorize various rehabilitation and maintenance strategies as well as to define various types of maintenance. A simple questionnaire was employed to obtain the required subjective input from maintenance staff. Measures of pavement distress routinely collected by trained observers and appropriately weighted, using a Delphi technique, proved to correlate the best. Roughness measured by a response-type road measurement device and correlated with the Quarter Car Index also proved significant, but to a lesser degree. A series of other variables made only nominal improvements in the models. A model to predict repair categories from similar data was also developed. Recommendations are offered for providing tabulated information to maintenance personnel to use as a "tool" in establishing priorities.

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During the past two decades a vast amount of research has been conducted on methodologies for improving the ability to manage pavement investments. The concept of a pavement management system (PMS) originated from this research.

Pavement management emphasizes the importance of the integration of planning, design, construction, maintenance, and evaluation of and research on pavements.

In 1981 the Maine Department of Transportation (MeDOT) initiated an in-house study to evaluate its pavement management process and developed short- and long-range plans for pavement management improvements. In 1982 the department released a report *(1)* that summarized the task force efforts. In 1983 an independent study *(2)* was conducted of pavement management practices in the department. A subsequent review by the department of the findings of that study indicated that from 1981 to 1984 the PMS process had progressed satisfactorily. However, several areas in which improvements could be made were identified. One weakness identified was the difficulty of integrating maintenance functions into the pavement management process. This research was undertaken to address that problem.

OVERVIEW

There is a variety of maintenance and rehabilitation strategies that a highway agency can adopt; each strategy provides a measurable benefit to the system. However, there are a large number of candidates competing for the limited available budget. Consequently, to optimize its available funding, an agency must determine the proper time and location for carrying out the appropriate strategies. To accomplish this, a methodology for identifying and establishing proper maintenance actions, in conjunction with other rehabilitation options, must be developed.

The state of Maine systematically measures and evaluates the condition of all 8,700 centerline miles of highway within its jurisdiction. This information, in the form of pavement condition ratings and serviceability indices, is routinely used to develop rehabilitation and other capital improvement programs. Maintenance actions are normally initiated by maintenance engineers on the basis of their best judgment and knowledge of budgeted monies. This effort is intended to develop a methodology, which incorporates the same kinds of data used to establish other programs, to objectively assess maintenance needs and priorities.

OBJECTIVES

The main goal of this research was to identify one or more objective measures that could be used to reliably predict the level of maintenance required on a highway section. There were six specific objectives:

1. Devise a rating scale to convert subjective ratings of maintenance needs (by operating personnel) to a numerical scale;
2. Conduct subjective evaluations of a sample of existing pavements by having a number of maintenance personnel rate the selected sites using the previously mentioned scale;
3. Obtain from department records all pertinent data on each of the selected sites;
4. Perform a multiple regression analysis to relate the subjective ratings to the available section data;
5. Develop a simple equation to provide a mechanism for identifying maintenance needs and priorities; and
6. Identify the appropriate priority levels or develop a unique index to establish the appropriate ranges of ratings that indicate the need for preventive maintenance, capital maintenance, and rehabilitation.

INITIATING RESEARCH

Before this study was undertaken, a series of informal discussions with several Bureau of Maintenance personnel was held. Personnel interviewed included a Division (District) Engineer, a Division (District) Superintendent, and Area Maintenance Supervisors. (The MeDOT is one of a few states that still call their geographic field units divisions.) The intent of those discussions was to explain the overall goal of the study and to see if the methods being considered would be properly interpreted by key maintenance personnel. These initial discussions confirmed some early assumptions and provided insight and guidelines on the subsequent evaluation of data.

After these early discussions a literature review was initiated. A Highway Research Information Service search on maintenance cost and needs generated 15 references that were reviewed *(3–17)*. Efforts were concentrated on methods that would accommodate existing departmental records and data.

AVAILABLE DATA

To meet the needs of its network pavement management process, the department periodically collects a variety of data. The three major types of data are

1. Pavement condition,
2. Ride quality (roughness), and
3. Structural adequacy.

These were considered the prime factors on which this study should be focused. A brief description of each is provided next.

Pavement Condition

The department performs a detailed survey of its entire highway network every 2 years. The survey is performed by a trained two-person team in each division. The 1,200± mi per division require approximately 1 month to complete. Depending on the uniformity of conditions, a minimum of one site per mile is evaluated. The survey team observes or measures, or both, the severity and extent of seven types of distress: longitudinal cracking, transverse cracking, load-associated cracking, edge cracking, distortion (rutting and crown), patching, and shoulder condition.

Ride Quality

The department performs an annual ride quality survey of its entire highway network. It employs a device known as a Mays ridemeter. The output is calibrated to a Quarter Car Index *(QI)* using a rod and level–generated profile. The data are further correlated with a panel-generated present serviceability index (PSI).

Structural Adequacy

In addition to the two statewide surveys, the department also performs nondestructive testing of selected pavement structures employing a Road Rater. This device measures the "deflection" of a pavement structure under a known dynamic load. This represents the capacity or strength of a pavement structure. It is primarily used to evaluate specific highway projects.

STUDY METHODS

Given the nature of the readily available data, the following methods were considered:

1. Employ the Pavement Condition Rating (PCR) developed from the department's biennial distress survey and correlate it with some measure of maintenance need;
2. Weigh the PCR to reflect maintenance;
3. Using data from the biennial pavement survey, develop an index totally independent of the PCR;
4. Use a measure of road roughness developed from Mays meter data; and
5. Use structural evaluations performed with the Road Rater.

After considerable examination, it was determined that there were no other known methods that could be adapted to the available data or to the time and resource constraints of this study. Therefore, the following conclusions were drawn:

1. Even though the first three methods would all rely on the same pavement survey data, the third method was preferred because it would introduce less error;
2. The fourth method was also considered a reasonable approach, and it could be used in parallel with pavement evaluation; and
3. The fifth method was considered impractical.

The approach selected consisted of three major tasks:

1. Identification of a method of assessing the significance of all of the factors observed or measured by the department in its biennial pavement evaluation process;
2. Development of methodology to "correlate" the sum of these distress factors with some measure of maintenance need; and
3. Examination of the relationship between the same measure of maintenance need and the *QI* obtained annually from the department's Mays meter survey.

MAINTENANCE SUBSYSTEM

To meet the objectives of assessing maintenance needs, it became imperative to define which specific activities, within the maintenance subsystem, were significant in establishing priorities. Emphasis was placed on defining the activities applicable to MeDOT policy, as well as identifying the broad categories of rehabilitation and capital maintenance.

DEFINING MAINTENANCE

The definition of maintenance varies among transportation departments. In a technical sense, maintenance includes a range of activities directed at keeping a pavement in an acceptable state. A variety of terms is used to describe the various components of maintenance. Although some maintenance activities have little or no effect on the performance of pavements, and are not considered directly in a PMS, they are of direct interest to the maintenance department in a maintenance management system (MMS).

CATEGORIZING MAINTENANCE ACTIVITIES

In general, maintenance activities in Maine can be divided into six basic categories. A few typical activities are listed for each.

1. Service maintenance, summer: mowing, litter patrol, and sweeping;
2. Service maintenance, winter: plowing, sanding, and salting;
3. Traffic maintenance: striping and sign repair or replacement;
4. Bridge maintenance: painting and deck repair;
5. Preventive maintenance: crack sealing and cleaning drainage structures; and
6. Capital maintenance: shimming ruts, base and pavement repair, shoulder repair, and filling potholes.

This study was directed to those activities listed in Categories 5 and 6.

The six categories were established to replace the term "routine maintenance," which is avoided throughout this study. One of the problems in attaining a uniform definition of maintenance originates with the use of that term. "Routine," to some agencies, covers the majority of roadway activities, many of which may be performed regardless of pavement condition. Some may, indeed, be triggered by one of several other factors such as traffic, safety, or aesthetics. In some other agencies "routine" refers to preventive or minor maintenance only. The categories established in lieu of "routine" are consistent with MeDOT operations and are more likely to be uniformly interpreted.

DEFINING REHABILITATION AND CAPITAL MAINTENANCE

One point that required clarification was that of defining rehabilitation as opposed to capital maintenance. For purposes of this study, specific definitions that conform to MeDOT's definition were established. A similar approach could be adopted by any agency. The definitions established were based on actual programs and activities undertaken and include

1. Any placement of a thin overlay, which exceeds ⅝ in. and is preprogrammed for sections of highway as part of the department's annual maintenance resurfacing program, is considered the dividing line between maintenance and rehabilitation. This activity is normally restricted to low-volume roads and always involves contract work. The activity ususally takes place on contiguous sections of highway that are longer than 1 mi.
2. Rehabilitation activities on the collector system, which are preprogrammed and involve contract work, are considered rehabilitation in present PMS strategies. These are normally at least ¼ mi in length and are usually performed on low-volume roads. They should not be confused with the contract work performed on the federal and state highway systems.
3. Paving activities involving state maintenance forces, applied to short stretches of highway of between ¼ and ½ mi as well as occasional longer stretches, are considered capital maintenance.

4. Localized rehabilitation performed by state maintenance forces on any system is considered capital maintenance. These activities are normally restricted to sections of less than ¼ mi.

STUDY LOCATION

After the objectives of the study were established a decision had to be made about the most effective way to implement the study. The first task undertaken was to establish appropriate field locations at which to perform the further investigations. Two fundamental approaches were identified:

1. Select a sample of highway sections on one or more systems or functional classes of highway throughout the state or
2. Examine in greater detail sections of highway within one of the state's seven highway divisions.

After both options had been evaluated, the second was selected because it offered the best opportunity to meet the study objectives within a reasonable amount of time. Initial estimates of the time required to collect and extract the required pavement data, as well as obtain the appropriate maintenance input under the first approach, were considered excessive.

SELECTING DIVISION AND SYSTEM

Both subjective and objective input were used in selecting a highway system and maintenance division. Four systems were considered: *(a)* Interstate, *(b)* rural arterials, *(c)* urban arterials, and *(d)* collectors.

The Interstate system was excluded because many of the maintenance activities of interest to this study rarely occur on it. The urban systems were excluded because a majority of their maintenance responsibilities are undertaken by local communities. Because of the volume of data required to address both remaining systems, a decision was made to concentrate on the rural arterial system. This remaining mileage comprises both the major and the minor arterial classes. These functional classes generally parallel the Federal-Aid Primary and Secondary State Highway Systems and amount to about 400 to 500 centerline miles per division, which is about ⅓ of their jurisdictional mileage.

To ensure a uniform and broad range of conditions, it was considered desirable to select a division with a wide distribution of pavement conditions. For this purpose, recent pavement condition data were examined. Using this criterion, the selection was narrowed down to Divisions 3 and 7. Division 3 was selected because

1. As a result of a prior pilot study, there were more objective data available on both pavement condition and maintenance priorities in Division 3.
2. Division 3 contained a better distribution throughout the entire range of conditions.
3. Division 7 would make an ideal candidate for a follow-up investigation because it had been selected for a parallel study to examine suggested modifications to the MMS.

SELECTION OF APPROPRIATE HIGHWAY SECTIONS

To satisfy the objectives of this study, the highway sections selected had to exhibit a wide range of conditions. To ensure that the sites represented all of the categories and levels of need encountered, a variety of records from both the Bureau of Maintenance and the Bureau of Planning had to be examined. To satisfy all of the requirements, a total of 84 sections representing 117 mi were selected. The sections were carefully selected to represent all crew areas and to be equally distributed among the three geographic areas in the division. Table 1 gives the breakdown and perecentage of the study mileages within each area and crew responsibility.

TABLE 1 DISTRIBUTION OF STUDY MILEAGE

Area	Crew	Total Miles	Study Miles	Total (%)
33	3321	70.0	8.6	12
	3322	71.0	26.7	38
		141.0	35.3	25
34	3421	59.0	22.4	38
	3422	62.8	8.3	13
	3423	33.2	6.6	12
		155.0	37.3	24
35	3521	53.9	29.1	54
	3522	66.9	15.8	24
		120.8	44.9	28
Division total		416.8	117.5	28

FACTORS CONSIDERED

Many factors in addition to pavement distress and ride roughness were considered. The following list summarizes all of the other factors eventually examined.

1. Average annual daily traffic *(AADT)*,
2. Daily 18-kip equivalent single axle load (ESAL),
3. Years since construction *(AGE)*,
4. Years since last improvement *(ASLI)*,
5. Drainage condition (DV), and
6. Cut or fill status (XV).

Because flexible pavements account for more than 95 percent of the total highway mileage, pavement type was not included as a study variable.

DELPHI DATA

To establish a starting point, a decision was made to examine data collected under an earlier study performed by the MeDOT *(18)*. That effort was directed at establishing the pavement condition rating (PCR) previously mentioned.

To develop the rating, a list of distress measures, which a panel of departmental experts agreed were the most predominant ones to consider, was identified. Thereafter, the Delphi technique was employed. Briefly, Delphi utilizes the opinions of "experts" to establish a measure of a particular study's objective. To obtain these opinions, two or more iterations of a questionnaire are used. The second and any subsequent round of questions are usually provided in a form summarizing the results of the previous one. This tends to "calibrate" the panel of experts and generally enables a convergence of opinions. One noteworthy effort that used a conventional Delphia approach was the development of maintenance levels-of-service guidelines as part of a NCHRP Study 223 conducted in 1980 (*19*, p.118).

When the objectives of the Delphi process had been determined, the PMS group selected 25 engineers representing the various disciplines within the department. These experts were asked to give their opinions about the significance of various levels of severity and extent of distress for five pavement attributes. A total of three rounds of questions were performed at that time.

The five attributes of interest were *(a)* pavement condition, *(b)* safety, *(c)* roughness, *(d)* structural adequacy, and *(e)* maintenance needs. Of the five attributes on which data were collected, only values representing pavement condition were analyzed at that time.

INITIAL DELPHI DISTRESS RELATIONSHIP

As previously noted, Delphi significance values had been obtained on the five attributes identified. The Delphi data associated with maintenance needs were selected as a starting point for this study. The final Delphi iteration results were analyzed, tabulated, and modified slightly to account for some minor format changes. These Delphi results represented significance values for each level of distress. These values were then normalized so that the total of the most severe case that could theoretically exist would equal 100. The maximum value of 100

TABLE 2 NORMALIZED DEDUCT-VALUES FOR MAINTENANCE NEED

Distress	Severity Level	Deduct-Values for Extent Level (%)		
		<25	25–50	>50
Transverse cracking	>Hairline	1	4	(6)
Longitudinal cracking	>Hairline	1	5	(7)
Load-associated cracking	Initial	2	5	9
	Advanced	5	10	15
	Severe	7	(16)	22[a]
Edge cracking	Initial	2	5	9
	Advanced	5	10	16
	Severe	7	16	(22)
Distortion	<½ in.	1	2	4
	½–1 in.	2	6	9
	>1 in.	4	10	(17)
Patching	Good	1	4	5
	Fair	2	9	13
	Poor	5	12	(22)
Shoulder condition	Deficient	2	6	(10)

NOTE: The maximum of 100 deduct points is the total of the values within parentheses.

[a]By definition it is not possible to have more than 50 percent of both severe load-associated and edge cracking.

deduct points represented a "terminal" state. The total range of deduct-values is given in Table 2.

To enable raw field data to be rapidly processed, a simple procedure was written using Statistical Analysis Systems (SAS) *(20)*. Total deduct-values were calculated for each section.

SUPPLEMENTARY SECTION DATA

After the Delphi results were processed, the pavement management files were processed to extract appropriate section data. When this process had been completed there were still three variables for which data had to be obtained: axle loading (ESAL), drainage condition (DV), and cut or fill status (XV). The Bureau of Planning was contacted to obtain the required loading information. The remaining two items required field visits by a technician. Approximately 1 week was required for each of these operations. When these had been completed, all of the data were aggregated into one file.

INFORMATION NEEDS

To meet the study objectives, a shopping list of various types of desirable information was developed:

1. Verification of maintenance activities performed on each section,
2. An indication of where maintenance had been deferred,
3. A subjective measure of maintenance need on each study section,
4. A measure of a section's relative demand on maintenance crews, and
5. An indication of preference for maintenance or rehabilitation on each section.

It soon became apparent that maintenance personnel should be involved to obtain the desired information. After the list had been reviewed, a decision was made to employ a questionnaire. In developing the questionnaire, several factors were considered important if the information was to be consistent:

1. The survey should not rely on lengthy written responses,
2. The form had to be simple with check-off or numerical input,
3. The form should probably be confined to one page, and
4. Instructions and questions should be written in language understood by maintenance personnel.

Several versions were experimented with until one that met all of the guidelines was developed.

MAINTENANCE QUESTIONNAIRE

The questionnaire contained five questions. Two were directed at objectives beyond those discussed in this paper. Those findings can be found in a complementary document *(21)*. The major emphasis of the remaining questions was on obtaining the required subjective input on maintenance needs and priorities and preferences among preventive maintenance, capital maintenance, and rehabilitation on each study site.

To establish a yardstick against which the participants could subjectively rate, a scale was employed and described on the form. For consistency with other pavement indices, a 0 to 5 scale was adopted. The scale served to define extremities: out of service = 0 and new pavement = 5. Brief descriptions of increasing levels of maintenance were introduced at four points (1, 2, 3, and 4) along the scale. Participants were asked to place a mark along the scale at the point that best described each section evaluated. This scale provided the mechanism for establishing the required correlations.

CONDUCTING THE SURVEY

To initiate the survey, the Division 3 Engineer was contacted and it was agreed that the three area supervisors would be the most appropriate personnel to participate. They were contacted and agreed to take part. The questionnaires, instructions, and maps identifying each site were forwarded to each of the participants. They were instructed to visit each site when completing the form. The forms were completed with little difficulty and quickly returned. The entire field process was accomplished within a normal week's work load. In addition to the required responses, some additional and beneficial information was voluntarily provided.

MAINTENANCE NEEDS INDEX

The main purpose of this phase was to establish an objective measure of need for maintenance on specific sections of highway. The aim was to identify those significant measures that could reasonably predict the responses offered by a group of area maintenance supervisors.

Some uncertainty was introduced because only one person evaluated each section. However, in contrast, those selected were considered to be most knowledgeable and intimately aware of need. Even so, "between-area" differences were a possibility, and "calibration" of the raters would have required an expanded investigation. Suggestions about how to address this are offered later.

Initially, the eight variables identified earlier were each evaluated against the subjective rating in order to establish correlation tendencies. The variable that correlated best with the subjective rating was the total deduct points established from the Delphi weights. A second promising variable was the Mays meter roughness measurements represented by the *QI*.

DEVELOPMENT OF MODEL

To investigate possible rater variation, the most significant variable was evaluated against the responses of each participant. These initial evaluations produced fair-to-good results, and, as expected, variations did occur between raters. There were no data that indicated that the initial relationship was nonlinear within the range of values experienced in the study. Deduct-values ranged from 4 to 65.

The single-variable models obtained for the respective areas along with the composite model are

Maintenance Area	*Model*	R^2	
33	$MNI = 4.55 - 0.057$ (deducts)	0.56	(1)
34	$MNI = 4.74 - 0.056$ (deducts)	0.62	(2)
35	$MNI = 3.68 - 0.040$ (deducts)	0.39	(3)
Combined	$MNI = 4.41 - 0.050$ (deducts)	0.53	(4)

where *MNI* stands for maintenance needs index. Note that R^2 is a measure of the ability of an equation to predict actual values. A perfect model would have an R^2 equal to 1.

The models derived from Areas 33 and 34 were strikingly similar. A more detailed investigation of possible causes for the variation in the Area 35 responses revealed major clustering of subjective ratings around the whole numbers on the form where maintenance levels were described. This phenomenon appears to account for the observed inconsistency.

The two similar sets of data (33 and 34) were used to generate the following equation:

$$MNI = 4.65 - 0.056 \text{ (deducts)} \qquad (5)$$

with a corresponding R^2 equal to 0.59. At this point the second most significant variable (*QI*) was introduced and another model was developed. Although *QI* was significant, it did not improve the correlations as greatly as anticipated. It did account for an increase of 5 percent in prediction. This model was

$$MNI = 5.25 - 0.044 \text{ (deducts)} - 0.02\ (QI) \qquad (6)$$

with a corresponding R^2 equal to 0.64

To examine the effects of the remaining variables, a stepwise selection process was used to generate multiple regressions of the maintenance needs rating on all eight original variables in the correlation matrices. Stepwise selection builds the regression model incrementally, one variable at a time, so as to maximize predictability while carefully controlling for statistical significance. This process is terminated when no new variable can be added to the model, or substituted for a variable already in the model, while producing a significant increase in predictability.

This process generated a three-variable model of the form:

$$MNI = 5.00 - 0.044 \text{ (deducts)} - 0.018\ QI - 0.0001\ AADT \qquad (7)$$

with $R^2 = 0.66$.

To this point all the significant variables consisted of readily available data. The process also generated a series of models beyond the three-variable form indicated. Although the variables introduced were statistically significant, improvements were minimal. The maximum R^2 obtained was 0.70. These extended models were not pursued because the slight improvements did not warrant the effort required to obtain the extra data.

MODEL EVALUATIONS

For purposes of this study the two-variable form (Equation 6) was further evaluated for statistical validity. The model was first employed to generate predicted values for each section. Residuals (difference between predicted and actual) were also generated for each of the 79 sections with available data. To check the prediction capability of the model, the subjective Maintenance Needs Ratings (MNRs) were plotted against the predicted values, as shown in Figure 1.

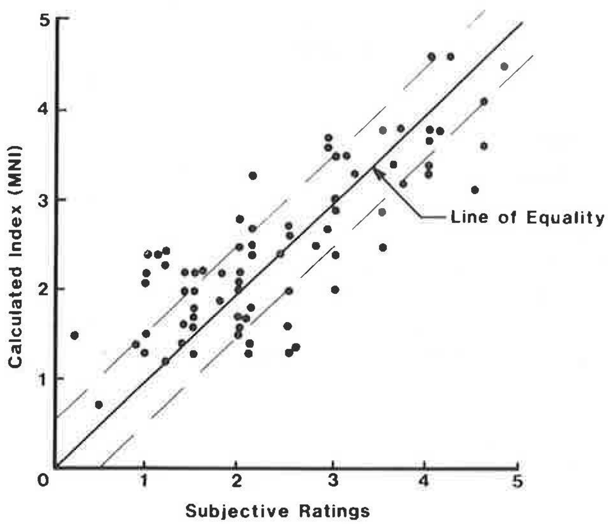

FIGURE 1 Plot of calculated needs index against ratings.

As with most models, an error term exists unless the model accurately predicts every observation. Residuals are estimates of error and used to test for normality and variance. In checking for a good regression model, the error is assumed to be normally distributed with a mean of zero and constant variation. To verify this, two diagnostic checks were performed. To check for normality, a frequency histogram for the residuals from Equation 6 was constructed. The bell-shaped curve that was observed supports normality (mean of zero and normal distribution).

To check for variance, the residuals were plotted against the predicted needs index for the model. The random distribution of errors confirmed constant variance.

ESTABLISHMENT OF A REHABILITATION THRESHOLD VALUE

One of the questions offered an opportunity for maintenance personnel to indicate their opinion about the most cost-effective strategy to perform. The three choices of strategy were

1. Preventive maintenance,
2. Capital maintenance, and
3. Rehabilitation.

Because some of the study sections were currently programmed for rehabilitation, this also offered an opportunity to examine those sections selected through the existing PMS programming process.

Before developing a separate model, each response to the subject question was matched with the calculated maintenance needs index previously recommended. The data were then analyzed to pinpoint any apparent "ranges" within which strategies were predominant. This exercise identified three significant ranges:

1. Perform preventive maintenance at index levels greater than 3.2,
2. Perform capital maintenance at index levels between 2.4 and 3.2, and
3. Perform rehabilitation at index levels below 2.4.

These ranges produced an overall prediction rate of 84 percent. To determine if these values could be improved on, a second exercise was performed starting with the same variables used earlier. Regressions were run between the responses and each variable. Again, deduct points correlated best.

Another set of evaluations, again employing a stepwise regression, was made using the study variables. After several versions, a final form was selected:

$$RSI = 0.563 + 0.031\,(\text{deducts}) + 0.021\,(QI) - 18\,IR \qquad (8)$$

where

RSI = repair strategy index and
$IR = [(AGE + 1) - ASLI]/AGE^2$.

This index represented a range from zero (no maintenance required) to four (pavement requires extensive rehabilitation).

Values were calculated for each section and matched with the associated response. Analysis of this data revealed the following:

1. An RSI below 1.5 accurately indicated sections for which preventive maintenance was optimum 100 percent of the time.
2. At index levels greater than 2.5, rehabilitation proved to be the response 91 percent of the time. The mean value of the index for this range was 2.90.
3. All 10 sections for which rehabilitation had been programmed exceeded an index of 2.8 with a mean value equal to 3.15.
4. Index values between 1.5 and 2.5 included all three strategies. However, a secondary break was identified around a level of 2.1.

As was the case for the maintenance index, the middle range proved the most difficult to tie down. There was a tendency in this area for participants to recommend rehabilitation when, in reality, adequate funds would probably never be available, given all of the other candidates. Selection of capital maintenance would be a reasonable alternative.

On the basis of this investigation the following ranges of index values were established:

1. Perform preventive maintenance at index values less than 2.1,

TABLE 3 SUMMARY OF INDEX RANGES, RESPONSES, AND PERCENTAGE AGREEMENT

Strategy	Needs Index Responses Range	No.	Percentage	Repair Strategy Index Responses Range	No.	Percentage
Preventive maintenance	3.2–5.0	22	77	0.0–2.09	22	82
Capital maintenance	2.4–3.19	17	71	2.1–2.49	15	71
Rehabilitation	0.0–2.39	40	93	2.5–4.0	42	93
Overall predictions			84			86

2. Consider capital maintenance for index values between 2.1 and 2.5, and
3. Consider rehabilitation when the index exceeds 2.5.

Using these ranges a reasonable prediction would have been made 86 percent of the time.

Table 3 gives a summary of both models with their associated prediction percentages. Although it is not apparent from the table, the results obtained by employing the *RSI* do not reflect much of a "spread" of responses. Simply put, this means that those responses that do not conform when the strategy index is used come closer to being correct than they do when the needs index is employed. The distributions of responses observed were extrapolated and frequency histograms by ranges of conditions were constructed. This is shown in Figure 2. The areas of conflict (the 15%± observed) is represented by the shaded overlapping tails of the distributions. This is probably due to nonquantifiable factors. It is doubtful that significant model improvements are probable or even necessary. It would appear that the minimal improvement offered by developing a separate model suggest that it is not necessary. However, initially it might be advisable to use both so that a more extensive evaluation of the spread of responses can be made.

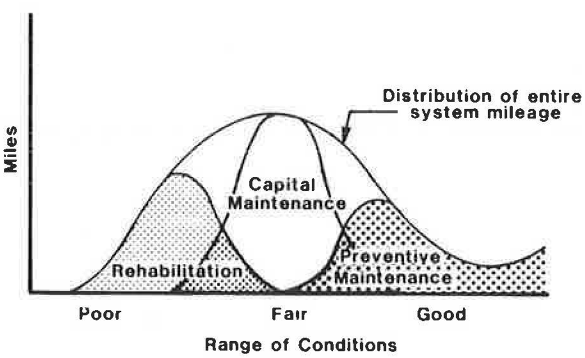

FIGURE 2 Distribution of preferred strategies by condition.

CONCLUSIONS

Findings from the study as well as comments on the research follow:

1. The MeDOT has designed a procedure for obtaining distress and serviceability data that are pertinent to the network pavement management process.
2. This research project has benefited from data pertinent to maintenance needs, collected earlier in a Delphi experiment.
3. The extensive volume of data routinely gathered provides a substantial data base on which to exercise the models developed in this research.
4. In all of the investigations, measures of pavement distress as represented by the total deduct points proved to be the most significant measure in predicting maintenance need as well as the appropriate repair strategy.
5. Although there may appear to be some redundancy between the pavement condition rating (PCR) and the maintenance needs index *(MNI)*, the latter specifically addresses maintenance. Consequently, it is intended for that purpose and should be used in parallel, not combined, with PCR for network analysis.
6. The recommended models offer an opportunity to provide the maintenance division a variety of tabulated information on all highway sections.
7. The same information that is provided to maintenance can be readily incorporated as complementary information into the network analysis process.
8. No data in addition to those that are routinely collected and updated for the network program are required to provide the indices suggested by this research.

RECOMMENDATIONS

The recommendations offered are reasonable and consistent with the department's overall goal of improving its pavement management process.

1. The maintenance needs regression model developed should be applied to the distress data routinely gathered by the department. The model should be added to the current PMS analytical process. This would allow automatic calculation of indices during data input.
2. The data should be tabulated by system and by division and provided to each maintenance unit.
3. A basic training session lasting just a few hours should probably be scheduled to familiarize maintenance personnel with terminology, location information, field interpretation, and use of the data generated.
4. In addition to the maintenance needs index the repair strategy model should be employed and values tabulated in parallel with the maintenance needs index so that a further evaluation of the ranges associated with the three categories can be made.
5. It is suggested that these tabulations be initially used for

preliminary assessment of the values to isolate any obvious discrepancies.

6. The rating provided is an estimate and should be used as a preliminary tool by maintenance operating personnel for comparing sections within similar systems (e.g., primary, secondary). The major advantage offered by the model is that it provides a ready and quick tool for eliminating the need to look at sections that do not require immediate attention.

7. As a means of improving the needs model, the department could consider expanding the original experiment by employing three participants in each area. This would have to be carried out in another field division. The final decision on this second exercise should be based on whether the increased confidence would be worth the effort. Because the information is initially intended as a planning or scheduling tool, it is doubtful that such an effort is warranted at this time.

8. The department should consider employing a similar approach to the Repair Strategy Index for identifying specific rehabilitation types (i.e., light, medium, and heavy overlay). This could be accomplished by employing a similar questionnaire on a sufficient number of field sites.

ACKNOWLEDGMENTS

The findings presented in this paper represent those of a portion of a larger study *(21)* funded jointly by the Federal Highway Administration and the Maine Department of Transportation. Appreciation is expressed to supervisory personnel of Maintenance Division 3 in Bangor who first participated first in interviews and then provided staff for completion of the questionnaires. Special thanks are extended to the Pavement Management staff for providing the required base data and to the Technical Services staff for general assistance.

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Development of a Routine Pavement Maintenance Data Base System

KHALED KSAIBATI AND KUMARES C. SINHA

When a routine maintenance management system is developed, the creation of a meaningful data base should be considered. In this paper is presented the development of a microcomputer data base that can be used at different maintenance management levels of the Indiana Department of Highways. To determine what type of data to include in the data base, the relationship between roughness and level of routine maintenance expenditure was analyzed. Condition survey information, based on unit foremen's evaluation of highway deficiencies, may be included in the proposed data base. The condition survey information along with roughness measurements can be used in two ways. First, the Central Office can use the information in programming maintenance and rehabilitation activities. Second, the data can be used by subdistricts to set priorities for routine maintenance work on highway sections within their jurisdictions. Information on rehabilitation activities, such as resurfacing, was included in the data base to increase the level of coordination between the programming of major maintenance and routine maintenance activities. This coordination may result in substantial savings in pavement maintenance and rehabilitation. Some other supplementary information, such as average daily traffic, contract number, county, subdistricts, and pavement type, was included in the data base. A pilot implementation plan is proposed. Performance of the data system in pilot implementation should be evaluated to provide the feedback necessary to assess the value of the information included in the data base.

Routine maintenance of the Indiana state highway system consumes from 40 to 50 percent of total highway expenditure *(1, 2)*. However, as in many other states, the routine maintenance program in Indiana is not effectively coordinated with major activity programs. This lack of coordination is because the philosophy of the development of major activity programs is different from that of routine maintenance programs.

Major activity programs identify highway sections that are at or near a prescribed structural failure level and then allocate resources to upgrade these sections within the available funds. Routine maintenance programs, on the other hand, consider only the apparent condition of a highway element (pavement surface condition of a highway) and try to keep the serviceability as high as possible, regardless of the structural adequacy of the highway element.

Although the criteria for the development of major activity programs may differ from those of routine maintenance programs, both programs have a common goal of preserving the condition of the highway system. An effective coordination between the two programs may result in considerable savings. For instance, sometimes expensive routine maintenance activities, such as seal coating, get applied on sections that have been scheduled to receive resurfacing within a few months *(3, 4)*. Such a situation arises in the absence of coordination. An effective exchange of information between two programs is thus essential. Figure 1 is a schematic presentation that shows an example of how information on seven major activities can relate to routine maintenance activities. Table 1 is a list of pavement maintenance activities included in the roadway and shoulder group.

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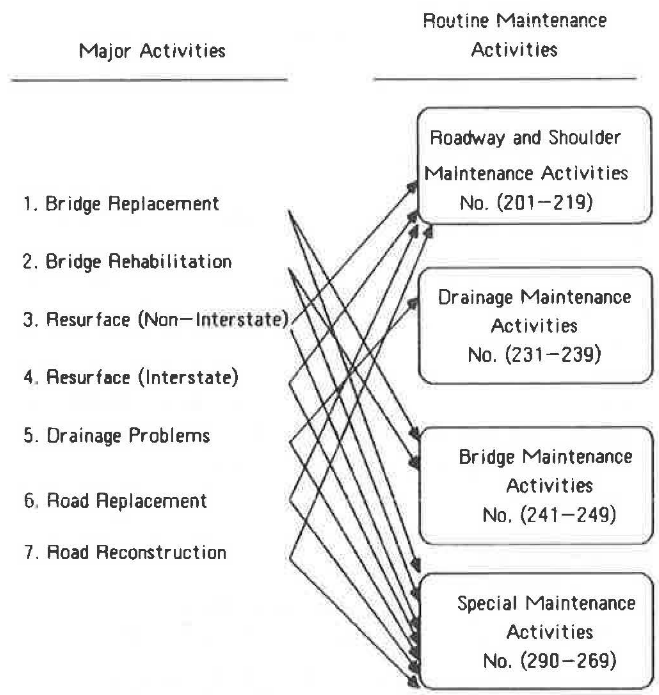

FIGURE 1 Relationship between major activities and routine maintenance activities.

TABLE 1 ROUTINE PAVEMENT MAINTENANCE ACTIVITIES

Activity	IDOH Code	Production Unit
Shallow patching	201	Tons of mix
Deep patching	202	Tons of mix
Premix leveling	203	Tons of premix
Seal coating	205	Lane-miles
Sealing longitudinal cracks and joints	206	Linear feet
Sealing cracks	207	Lane-miles
Cutting relief joints	209	Linear feet
Joint and bump burning	210	Bumps removed
Others	219	Man-hours

The purpose of this research effort was to develop a computerized information system that could transfer the available information on the current condition of highway elements and programmed improvement activities to different levels of routine maintenance management (central office, district, and subdistrict). It is believed that the availability of such information can result in substantial savings in maintenance expenditure and in the development of an effective maintenance program.

A routine maintenance data base system (RMDBS) is a procedure for collecting, storing, processing, and retrieving the information required in a maintenance management system. It represents the basis for a maintenance management system because all pavement decisions must be made according to a common, integrated source of information derived from reliable, good-quality data.

EFFECT OF ROUTINE MAINTENANCE ON PAVEMENT CONDITION

To ascertain what types of surface condition data were to be included in the information system, an analysis was performed to investigate the relationship between routine maintenance and surface roughness. For the purpose of this analysis, only the Interstate highway system was considered, and factors such as traffic level, surface type, and pavement age were taken into account. The general approach of the analysis can be summarized as follows:

1. Divide the network into homogeneous pavement sections,
2. Determine the roughness values and the years for the two most recent roughness measurements on each section,
3. Determine the amount of routine maintenance applied between two dates by type of maintenance activity on each section, and
4. Analyze the effect of routine maintenance expenditure level on the rate of change in roughness during the period between the two measurements.

The major findings of this analysis were as follows:

1. The rate of increase in roughness varied inversely with routine maintenance expenditure. This means that when a higher level of pavement maintenance expenditure is made on a section, it shows less increase in roughness (Figure 2).

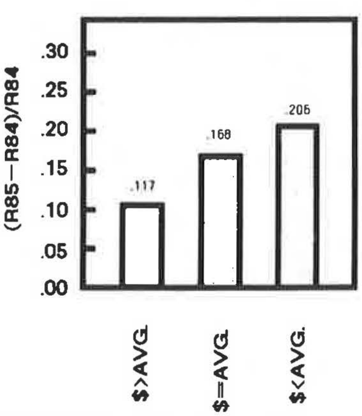

FIGURE 2 Effect of sealing expenditure (206 + 207) on level of roughness.

2. Because maintenance records are available by highway sections within the boundaries of a county and roughness data are recorded by contract sections, the analysis was impaired because of inadequate location-specific data. Nevertheless, the analysis provided sufficient evidence that roughness measurements can be used as an indicator of routine maintenance needs. In other words, ranking of highway sections according to their roughness values can be used along with other information in making decisions about the allocation of highway routine maintenance funds.

DEVELOPMENT OF A MICROCOMPUTER DATA BASE

When the proposed data base was developed, the following points were taken into consideration:

1. Use currently available data;
2. Structure the data base to permit future modifications and improvements;
3. Provide a data base that can provide management with timely access to the routine maintenance and capital programs information base at a reasonable cost; and
4. Simplify the use, maintenance, and updating of the data base.

Work Plan

The general approach followed to develop the data base can be summarized as follows:

1. Review of the existing maintenance computer files.
2. Transformation of the available highway information sources, such as highway inventory and roughness and skid resistance files, to a form suitable for use in the current organizational structure. For example, routine maintenance data are currently recorded by highway section in Indiana. A highway section refers to the stretch of a highway within the boundaries of a county. However, to be compatible with the other information, the records had to be reorganized so that they represented subdistrict boundaries.
3. Development of a computer file for future highway improvement programs.
4. Development of a computer program to prepare several information reports. These reports would inform various management units of actions taken by other units. For example, subdistricts would be able to identify sections with high roughness values and sections that were scheduled for future improvement activities in a specific location.

Data Base Elements

With the growth of available data and the increasing power of modern computers, managers are practically flooded with information. The problem is to discriminate among all of the data available and the data that the manager needs and can use. In the proposed integrated data base, only the data that can assist managers in identifying maintenance needs are included. The data base would be composed of the following three elements:

1. The Indiana Department of Highways (IDOH) roughness computer files,
2. Future programmed major activities, and
3. Results of condition survey.

These data base elements are shown in Figure 3.

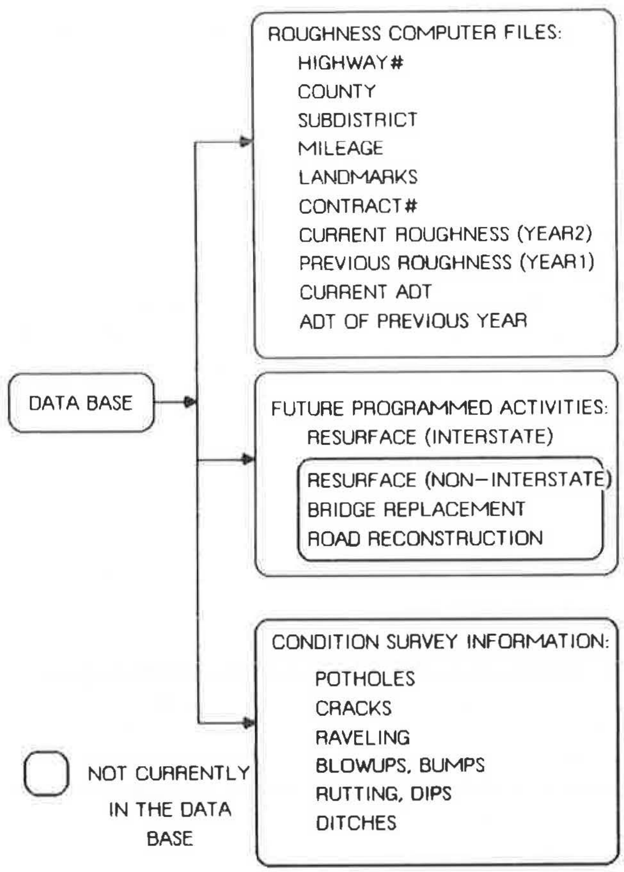

FIGURE 3 Data base elements.

IDOH Roughness Computer Files

The IDOH roughness computer files form the backbone of the proposed data base. The following information was taken from these computer files: highway number, county, year resurfaced, mileage, landmarks, contract number, current roughness, roughness of previous year, current average daily traffic (ADT), and the ADT of the previous year.

Future Programmed Major Activities

The future programmed major activities were taken from the Biennial Highway Improvement Program (HIP) *(5)*. In the HIP, projects are arranged in the following categories: bridges, resurfacing, safety improvements, roadside improvements, new facilities, park facilities, and toll facilities. Information from the 1984–1986 Biennial Highway Improvement Program was used in the data base developed in this study.

Condition Survey Information

The current pavement data collection program in Indiana does not include any kind of statewide condition surveys. Montenegro and Sinha *(6)* have recently proposed a periodic condition survey procedure to be carried out by unit foremen. The foremen would be responsible for filling out standard forms evaluating the condition of road sections within their respective units. If the survey procedure is implemented, this information would be entered into the computer and would be part of the proposed data base developed in the present study.

Uses of the Data Base

The possible uses of the proposed data base are

1. To provide timely information to managers in an understandable and easily applicable form;
2. To provide coordination between major maintenance and routine maintenance programs;
3. To provide uniform method for using surface condition data;
4. To provide information for the central office and district to use in setting priorities in the allocation of funds by activity and by subdistricts; this may be accomplished by using the results of the proposed condition survey, which may be included in the proposed data base; and
5. To allow subdistrict foremen to identify routine maintenance needs, set priorities on these needs, and program the work in accordance with the resulting priorities.

EXAMPLE APPLICATION OF THE DATA BASE SYSTEM

Personal computers are now in common use and have proven highly cost effective in information system applications. Therefore, the IBM personal computer was chosen to accommodate the proposed RMDBS for Indiana. The data base system includes a series of programs that interact with the users and produce simple reports about any specific highway section. For the purpose of this study, only the Interstate system was considered; the rest of the state highway system can be added later. The programming language of KnowledgeMan software *(7)* was used to write all of the computer programs.

Structure of the Data File

Because of the large size of the data file and the slow speed of the personal computer, the indexed sequential technique was used to access information quickly. The index file developed was named INDEX. This file is composed of 37 numbers, each of which is assigned to one of the 37 subdistricts in Indiana. Then the large data file was divided into 37 subfiles, so that each subfile includes the records of one subdistrict. A record can be defined as the set of information that describes a specific segment of the road within a subdistrict. When the computer program needs to access a specific record in a subdistrict, a search is made in the index file for the subdistrict, then a search for the record will be made only inside the subfile that includes the records for this specific subdistrict, as shown in Figure 4. This technique saves a great deal of computer time during the execution of the RMDBS, and it can help managers to obtain reports easily and quickly from the data base.

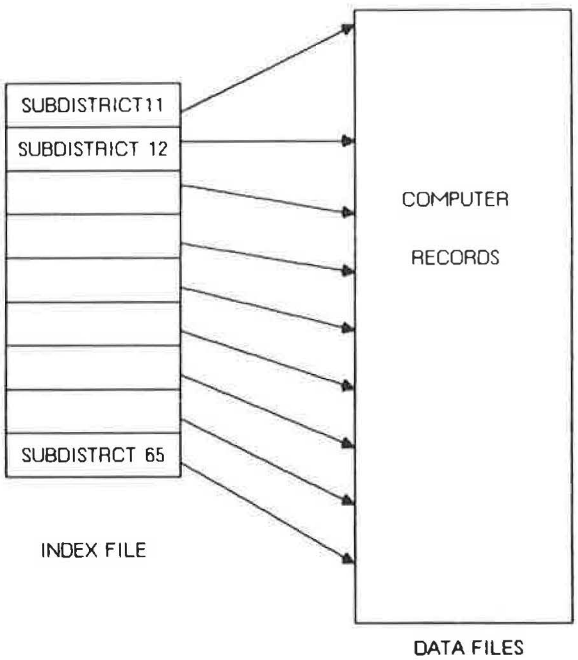

FIGURE 4 Hierarchical sequence structure.

Input Format

Communication between user and computer is perhaps the most difficult aspect of an information system. Therefore, the RMDBS was created as a tool that can be learned quickly by users of any level of computer background. The inputs and outputs of the program were made simple. Friendly interactive programs were developed to produce different menus that show the user the different available options from which he may choose. When the user selects an option, the program will automatically show a submenu, ask for information, or produce an informational report that matches the selected option. There are eight different menus; Figure 5 shows the main menu.

The program is set up so the user sees what input is typed, then the program checks the input and gives an error message if there is any input error. This kind of input is simple even for an inexperienced program user.

A carriage return locks the input to the indicated item and automatically advances to the next item for input. The item that is ready to receive an input is identified by underlining.

INDIANA DEPARTMENT OF HIGHWAYS
ROUTINE MAINTENANCE
DATA BASE SYSTEM
MAIN MENU

1. ADD HIGHWAY SECTIONS
2. MODIFY EXISTING INFORMATION
3. REPORT AND REVIEW INFORMATION
4. QUIT

SELECT A NUMBER:

FIGURE 5 RMDBS main menu.

Description of Output

The RMDBS produces 10 different reports:

1. Roughness report: This report gives the roughness measurements for the current and previous year. The roughness measurements are given between mileposts.
2. Increase in roughness report: This report shows the rate of increase in roughness between the current and the previous year for the specified section.
3. Mileage report: This report gives the milepost reading of the start and the end of the specified section.
4. Contract number report: This report shows the contract number of the specified section.
5. Date report: This report gives the date when the section was opened to traffic.
6. Traffic report: This report shows the ADT for the current and previous years.
7. Landmarks report: The landmarks that are within the boundaries of the section are shown in this report.
8. Surface type report: This report indicates whether the specified section is rigid or flexible.
9. Resurfacing project report: This report indicates anticipated future resurfacing projects and the cost of these projects.
10. Highway section report: This report lists all of the highway sections within a subdistrict.

More reports could be generated after condition survey information is added to the data base. These reports would indicate the type and extent of distress on each segment of the highway system.

If the user wants to get one of the reports, the roughness report for example, he has to operate the program to get the main menu (Figure 5). The menu consists of four options, and Option 3 will provide information and reports. After this procedure has been completed, the computer will present the user with the list of the subdistricts, and the user will choose the relevant subdistrict. The computer will then show all of the highway sections within this subdistrict and once again the user will select the appropriate section. Next, the user will select Option 1 from the menu shown in Figure 6 to get the available reports on roughness. The final step will be to choose Option 1 in the roughness and traffic information menu shown in Figure 7. Figure 8 shows the roughness report for a section in Terre Haute (Subdistrict 11).

INDIANA DEPARTMENT OF HIGHWAYS
ROUTINE MAINTENANCE DATABASE

1. ROUGHNESS & TRAFFIC INFORMATION
2. GENERAL INFORMATION
3. CONDITION SURVEY INFORMATION
4. BACK TO THE PREVIOUS MENU
5. QUIT

SELECT AN OPTION:

FIGURE 6 RMDBS information menu.

Software

Software is defined as computer programs, procedures, and associated documentation used in the operation of computer hardware. The two major categories of software are system programs and application programs.

System Programs

System programs are the computer programs used to coordinate and control the overall operation of a computer system. The PC-DOS 2.1 and KnowledgeMan 1.07 were the system programs used in the RMDBS.

Application Programs

Application programs are written for specific applications. These programs depend on the system programs during execution.

The RMDBS consists of 36 application programs. The operation of the RMDBS program is dependent on all of the available options, and all of the programs are therefore interrelated. Figure 9 shows the relationship among application programs, KnowledgeMan, and computer files.

INDIANA DEPARTMENT OF HIGHWAYS

ROUTINE MAINTENANCE DATABASE

1. ROUGHNESS FOR CURRENT AND PREVIOUS YEAR
2. ADT FOR CURRENT AND PREVIOUS YEAR
3. ROUGHNESS RATIO
4. BACK TO THE PREVIOUS MENU
5. QUIT

SELECT AN OPTION:

FIGURE 7 RMDBS roughness and traffic menu.

H.W. :70 Direction :E

Subdistrict :11 County :84

Mileage From	Mileage To	Year 1 Roughness	Year 2 Roughness
0	1	378	423
1	2	431	497
2	3	376	336
3	4	425	337
4	5	490	416
5	6	385	694
6	7	417	970
7	8	406	1168
8	9	382	989
9	10	344	437
10	11	413	443
11	12	416	804
12	13	1906	399
13	14	1668	311

FIGURE 8 Example of roughness report produced by RMDBS.

The RMDBS programs serve four different purposes. They

1. Show different menus with available options,
2. Show the input menu to identify a section or to input condition survey data,
3. Produce reports according to the option selected by the user, and
4. Confirm input information and detect input errors.

Error Messages

Because of the simplicity of the input format, the user of the RMDBS can commit few input errors. The program prints error messages in response to the kind of errors detected. After

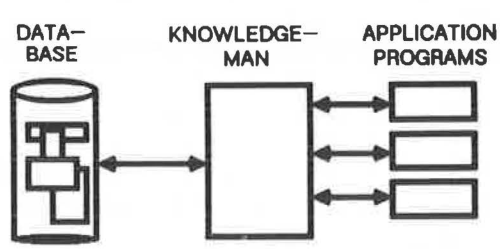

FIGURE 9 Relationship among application programs, KnowledgeMan, and the data base.

printing the error message, the program will give the user the opportunity to change the input.

System Configuration

The following computer components are essential to use of the RMDBS:

1. System unit, containing a minimum of IBM personal computer family with hard disk or true compatibles, monochromatic or color video monitor (25 lines × 80 characters), keyboard, and printer (80 column);
2. PC-DOS 2.1 or later version; and
3. KnowledgeMan software 1.07 or later version.

Information Updating

When the RMDBS was created, the need for continued updating of the data base was taken into consideration. The data of each record were divided into two parts:

1. The first part includes the results of the condition survey. It is proposed that these data be collected by the foremen. It is suggested that this part of the data base should be updated by subdistrict management. Then the information can be transferred to other management levels by computer instead of the condition survey forms. The updating of this part of the data base should be done twice each year because it is proposed that condition survey data be collected biannually.
2. The second part includes roughness, ADT, future programmed activities, and surface type. This part of the data should be updated by the central computing facilities in Indianapolis because all of these data are located in computer files there. The updating of this part should be done yearly because roughness measurements are collected on a yearly basis.

IMPLEMENTATION AND EVALUATION OF THE RMDBS

The following steps may be involved in implementation of the proposed information system on a pilot basis.

1. Selection of management units: For pilot implementation two subdistricts from each of two separate districts should be selected. Four subdistricts, two districts, and the central office should then be supplied with the necessary hardware and software.
2. User training: Training is critical for the successful implementation of an information system. Users must be informed about the formats and contents of reports and terminal displays and how to request reports. The personnel from the management units selected for pilot implementation should be given appropriate training in the use of the RMDBS.
3. Data collection: All necessary data except the condition survey information already exist in computer files. The condition survey data, if available, should be added to the data base.
4. Evaluation: Evaluation provides the feedback necessary to assess the value of information included in the system. This feedback provides direction for adjustments that may be necessary. First, the adequacy of the software should be evaluated. Ease of use can be taken as an indication of software adequacy. Next, the RMDBS should be evaluated in terms of the information provided. The objective of the RMDBS is to generate information to support maintenance decision making. Therefore, the extent to which information is relevant or not for decision making is the area of concern in evaluating the performance of the RMDBS. This evaluation can be accomplished by systematically interviewing the users in the management units selected for pilot implementation. If management is satisfied with the information system, it is reasonable to assume that the system meets the requirements. If management is not satisfied, modifications ranging from minor adjustments to complete redesign may be required *(8)*.
5. Statewide implementation: After modification, the RMDBS can be generalized for use by all maintenance management units in Indiana.

SUMMARY AND CONCLUSIONS

The most difficult task in the development of an information system is to determine the information requirements of the users. This was done by examining the available data in terms of the functions that were to be performed and by determining who might be interested in different combinations of these data. The major findings of the study are summarized next.

1. When the relationship between level of maintenance and roughness was considered, it was found that the rate of increase in roughness varies inversely with the level of routine maintenance. Because of this relationship, roughness measurements were included in the data base. These measurements could be used by subdistrict management to determine necessary surface treatments.
2. Information on future programmed resurfacing activities was included in the data base. This will help identify highway sections that are scheduled to be resurfaced. Spending on routine maintenance for these sections could be eliminated or decreased.
3. Data collected from condition surveys, suggested by Montenegro and Sinha *(6)*, may be part of the data base. This information can help management at the central office monitor the surface condition of the highway network within a subdistrict on a periodic basis. The same data could be used by subdistrict management to set priorities for performing routine maintenance activities.

After the appropriate information for different users of the RMDBS had been determined, the information was integrated in a data base. Thirty-six application programs were written to provide explicit instructions for physical location of the data elements required to satisfy a particular on-line terminal query or to produce a particular report. The integrated information management system KnowledgeMan was used to reduce the difficulties of manipulating the data contained in the data base.

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The contents of this paper reflect the view of the authors who are responsible for the facts and the accuracy of the data presented herein.

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Performance Evaluation of Jointed Concrete Pavement Rehabilitation Without Resurfacing

DAVID L. LIPPERT

The results of a research project that was conducted to evaluate the effectiveness of concrete pavement restoration (CPR) on jointed portland cement concrete pavement are described. The CPR methods evaluated were pavement grinding, grout undersealing, installing underdrains, retrofitting double-vee load transfer devices, and pavement patching. Five construction sections, located on Interstates in Illinois, were selected for evaluation. The original pavement sections were constructed between 1960 and 1963, then rehabilitated in 1983 and 1984. All pavements were of the same design: a 10-in. slab over a 6-in. granular base and a joint spacing of 100 ft. The evaluation began just before rehabilitation of each section and continued until May 1986. Evaluation was done using crack surveys, destructive testing, and nondestructive testing. Performance factors monitored were faulting, pavement cracking, pavement roughness, skid resistance, deflection, load transfer, void development, and drainage. A great deal of emphasis was placed on grout undersealing and doweled patching in laboratory and field experiments. Effectiveness of undersealing was determined by deflection testing using a Dynatest 8002 falling weight deflectometer and a Road Rater 2008. Another field experiment was conducted to investigate the effects of dowel bar size and number of dowels on full-depth patch performance. Several different techniques for dowel bar grouting were tested in the laboratory to establish grouting procedures. The findings of this research resulted in improvements in full-depth patch design, improved construction procedures, and proper use of undersealing.

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The state of Illinois has many miles of highways composed of jointed portland cement concrete (JPCC). Many of these pavements have approached or are approaching the end of their design life and are in need of major rehabilitation.

Resurfacing with asphalt concrete (AC) is one of the primary methods of rehabilitation used in Illinois. However, asphalt concrete overlays may not be cost-effective for a JPCC pavement that has faulted joints, transverse cracks, and possibly some spalling but is otherwise sound. On this type of pavement, it is possible that rehabilitation without resurfacing can be much less expensive and more cost-effective.

The main objective of this study was to determine whether pavements with faulted joints and transverse cracks or general joint deterioration can be restored by grinding, pressure grouting, placement of underdrains, retrofitting with load transfer devices, or replacement of the joints more economically over the long run than by resurfacing. Five rehabilitation projects were monitored for performance by studying such indicators as ride quality, faulting, and pavement deflection.

Initial poor performance of doweled patches resulted in a pavement patching experiment. Such variables as number of dowel bars, size of dowel bars, and grout type were investigated. Also included in the experiment were sawed and sealed joints at the patch-pavement interface. Patching performance was monitored by measuring pavement deflection, faulting, and patch distress.

Also of interest was the effectiveness of grout undersealing. Two areas of undersealing were investigated. First, undersealing of joints and cracks to fill voids (*1*) and, second, the filling of voids to stabilize rocking and pumping patches. Undersealing was evaluated using Road Rater deflection testing and void detection procedures (2) using a falling weight deflectometer (FWD).

In the laboratory the problem of proper dowel bar grouting was investigated. This was done by using several methods of applying the grout at different consistencies. The methods were then rated subjectively with respect to grout coverage, ease of use, and cost.

PAVEMENT EVALUATION SECTIONS

Five projects, which were located on Interstate routes 55, 70, 80, and 280-74 were evaluated. All pavement sections consist of 10-in.-thick jointed reinforced concrete pavement on a 6-in. granular subbase. Joints are spaced 100 ft apart and include 1¼ in. × 18 in. dowel bars spaced at 12 in. for load transfer. Figure 1 shows the general location of the projects.

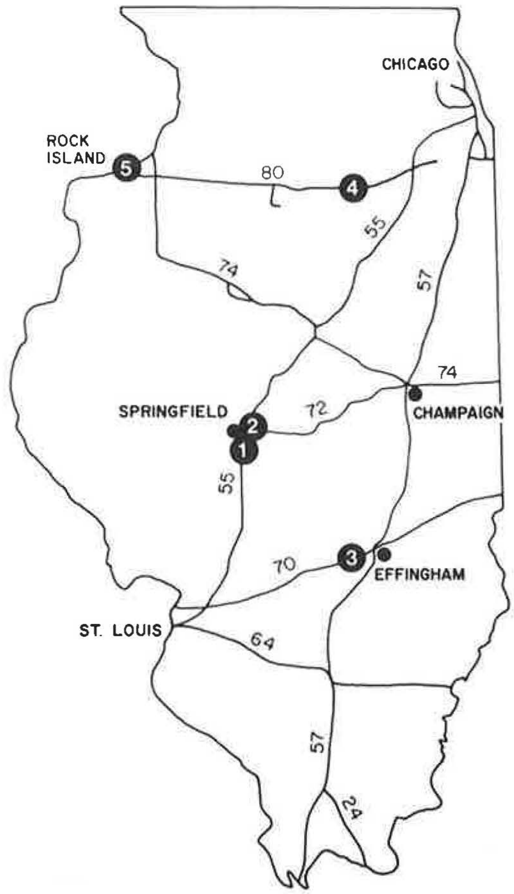

FIGURE 1 Project location map.

Each rehabilitation project was designed to address such problems as ride quality, joint deterioration, filling of voids, and restoring load transfer. Ride quality was improved by diamond grinding of faults, partial-depth pavement patching, and full-depth pavement patching of deteriorated joints. Voids were filled by grout undersealing and load transfer was restored by use of the Double Vee load transfer devices developed at the University of Illinois. The Double Vee device used is shown in Figure 2.

Rehabilitation features such as experimental undersealing, experimental patching, load transfer devices, and drainage mats are not included in the economic analysis because of the smaller quantities associated with these experiments. Summaries of the original pavement designs and rehabilitation techniques used are given in Tables 1 and 2, respectively.

Project 1

This project is located on I-55 near Springfield, Illinois, between mileposts 98.00 and 102.36 and has two lanes in both directions. Traffic on this project averages 11,500 vehicles per day with 26 percent trucks. The pavement has a low to medium D-cracking aggregate, which was evident at joints and cracks at the time of rehabilitation. The main distress types were medium-severity joint faulting, medium- and high-severity joint deterioration (due to D-cracking), and poor ride due to faulting and joint distress.

Rehabilitation first took place in 1983 and again in 1984. The 1983 rehabilitation was in the form of full-depth patching using a 3-2 patch design, as shown in Figure 3, to replace deteriorated joints. Pipe underdrains were installed to facilitate removal of water that may have been causing faulting. In 1984 a number of poorly performing patches needed to be replaced as well as a few joints that had further deteriorated since 1983. Pavement patching in 1984 used the 3-2 patch design in the northbound lanes and the 3-3 patch design in the southbound lanes, as detailed in Figure 3. Also part of the 1984 rehabilitation was blanket undersealing of existing patches.

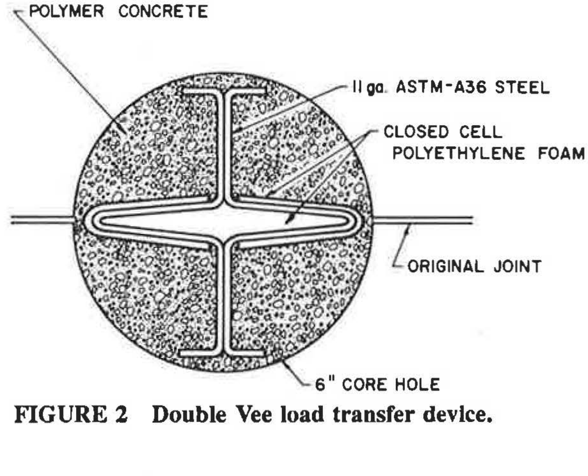

FIGURE 2 Double Vee load transfer device.

TABLE 1 ORIGINAL PAVEMENT DESIGN

Project	Route and Milepost Location	Year Constructed	PCC Pavement Thickness (in.)	Joint Spacing (ft)	PCC Aggregate Quality	Subbase Thickness and Type	Shoulder Design	Pavement Distress[a] at Time of Rehabilitation
1	I-55 98.00–102.36	1962	10.0	100	Fair (moderate D-cracking)	6 in. granular	0.5-in. surface treatment on granular base[b]	MS joint faulting MS & HS joint deterioration Rough ride
2	I-55 102.36–105.52	1962	10.0	100	Very good	6 in. granular	0.5-in. surface treatment on granular base[b]	HS joint faulting Rough ride MS high reinforcing steel spalling Limited MS joint deterioration
3	I-70 74.28–82.23	1960–1963	10.0	100	Good (low D-cracking)	6 in. granular	3-in. bituminous concrete on granular base	MS joint deterioration Moderate joint faulting Rough ride
4	I-80 105.30–111.70	1960	10.0	100	Very good	6 in. granular	3-in. bituminous concrete on granular base	HS joint faulting MS joint deterioration Rough ride
5	I-280 14.70–18.0 I-74 5.00–9.64	1962	10.0	100	Very good	6 in. granular	0.5-in. surface treatment on granular base	LS joint faulting Poor skid properties MS joint spalling

[a]Distress ratings: LS = low severity, MS = medium severity, HS = high severity.
[b]Surface treatment replaced with 3-in. bituminous concrete in early 1970s.

Project 2

This project is also located on I-55 near Springfield, Illinois, and is immediately north of Project 1. This section is between mileposts 102.36 and 105.52 and has two lanes in the north-bound direction and three lanes in the southbound direction that taper into two lanes near the southern end of the project. The traffic is the same as that on Project 1.

The pavement showed no signs of a D-cracking aggregate at the time of rehabilitation; there were only localized distress and spalling at a limited number of joints. The main distress types were high-severity joint faulting, limited medium-severity joint spalling, limited low-severity joint deterioration, medium-severity spalling and scaling due to high reinforcing steel, and poor ride quality due to joint faulting.

The main feature of this rehabilitation project was removal of joint faulting by diamond grinding to improve the ride of the pavement. Other features were full-depth patching using a 3-2 patch design, partial-depth patching to remove areas of spalling and scaling due to high reinforcing steel, and installation of pipe underdrains. Included in the rehabilitation was a grout undersealing experiment (*1*). Because of the experimental nature of the undersealing, it was not included in the economic analysis. A more detailed account of the undersealing experiment is given in the section on Undersealing.

Project 3

This project is located on I-70 near St. Elmo and Altamont, Illinois, from mileposts 74.28 to 82.23. This section of highway consists of two lanes in each direction. Traffic on this project

TABLE 2 REHABILITATION TECHNIQUES USED ON PROJECTS

Project	Route	Year of Rehabilitation	Full-Depth Patching	Partial-Depth Patching	Joint Sealing	Blanket Under-sealing	Retrofit Load Transfer Devices	Pavement Grinding	Under-drains
1	I-55	1983	X	–	–	–	–	–	X
		1984	X	–	–	X	–	–	–
2	I-55	1983	X	X	–	X[a]	–	X	X
3	I-70	1983	X	–	–	–	–	–	–
		1984	X[a]	–	X[a]	X	–	–	–
4	I-80	1983	X	–	–	X	X[a]	X	X
		1985	X	–	X[b]	–	–	–	–
5	I-280 I-74	1984	X	X	X	X	–	X	X

NOTE: Dash means technique not used.
[a]Experimental (not included in economic analysis).
[b]Patches only (not included in economic analysis).

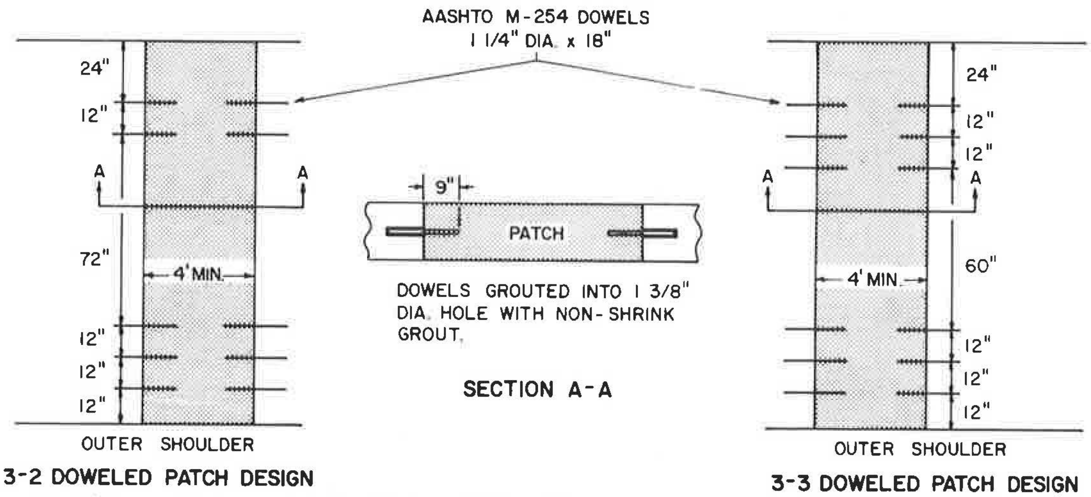

FIGURE 3 Patch design details of 3-2 and 3-3 patches.

averages 10,500 vehicles per day with 36 percent trucks. The aggregate in this pavement has low-severity D-cracking, which was evident at joints before rehabilitation. The main types of distress in the pavement at the time of rehabilitation were medium-severity joint deterioration, medium-severity joint faulting, and poor ride.

In 1983 the pavement was first rehabilitated by full-depth patching of deteriorated joints and cracks using the 3-2 doweled patch design. The following spring a number of these patches began to experience deep spalling and pumping. In a few cases, the spalling was so deep that the dowel bars were exposed. As a result, blanket undersealing of existing patches was undertaken in 1984. Several poor, fair, and excellent performing patches were removed to investigate the cause of the deep spalling and why it did not occur in all patches. It was found that the concrete was being overstressed; this was evidenced by large egg-shaped holes around the dowel bars. No evidence of the nonshrink grout used to install the dowel bars could be found in the dowel holes. As a result, a doweled patching experiment was conducted to investigate the impact of dowel bar size, bar arrangement, grout type, and joint sealing on patch performance. This study is detailed in the section on Full-Depth Patching.

Project 4

This project is located near Morris, Illinois, on I-80 between mileposts 105.30 and 111.70. This section of highway has two lanes in each direction. Traffic on this project averages 13,300 vehicles per day with 32 percent trucks. At the time of rehabilitation, the pavement showed no signs of a D-cracking aggregate. The main distresses were medium-severity joint spalling, medium-severity localized distress, and high-severity faulting. Because of the faulting, this section of pavement had a rough ride.

In 1983 this section underwent an ambitious rehabilitation that included full-depth patching using a 3-2 patch design, blanket undersealing of cracks and joints, retrofitted Double Vee load transfer devices, pavement grinding, and installation of underdrains. Another experimental feature of this project, in addition to the load transfer devices, was the use of a drainage mat developed by Monsanto in the place of standard pipe underdrains on a portion of the project.

As a result of poor patch performance, similar to that of Projects 1 and 3, several patches were replaced in 1985 using 10 dowel bars in each joint rather than the 5 or 6 used previously.

Project 5

This project is located on I-280 and I-74 near Rock Island, Illinois, between mileposts 14.70 and 18.00 on I-280 and mileposts 5.00 and 9.64 on I-74. Traffic on the I-280 section averaged 16,400 vehicles per day with 19 percent trucks, and the I-74 section averaged 8,700 vehicles per day with 25 percent trucks. This section of highway has two lanes in each direction and showed no sign of a D-cracking aggregate. Distress before rehabilitation consisted of medium-severity joint spalling, medium-severity localized distress, and low-severity faulting.

The rehabilitation of this section consisted of full-depth patching using a 3-3 patch design, partial-depth patching, joint sealing, blanket undersealing, pavement grinding, and installation of underdrains.

PERFORMANCE OF REHABILITATION TECHNIQUES

Pavement Grinding

Projects 2, 4, and 5 used pavement grinding to improve the riding qualities of the pavement. To remove the severe faulting in Projects 2 and 4, the pavement was ground in the opposite direction of traffic. This was done to better utilize the leveling properties of the grinding machine and resulted in a smoother profile. Project 5 was ground in the direction of traffic because faulting on this project was minor. A limited amount of friction and faulting data was collected before and after rehabilitation.

TABLE 3 FRICTION NUMBER HISTORY OF GRINDING PROJECTS

		Friction Number in						
Project	Route	1980	1981	1982	1983	1984	1985	1986
2	I-55	–	44	–	–	46[a]	41	–
4	I-80	35	–	–	48[a]	40	–	40
5	I-280 and I-74	–	–	–	–	–	42[a]	–

NOTE: Dash means data not available.
[a]After grinding.

TABLE 5 ROADMETER HISTORY OF REHABILITATION PROJECTS (in./mi)

		Year Tested				
Project	Route	1982	1983	1984	1985	1986
1	I-55	–	129[a]	–	–	113
2	I-55	–	165[a]	73	66	78
3	I-70	104[a]	–	–	–	112
4	I-80	151[a]	51	73	64	82
5	I-280 and I-74	–	–	82[a]	40	47

NOTE: Dash means not tested. The following ranges are used to rate ride quality:

RI (in./mi)	*Quality*
Under 75	Very smooth
76–90	Smooth
91–125	Slightly rough
126–170	Rough
Over 171	Very rough

[a]Before rehabilitation.

The friction and faulting data are given in Tables 3 and 4, respectively. Roadmeter readings were taken before rehabilitation and annually thereafter. The roadmeter data are given in Table 5. From the roadmeter data, predictions of the roughness index (RI) were made by plotting the data on log-log paper and extending a best fit line through the points. The results are shown in Figure 4. It is expected that diamond grinding will provide a smooth ride for about 5 years after construction and an acceptable ride for another 5 years.

In isolated areas of Project 2, high reinforcing steel caused spalling and scaling of the pavement surface. Known areas of high steel distress were partial-depth patched in an effort to eliminate the problem. The grinding process removed only a fraction of an inch from these areas but may have caused the spalling to accelerate in areas not patched. Although grinding a pavement with a few isolated areas of high steel should not present much of a problem, caution should be used in grinding a pavement with uniform high steel.

Pavements with D-cracking aggregate should not be ground because this opens the aggregate to water intrusion and can greatly accelerate the D-cracking process. Unfortunately, in Illinois, many miles of pavement have low to moderate D-cracking aggregates and, therefore, care must be taken in selecting pavements to be diamond ground.

Retrofit Double Vee Load Transfer Devices

Project 4 was the only project to use the retrofit load transfer devices shown in Figure 2. Several problems were encountered in the installation of these devices. First, the contract did not call for grooving or roughing the inside of the core hole before installing the device. The manufacturer of the load transfer device provided a tool that applied several grooves to the inside of the core hole. Grooving of the holes provided more surface area for the polymer concrete to bond. Because the contract did not require the grooving, only a small number of the core holes were grooved. These locations were noted for comparison.

The second problem was the polymer concrete used. Improper mixing resulted in a mixture that did not take a final set for several days. When mixed properly, the polymer concrete was almost unworkable. Prewetting the coarse aggregate resulted in an acceptable product.

TABLE 4 AVERAGE JOINT FAULT HISTORY OF GRINDING PROJECTS (in.)

		Year Measured			
Project	Route	1983	1984	1985	1986
2	I-55	0.31	0.00[a]	0.03	0.06
4	I-80	0.18	0.00[a]	0.04	–
5	I-280 and I-74	–	0.00[a]	0.01	–

NOTE: Dash means data not available.
[a]After grinding.

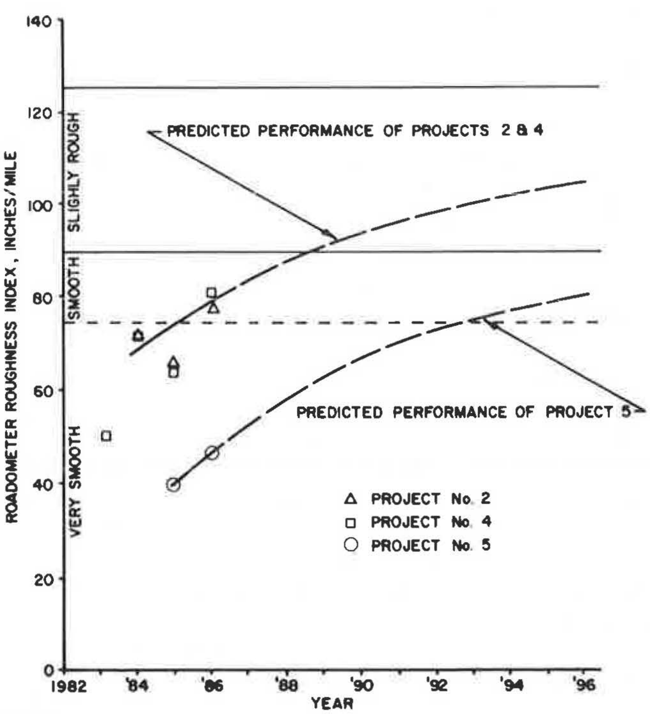

FIGURE 4 Predicted roadmeter roughness index.

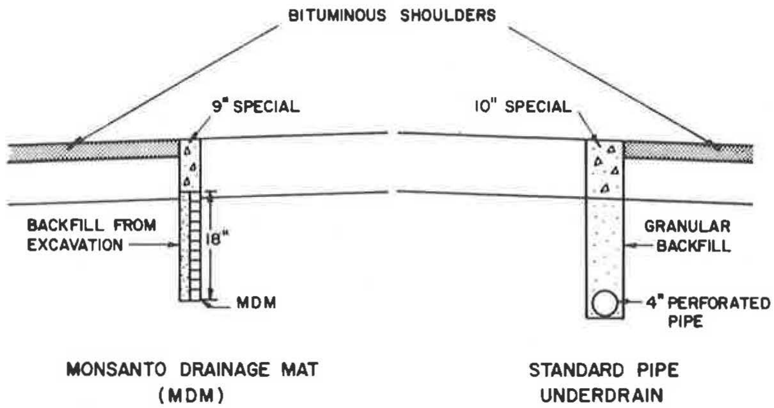

FIGURE 5 Details of Monsanto drainage mat and standard pipe underdrains.

Within a few months of installation, more than 50 percent of the devices debonded at the pavement-polymer concrete interface. There appeared to be no pattern to the debonding in that it occurred on either or both sides of the joint or crack. The grooving of the core hole had no influence on debonding. By 1985 all 671 devices installed have failed by debonding.

Underdrains

Underdrains were installed on all projects except Project 3, in which underdrains had been installed at the pavement edge in previous construction.

On Project 4, a new drainage product was included, the Monsanto drainage mat (MDM). Figure 5 shows a comparison of the design of standard pipe underdrains and the MDM. Sections of the pipe underdrains and MDM were monitored by use of a tipping bucket device that measures outflow with time. Figure 6 shows the typical outflow characteristics of the two underdrains.

The outflow characteristics of the MDM drain showed a desirable improvement in two areas. First, the MDM removed 1.11 to 1.87 times the water of the standard pipe underdrain. Second, the MDM removed the water from the pavement faster, whereas the standard drain continued to flow for several days.

Since first installed on this project, it has been found that a 12-in. mat will perform as well as the standard pipe underdrains. The reduction in mat size, along with the elimination of the granular backfill, has made the MDM quite competitive with pipe underdrains in cost. More long-term research is planned to determine the effectiveness of these and other types of underdrains.

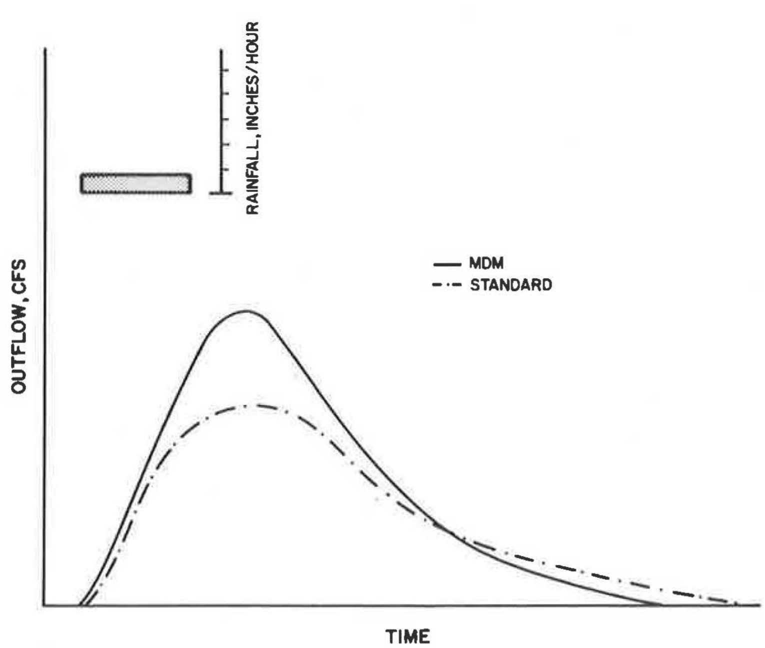

FIGURE 6 Typical outflow characteristics of Monsanto drainage mat and standard pipe underdrains.

Partial-Depth Patching

Projects 2 and 5 used partial-depth patching to repair spalls at joints, and in Project 2 it was used to repair areas of high steel that were exposed. Patches on both projects are performing exceptionally well and are expected to last the life of the pavement. The only problem found was that more areas should have been repaired using partial-depth patching on both projects. Since rehabilitation, several areas of spalls at joints on Project 5 and spalls due to high steel in Project 2 have shown up. Because these areas were outside the condition surveys, it is not certain if any visual evidence of distress was present at the time of rehabilitation. If these areas showed no visual distress, perhaps delamination detection techniques such as those used on bridge decks could be used to detect delaminations at joints.

Full-Depth Patching

All projects included full-depth patching with the percentage of patching ranging from 0.2 to 4.5 percent of the total pavement area. Projects 1, 2, 3, and 4 were patched in 1983 using a 3-2 patch design as detailed in Figure 3. The following spring (1984), Projects 1 and 3 experienced severe spalling of a number of patches. Along with the spalling, evidence of pumping could be seen. When heavy trucks passed over the patches, a rocking motion could be visually detected.

Detailed surveys of patch performance on Project 3 showed that about an equal number of patches fell in the good, fair, and poor categories. There was no apparent correlation between cut, fill, well-drained or poorly drained areas and patch performance. The spalling was limited to the new patch and rarely occurred in the original pavement. Several concrete cores were taken in good, fair, and poorly performing patches, but testing showed no significant strength differences in the samples.

The survey showed that the approach side of the patches had the greatest amount of spalling. On closer investigation, it was found that the approach joint of the patch (first joint to be crossed by traffic) was typically tight and, in spalled areas, the joints were closed. The opposite or leave joint would be $1/16$ to $1/8$ in. wide with little or no spalling.

As a result of problems with the 1983 3-2 patches, an extra dowel bar was placed in the inner wheelpath for 1984 construction as detailed by the 3-3 patch design in Figure 3. Project 5 used a 3-3 patch design and also included a joint seal at all joints. The 3-3 patch design was also used on Project 1 (southbound lanes only) in 1984 when a few badly distressed patches were replaced.

To better understand the effects of dowel bar arrangement, size, and grout used in dowel bar holes, a patching experiment was incorporated in Project 3 in 1984. The experiment called for removing 28 patches from good, fair, and poor performing groups. All patches were selected in the driving lanes of the roadway; 8 patches were in the westbound driving lane and 20 patches were in the eastbound driving lane.

Because the patches that were to be removed were 4 ft long, sawing requirements were such that the new patch would have to be 6 ft long. Patches were removed using the lift-out method. As patches were removed, the dowel bars were found to be lying loose in an egg-shaped hole. The holes measured about $1^3/_8$ in. horizontally, as drilled, but would be larger in the vertical direction. The egg-shaped hole occurred in both the old pavement and the patch. No evidence of the nonshrink grout could be found in any of the patches. Also, the epoxy coating used on dowel bars to prevent corrosion was chipped off or debonded for several inches near the center of the dowel bars. All evidence indicates that the concrete was being overstressed at the dowel bars in 3-2 patches with $1^1/_4$-in. dowel bars.

Variables in the experiment were selected such that some patch designs would be underdesigned and some overdesigned. The patching experiment investigated two dowel bar sizes, three dowel bar arrangements, tied joints, nonshrink grout, and an epoxy grout. The experimental features are given in Table 6 by number of patches constructed in each design variable. Another feature of the experiment was transverse joint seals that were included on all patches, as well as undersealing the old pavement near the patch joint. Design details of experimental patching are shown in Figure 7.

Evaluation of the patches was by visual inspection and deflection testing using an FWD. The visual inspection indicated that all of the experimental patches were in excellent condition, and there was no indication of spalling or any other distress. Deflection testing results are present in terms of a deflection due to a 9-kip load and load transfer across the patch joints in the outer wheelpath. Load transfer is measured by dividing the deflection on the unloaded side of a joint by the deflection on the loaded side and is reported as a percentage. Testing was conducted before the experimental patches were undersealed and again after undersealing before the roadway was opened to traffic. After the roadway was opened to traffic the patches were tested periodically. Deflection and load transfer results for dowel bar size are shown in Figures 8 and 9, and

TABLE 6 NUMBER OF PATCHES CONSTRUCTED BY VARIABLE IN PATCHING EXPERIMENT

	Dowel Design					
	3-3	4-4		5-5		Tied[a]
Dowel Size (in.)	Nonshrink Grout	Nonshrink Grout	Epoxy Grout	Nonshrink Grout	Epoxy Grout	Nonshrink Grout
1.25	4	3	1	4	1	3
1.50	4	4	–	4	–	–

NOTE: Dashes mean not constructed. All patch lengths are 6 ft.

[a]Tied patch design consists of No. 8 tie bar on approach joint and 1.25-in. dowel bar on leave side of patch.

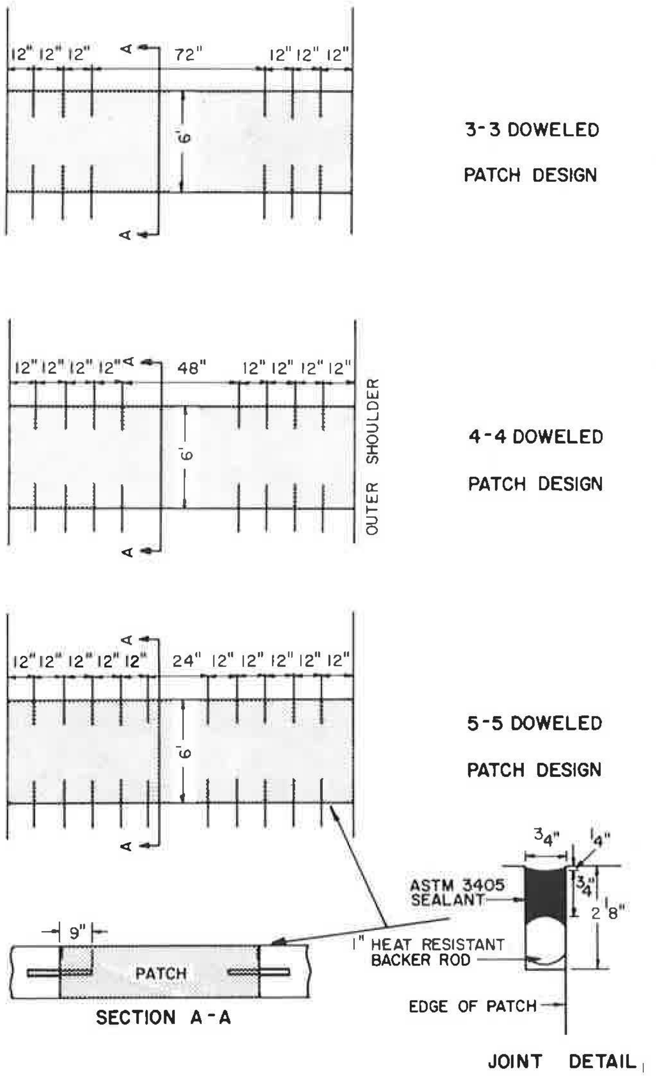

FIGURE 7 Experimental patch details.

results for dowel bar arrangement are shown in Figures 10 and 11. Also shown on the deflection plots are center of slab tests, which are taken at least 6 ft away from any joints or cracks and give an indication of subgrade support. At the end of the evaluation, all deflections and load transfers indicated that the patches were performing outstandingly.

The tied patches showed the best performance with respect to deflection and load transfer. Patches with 1.5-in. dowel bars and a 5-5 dowel bar arrangement performed second best. Although patches with 1.25-in. dowel bars and a 3-3 dowel bar arrangement performed the worst relative to the other designs, the deflection measurements and load transfer percentages were still good. Grout type appeared to have no influence on deflections or performance. It is thought that the sealed transverse joint is very important for a successful patch, not to keep water out of the patch but to prevent spalling. When 1.5-in. dowel bar, a 5-5 dowel bar arrangement, a sawed and sealed joint, and a 6-ft minimum patch are used, a low maintenance life of 10 or more years can be expected compared with 1 to 3 years for the original 3-2 patch design.

Joint Sealing

Project 5 was the only project to incorporate the complete sealing of all joints and cracks. Projects 3 and 4 used joint sealing but only at newly constructed patches, mainly for spall prevention. All joint sealing was done using a hot poured joint sealant in accordance with ASTM 3405.

In Project 5 existing cracks and joints were routed to a depth of 1 in. and a width of approximately 5/8 in. The crack or joint was then sandblasted and blown clean of dust before sealing. No backer rod or tape was required by the contract.

After the first winter it was noted that the sealant had failed by pulling away from one side of the concrete joint in full 100-ft panels. In areas where joints had been patched, the sealant

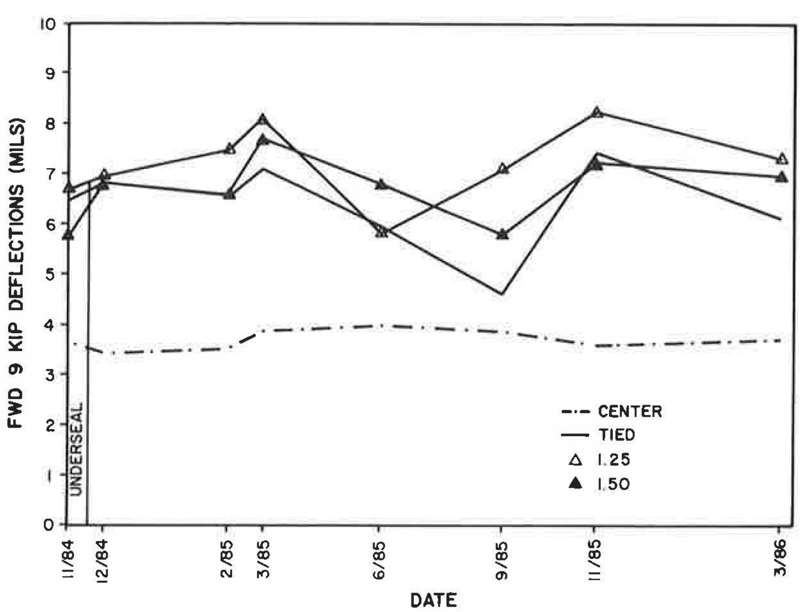

FIGURE 8 Deflection history of dowel bar size.

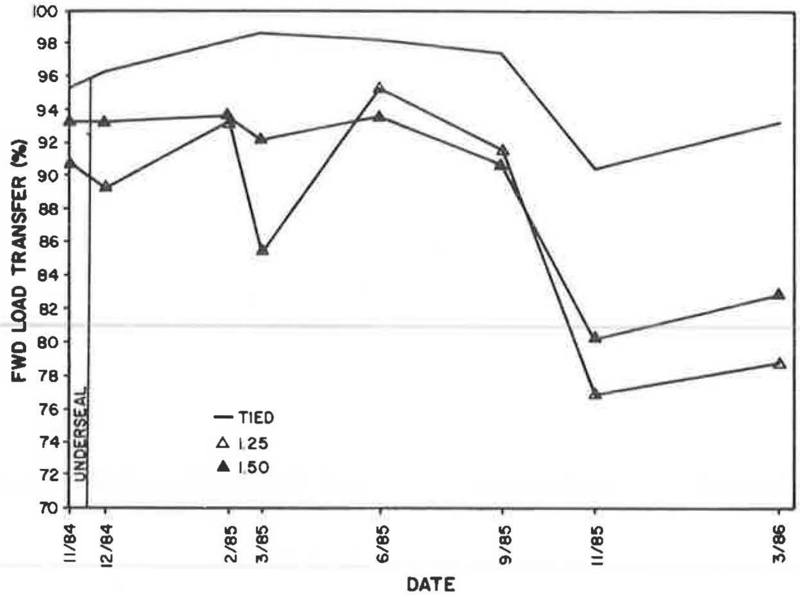

FIGURE 9 Load transfer history of dowel bar size.

was intact and performing well. This indicates that the joint reservoir did not have a properly designed shape factor for the amount of contraction in 100-ft panels.

Blanket Undersealing of Joints and Cracks

Blanket undersealing was part of the original rehabilitation on Projects 4 and 5. An undersealing experiment (*1*) on Project 2 compared grouts using fly ash and limestone as an aggregate as well as the effects of admixtures such as superplasticizers and water-reducers. Also investigated was the use of different pumping pressures, namely 10, 20, and 30 psi.

From the experiment, little difference was found between 20- and 30-psi pumping pressures, but the time required for injection at 10 psi was considerably longer. Fly ash grouts were found to be stronger and did a better job of reducing pavement deflection.

In several cases undersealing with limestone grouts actually increased pavement deflection. When fly ash grouts were used in areas of initial low deflection, minimal improvement in deflection was noted. Removal of four slabs after undersealing verified the increased flowability of fly ash grouts. Fly ash grouts spread out and covered significantly more void space than did the limestone grouts.

Blanket undersealing on Projects 4 and 5 was monitored by

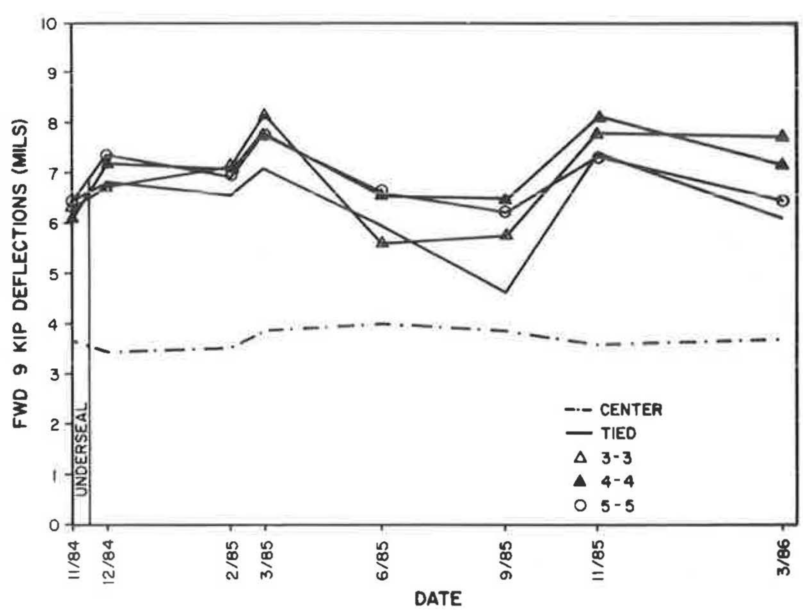

FIGURE 10 Deflection history of dowel bar arrangement.

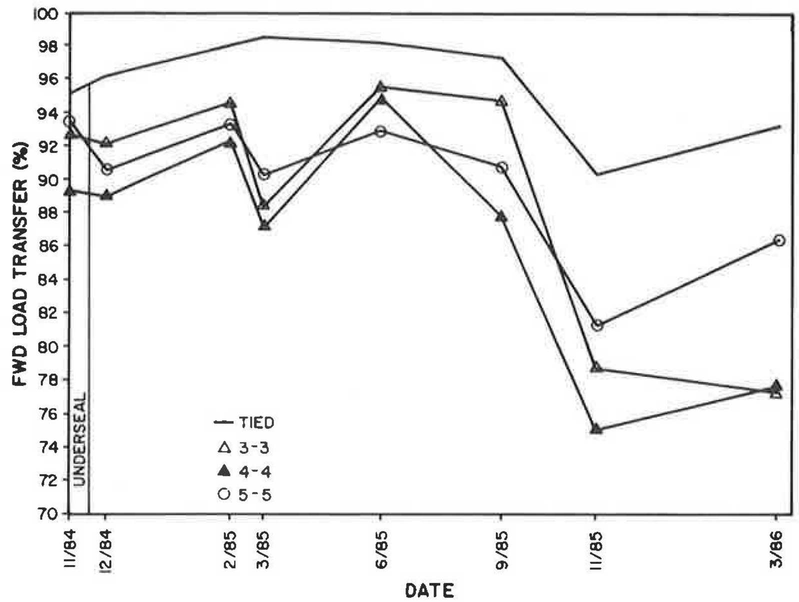

FIGURE 11 Load transfer history of dowel bar arrangement.

deflection testing before undersealing, after undersealing, and then periodically. On Project 4, the department's Road Rater 2008 was used to apply an oscillating 8-kip peak-to-peak load at a frequency of 15 Hz to the pavement. The deflection histories of outer wheelpath deflection and center of slab deflection of Project 4 are shown in Figure 12. The figure shows a reduction in deflection after undersealing, but the reduction is so small that it is doubtful whether the undersealing produced any benefits. The effects of subgrade support, which are reflected in the center deflection, appear to have more influence on deflections than does undersealing in this case.

On Project 5, a Dynatest 8002 FWD was used along with void detection procedures (*2*). In brief, the procedures for void detection are:

1. Three load ranges are applied to the approach and leave sides of joints and cracks. (Typically, these loads are 4, 8, and 12 kips or 6, 9, and 12 kips.)
2. Plots of load versus deflection are made for each test site.
3. A best-fit line is drawn through the points and extended to the axis.

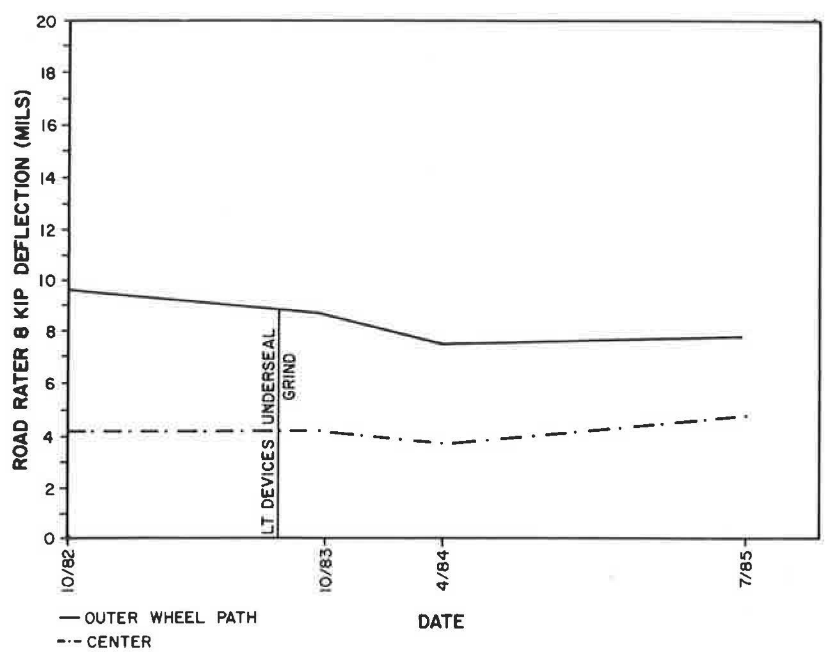

FIGURE 12 Deflection history of undersealing on Project 4.

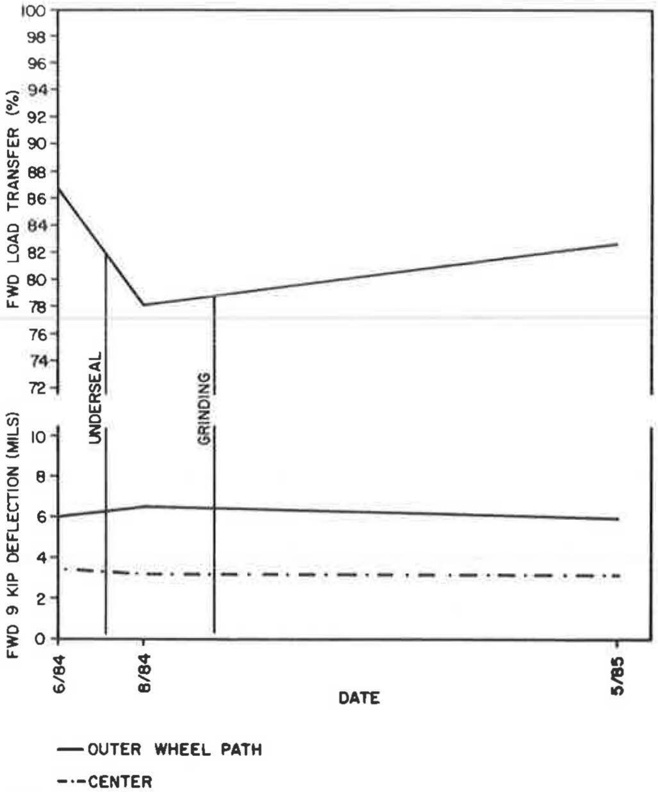

FIGURE 13 Deflection and load transfer history of undersealing on Project 5.

4. If the line intersects the deflection axis at a number greater than 0-1 mil, a void is present.

Also compared in the procedure are load transfer efficiency, 9-kip deflection, and center of slab deflections. By using these procedures, only 2 voids were found in the 53 joints and cracks tested before undersealing. Tests after undersealing and the following year showed that the voids were filled and remained stable. The deflection and load transfer histories are shown in Figure 13.

From the deflection graphs for Projects 4 and 5, it can be seen that little benefit was gained from blanket undersealing. To be effective, undersealing must be done on a selective basis only at locations of known voids.

Blanket Undersealing of Pumping Patches

In Project 1, nine patches, which were pumping and rocking under traffic, were selected for undersealing in conjunction with rehabilitation on Project 2 in 1983. Deflection testing, using the Road Rater, was conducted before and after undersealing. Deflections were greatly reduced, but continued testing showed that within 1 year deflections had nearly reached preundersealing levels. These patches, along with all other patches in Project 1, were undersealed again in the fall of 1984. The Road Rater deflection history is shown in Figure 14.

Project 3 was also undersealed in the fall of 1984 at all patch locations to arrest pumping and spalling of the patches. Before undersealing, a group of 21 patches was selected for evaluation. Deflection testing results using the FWD are shown in Figure 15, which again shows that patch undersealing was effective for about 1 year.

DOWEL BAR GROUTING EXPERIMENT

A laboratory experiment was conducted to resolve problems encountered when grouting in dowel bars. To evaluate the different techniques, a number of "hole" specimens were made by casting a polyvinyl chloride (PVC) pipe, with a 1⅝-in. outside diameter, into a concrete cylinder 6 in. in diameter and 12 in. high. A bond breaker was also cast into each specimen in order that the cylinder might be split in half along the dowel bar

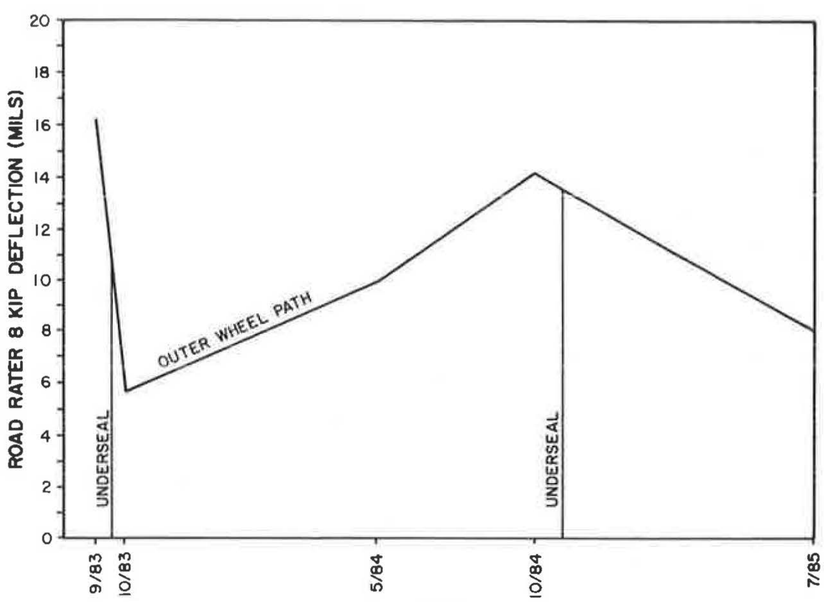

FIGURE 14 Deflection history of patch undersealing on Project 1.

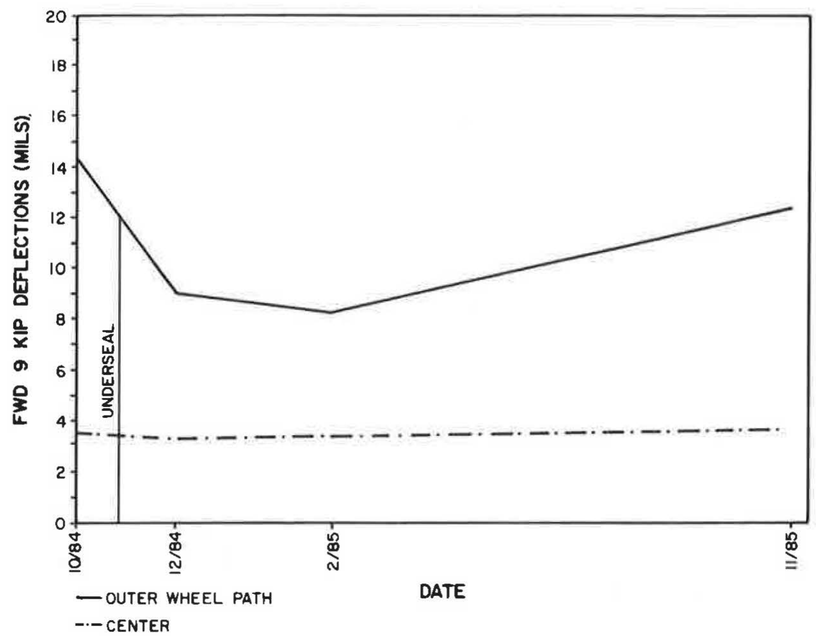

FIGURE 15 Deflection history of patch undersealing on Project 3.

after it had been grouted and cured. Cylinders were laid on their sides and different grouting techniques, along with different grout types and consistencies, were used to grout in 1½-in. dowel bars.

The following techniques and grouts were used:

- PVC pipe sleeve: Consists of a 9-in. section of 1⅝-in. inside diameter PVC pipe. The dowel bar is inserted about 3 in. into the pipe, then the pipe is filled with grout. The sleeve is then held against the pavement over the hole and the dowel bar is inserted through the sleeve into the hole, pushing the grout ahead of the bar. The sleeve is then pulled off the end of the dowel bar.
- Grout bag: This consisted of a vinyl bag similar to a pastry bag except that, in the bottom, a ¾-in.-diameter PVC pipe, 12 in. long with a reducer on the end, was used to extrude the grout. The pipe was inserted and grout deposited at the rear of the hole. The dowel bar was then inserted.
- Grout gun: This is a commercially available device that is similar to a caulking gun, except that the nozzle was extended with a 9-in. length of tubing. The nozzel was inserted and grout was deposited at the rear of the hole. The dowel bar was then inserted.
- Push rod and half sleeve: The push rod was made from a thread rod on which a 1.5-in. rubber "washer" was secured by metal washers and nuts. The half sleeve was made from a piece of sheet metal fastened to fit into the 1⅝-in. hole. The sleeve was filled with grout, inserted into the hole about an inch, and then the rod was used to push the grout to the rear of the hole. Last, the dowel bar was inserted.
- Grout pump: Grout is mechanically pumped through a hose into dowel bar hole by means of a commercially available grout pump. An on-off switch at end of the applicator allows high production. A mixer is also available that will mix grout and pour it into the pump. After the grout is applied, the dowel bar is inserted.
- Hilti Hit C-10 resin system: A two-component polyester resin is packaged in a caulking gun type of applicator that mixes components while applying resin in the rear of the hole. After application, the dowel bar is inserted. The resin has rapid strength gain.

Sand and silica nonshrinking grouts were evaluated for their ability to remain in the hole without excessive run out. The sand grout used worked well over a wide range of water contents. The silica grout was quite flowable, even at extremely low water contents, and continually ran out of the hole. Because of the run out problem, only sand grouts are recommended for dowel bar grouting. The sand grout was then used to evaluate the various grouting techniques.

When dowel bars had been grouted in and allowed to cure overnight, the specimens were split in half, the dowel bar removed, and a visual inspection made of the quality of coverage. Good coverage of grout was achieved with techniques that deposited the grout at the rear of the hole before the dowel bar was inserted. When using the PVC pipe sleeve, which uses the dowel bar to push the grout to the rear of the hole, small air voids were trapped on top of the dowel bar. This resulted in a fair to poor quality of coverage. The results of the experiment are given in terms of quality of coverage, production, workability, and material and equipment cost in Table 7.

As a result of the laboratory experiment, it was found that grout should be deposited at the rear of the hole before dowel bar insertion. Dowel bars should not be oiled before insertion. When bars are oiled, the cement is displaced or retarded, or both, around the dowel bar. The thickness of the grout should be such that it does not run out of the hole. It is important to use a back-and-forth twisting motion while inserting the dowel because this will eliminate any air voids. The back-and-forth twisting motion allowed dowels to be inserted easily, even when very dry grouts were used. There is no need to "drive" the dowel bar into the hole with a hammer unless the hole is out

TABLE 7 DOWEL BAR GROUTING TECHNIQUE RESULTS

Method	Quality of Coverage	Production	Workability	Approximate Nonshrink Grout Cost ($/hole)	Approximate Equipment Cost ($)
PVC pipe sleeve	Fair	Good	Good	0.10	0.30
Grout bag	Very good	Good	Good	0.10	5.00
Grout gun	Very good	Poor	Poor	0.10	40.00
Push rod and half round	Very good	Good	Good	0.10	5.00
Grout pump with mixer	Very good	Excellent	Excellent	0.10–0.15	1,500–2,600
	Very good	Excellent	Excellent	0.10–0.15	5,000
Hilti Hit C-10 resin	Very good	Good	Good	4.00[a]	0.00

[a]Two-component polyester resin.

of alignment or very dirty. Nonshrink grout properties are such that initial set does not take place for 2 hr and final set takes about 7 hr. For early-open patches, those to be opened to traffic in less than 24 hr, a high early strength resin or epoxy should be used.

COST-EFFECTIVENESS OF REHABILITATION

The actual bid prices for the five projects were converted to cost per two-lane mile and are given in Table 8. For patching, the percentage of pavement patched is also reported. Costs of experimental features were not included because these costs were not representative because of the small amount of work involved or work inexperience. The cost of the Double Vee load transfer devices was included only for informational purposes and is not included in the project total.

One form of rehabilitation that is used on Interstates in Illinois is to resurface with 3 in. of asphalt concrete. This consists of applying a prime or tack coat, prime coat aggregate, 1.5 in. of binder course, and 1.5 in. of surface course. The cost of 1 mi of resurfacing on 6 ft of inside shoulder, 24 ft of pavement, and 10 ft of outside shoulder averaged $124,500 in 1983, the year most of the projects were first rehabilitated. With traffic control and mobilization, the total cost for resurfacing 1 mi of Interstate was $137,000. Whether the pavement is resurfaced or not, full-depth patching and undersealing would be about the same in quality and cost because the procedures for determining patching and undersealing needs are the same. Underdrains are installed on all Interstate projects that have not had underdrains installed previously, so this cost would also be the same.

Items that are unique to CPR are partial-depth patching, joint sealing, and pavement grinding. When the cost of these techniques is compared with that of resurfacing, it is seen that the cost of CPR is about 50 percent of that of resurfacing.

CONCLUSIONS

Jointed concrete pavement can be rehabilitated cost-effectively. Pavement grinding of the projects evaluated is expected to provide good ride quality for 10 or more years. Improved full-depth patch design and procedures have resulted in greatly improved performance. Important features of the improved design are the use of 10 dowel bars, which are 1.5 in. in diameter, per joint; the use of a sawed and sealed joint; and a minimum patch length of 6 ft. The improved design is expected to give about 10 years of service. Another important requirement of full-depth patching is close inspection of dowel bar grouting. Grouting techniques that deposit grout at the rear of the drilled hole are best. Partial-depth patching has performed exceptionally well, but there is a need to locate and patch potential areas of spalling or delaminations to reduce pavement maintenance cost after rehabilitation. Underdrains will remove water from the pavement. More research is planned to deter-

TABLE 8 REHABILITATION TECHNIQUE COST ($/lane-mile)

Project	Route	Year of Rehabilitation	Full-Depth Patching	Partial-Depth Patching	Joint Sealing	Blanket Under-sealing	Retrofit Load Transfer Devices	Pavement Grinding	Pipe Under-drains	Traffic Control and Mobilization	Project Total
1	I-55	1983	42,910/4.5[a]	–	–	–	–	–	12,700	9,750	65,360
		1984	3,540/0.2	–	–	13,140	–	–	–	3,330	20,010
2	I-55	1983	4,340/0.3	8,740/0.4[a]	–	–	–	43,110	9,930	8,710	74,830
3	I-70	1983	38,300/3.9	–	–	–	–	–	–	6,600	44,900
		1984	–	–	–	7,210	–	–	–	1,820	9,030
4	I-80	1983	8,080/0.7	–	–	15,290	25,540[b]	36,700	20,220	6,690	86,980
		1985	10,260[c]/0.7	–	–	–	–	–	–	2,710	12,970
5	I-280	1984	26,070/2.4	2,570/0.2	5,870	16,220	–	45,590	33,010	18,930	148,260

[a]Percentage of pavement patched.
[b]Based on two devices per wheelpath and average spacing of 100 ft; presented for information only and not included in total.
[c]5-5 doweled patch design.

mine if underdrains are cost-effective. In the future, undersealing should only be done on a selective basis where voids are known to exist at cracks and joints. Undersealing of distressed patches is not cost-effective. Debonding failures of load transfer devices in Illinois and other states (*3*) indicate the need for more laboratory research in this area. When joints are resealed, the reservoir shape must be designed properly to prevent debonding of the sealant.

RECOMMENDATIONS

- Only sound, non-D-cracking pavements should be rehabilitated using CPR methods.
- The cost-effectiveness of pavement grinding can be further increased by grinding only the driving lane on Interstate routes.
- Delamination detection techniques should be used as an aid in locating areas in need of partial-depth patching.
- Full-depth patches should be constructed using 10 dowels with a diameter of 1.5 in. in each joint.
- When dowel bars are grouted in, the grout should be deposited at the rear of the hole and the dowel bar inserted using a back-and-forth twisting motion. The grout should be thick enough so there is no appreciable run out.
- A sawed or formed and sealed joint should be used in full-depth patching to prevent spalling.
- Void detection is necessary to determine the need for and locations of undersealing. Blanket undersealing is rarely needed.
- Spalled, rocking, and pumping patches should be replaced rather than undersealed.
- Future use of Double Vee or similar load transfer devices should be on a limited experimental basis to determine performance.
- More research is needed to determine the effectiveness of underdrains.
- Joint sealant reservoirs should be designed to provide the proper shape factors for the sealant and movement needs.

ACKNOWLEDGMENT

This paper is based on the results of Project IHR-514—Performance Evaluation of Jointed Concrete Pavement Rehabilitation Without Resurfacing. IHR-514 was sponsored by the Division of Highways, Illinois Department of Transportation, and the Federal Highway Administration, U.S. Department of Transportation.

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Rutting of Asphalt Concrete Overlays on Continuously Reinforced Concrete Pavements in Texas

C. L. SARAF, B. F. MCCULLOUGH, AND M. F. ASLAM

Rutting history data on asphalt concrete pavement (ACP) overlays on rigid pavements are being collected by the Center for Transportation Research (CTR) to study ACP overlay behavior under the traffic and environmental conditions of Texas. The available data were analyzed recently for this purpose. Overlaid sections located in three counties of the state were selected for this study. These sections were originally built as continuously reinforced concrete pavement (CRCP). Using the limited data available at the present time, it was observed that the rate of rutting was maximum in the first year because of the initial compaction of material in the wheelpath. In the second year, the material between the wheelpaths experienced more compaction than that in the wheelpaths themselves, and therefore rutting was observed to decrease in the second year. However, rutting increased in the years following full compaction of the lanes. A regression equation was developed to characterize the rutting behavior of ACP overlays on CRCP. The analysis of available data indicated that overlay thickness was an important predictor of rutting in overlays. The age of the overlay was not very significant in the regression equation. This may be due to the brief history of rutting data available at the present time. The rutting of the overlays in different counties was affected by the locations of the overlaid sections. Apparently the materials of construction and construction-related items, which may be different in each county, affected the performance of overlays.

The state of Texas has constructed several thousand miles of continuously reinforced and jointed concrete pavements. Some of these pavements were built in the early 1960s, and those pavement sections that have needed rehabilitation have been overlaid primarily with asphalt concrete pavements (ACPs).

The rigid pavement sections that have been overlaid with ACP and thin bonded portland cement concrete (PCC) are being monitored by the Center for Transportation Research (CTR) to study their performance and rutting behavior. The objective of the study described in this paper is to characterize the rutting of ACP overlays on CRCP.

The current data base on the rutting history of the ACP sections was initiated in 1979. Data for a total of about 100 sections have been recorded in the data base so far. A portion of the overlaid section approximately 100 ft in length was selected to represent the entire section, which is of the same thickness. Rut depths were measured at 10-ft intervals along this section and the average of the 10 readings was recorded as the average rut depth for the section. A typical printout of the current data base is shown in Figure 1. The data base contains section identification and average rut depth measurements.

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LOCATION OF OVERLAID SECTIONS

The approximate locations of the ACP overlaid sections that are being monitored by CTR are shown in Figure 2. These sections are located on I-10, I-20, I-35, and I-45. Because many overlaid sections included in the data base did not have enough rutting history, it was decided to analyze the data from overlaid sections located in Walker, Falls-McLennan, and Jefferson counties only. The base slab for all of these overlays was a continuously reinforced concrete pavement (CRCP). A summary of information related to overlaid sections selected for this study is given in Table 1.

COLLECTION OF RUT DEPTH DATA

The rut depths of the ACP overlays were measured with the help of the device shown in Figure 3. This device uses a linear variable differential transformer to measure the rut depth in the wheelpath (approximately 3 ft away from the pavement edge). Measurements were taken at 10-ft intervals on a section approximately 100 ft long, and the average of the 10 readings was recorded as the rut depth for the section. Typical averages of rut depth measurements are shown in Figure 1.

ANALYSIS OF RUT DEPTH DATA

For the data analysis, the rut depth data were summarized and tabulated (Table 1).

Jefferson County rut depth data are plotted in Figure 4. These plots show the relationship between the rutting of overlays and age. Similar plots of Falls-McLennan County data are shown in Figure 5. The effect of overlay thickness on rutting in Walker County is shown in Figure 6.

Because the plots of rut depth data indicated that rutting is affected by the overlay thickness and the age of the overlaid section, a regression analysis of the data given in Table 1 was performed. Rutting data for ages less than 2 years were not included in the analysis because sections located in only one county (Jefferson) were measured at ages 0 and 1 year. The resulting regression equation is

$$Y = 0.009 + 0.021X_1 + 0.014X_2 + 0.091X_3 + 0.141X_4 \quad (R^2 = 0.72, S = 0.0465) \qquad (1)$$

where

y = rut depth of overlay (in.);
X_1 = overlay thickness (in.);
X_2 = age of overlaid section (years); and

X_3, X_4 = county identifiers:

$X_3 = 0$ and $X_4 = 0$, if Walker County,
$X_3 = 1$ and $X_4 = 0$, if Jefferson County, and
$X_3 = 0$ and $X_4 = 1$, if Falls-McLennan County.

The t-ratios for the coefficients associated with $X_1, \ldots, X_4$ indicated that overlay thickness is a significant predictor of rut depth. Also, the rutting in each county (predicted by X_3 or X_4) is significantly different for any given overlay thickness and age. The age of overlay section did not show a significant effect on rutting in the set of data used for this analysis.

```
  9    15    2    18  902IH-35 SB     FALLS-MCLENNAN
326+25   267+00
                    MAY 81    .217
                    MAR 83    .275
                    MAR 84    .288
267+00   230+00
                    MAY 81    .306
                    MAR 83    .338
                    MAR 84    .352
230+00   214+50
                    MAY 81    .180
                    MAR 83    .231
                    MAR 84    .210
214+50   189+00
                    MAY 81    .241
                    MAR 83    .306
                    MAR 84    .310
189+00   176+50
                    MAY 81    .348
                    MAR 83    .442
                    MAR 84    .471
176+50   134+00
                    MAY 81    .290
                    MAR 83    .312
                    MAR 84    .298
  9    15    2    18  902IH-35 NB     FALLS-MCLENNAN
140+00   195+00
                    MAY 81    .287
                    MAR 83    .358
                    MAR 84    .375
195+00   326+25
                    MAY 81    .319
                    MAR 83    .369
                    MAR 84    .366
  9    15    3    10  901IH-35 SB     FALLS-MCLENNAN
134+00   110+00
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                    MAR 83    .399
                    MAR 84    .405
110+00    70+00
                    MAY 81    .241
                    MAR 83    .268
                    MAR 84    .268
 70+00    00+55
                    MAY 81    .313
                    MAR 83    .373
                    MAR 84    .366
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                    MAR 83    .255
                    MAR 84    .246
 55+00    68+00
                    MAY 81    .236
 68+00   107+00
                    MAY 81    .323
                    MAR 83    .383
                    MAR 84    .396
107+00   140+00
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FIGURE 1 Typical printout of current data base.

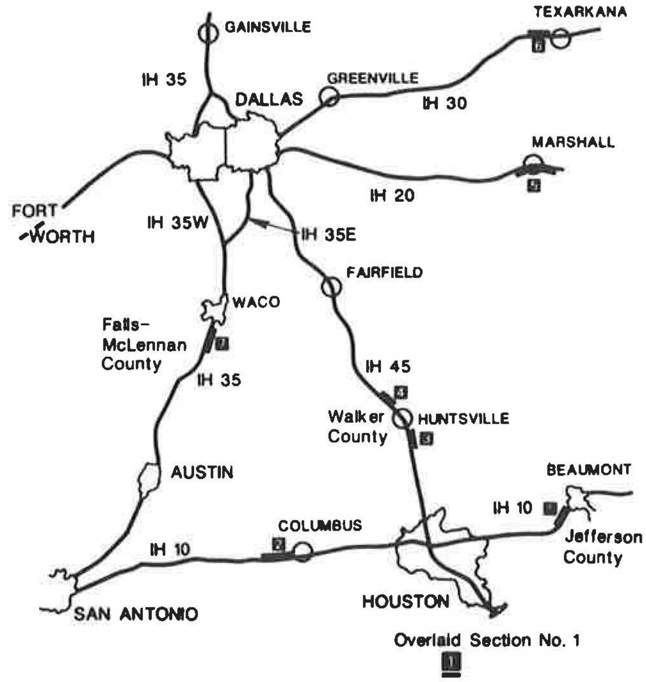

FIGURE 2 ACP overlaid sections monitored by CTR.

DISCUSSION OF RESULTS

Figure 4 shows the rutting history of overlaid sections in Jefferson County. The individual plots of rutting history for overlays of different thicknesses (3.25, 4, and 6 in.) indicate that the rutting in new overlays develops at a faster rate than in older overlays (more than 2 years of age). Also, after the first year's service, rutting tends to decrease before it starts increasing again, after 2 years. A possible reason for this phenomenon is that in the beginning the new overlay is subjected to compaction of material in the wheelpaths under the wheel loads of traffic in addition to the permanent deformation of fully compacted material. Therefore the rate of rutting in the first year is higher than in following years. Because the material in the wheelpath may be fully compacted during the first year, the increase in rut depth during the next year will be a fraction of the total rut depth during the first year. Also, it is likely that the high ridges developed around the center of the wheelpath during the first year may start subsiding because of compaction of this part of the cross section. The major portion of traffic that will cross this part of the roadway is passing traffic, which moves from the outer lane to the inner lane. Although this will happen simultaneously with the compaction of the roadway in the wheelpath, the amount of traffic will always be a fraction of the total traffic passing along the wheelpath. Therefore full compaction of material in the middle of the wheelpath will take place after the wheelpath has been fully compacted. When this happens, the ridges will smooth out and the rut depth will

TABLE 1 SUMMARY OF RUT DEPTH DATA

Section No.[a]	County	First Overlay Thickness (in.)	Overlay Age (years) 0	1	2	3	4	5	6	7
3	Walker	2.5							0.119	0.114
		4.0							0.123	0.136
		5.0							0.256	–
		6.0							0.250	0.302
1	Jefferson	3.25	0.162	0.188	0.179	0.184				
		4.0	0.249	0.258	0.255	0.255				
		6.0	0.248	0.263	0.229	0.259				
7	Falls-McLennan	3.5				0.268	0.339		0.335	
		7.0				0.296	0.333		0.340	

[a]See Figure 2.

actually appear to be less than that measured at the end of the first year.

After a certain period of time (in this case, 2 years), when the entire cross section has been fully compacted by traffic, further rutting in the overlay will be caused by the permanent deformation characteristics of the material. Therefore rutting will increase with time, as shown in Figures 4 and 5 and as indicated by Equation 1 (a positive sign for variable X_1).

It was mentioned earlier that the rut depth data for 0 and 1 years were excluded from the data set used in the development of Equation 1. Because the initial compaction characteristics of materials used in different locations may not be the same, it was decided to use only that portion of the rutting history curve that indicated consistent behavior for all three counties.

Figure 6 shows the effect of overlay thickness on rut depth for Walker County data. An increase in overlay thickness increases the rut depth at any given time. The laboratory studies of permanent deformations in asphaltic mixtures indicated that the permanent strain (ε_p) at the first load application is a function of deviatoric stress (σ_d) and elastic strain (ε_e) *(1)*. Because the rut depth of an overlay is the product of ε_p and layer thickness, it is conceivable that thicker overlays would rut more than thinner overlays of similar materials.

The results of this study, which are based on the analysis of available data, can be summarized as follows:

1. Initial compaction of ACP overlays (by service traffic) produces a high rate of rutting in the first year of service. The rutting of overlays decreases after the first year and starts increasing again after the middle of the wheelpath has also been fully compacted by passing traffic that changes lanes.
2. After the first 2 years of service, rut depth increases with age.

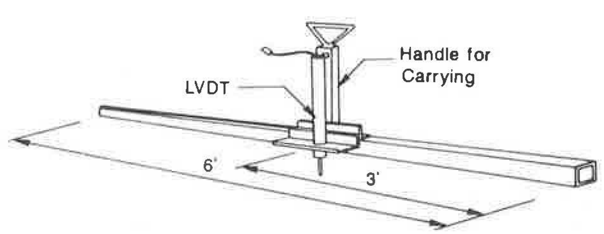

FIGURE 3 Rut depth measuring device.

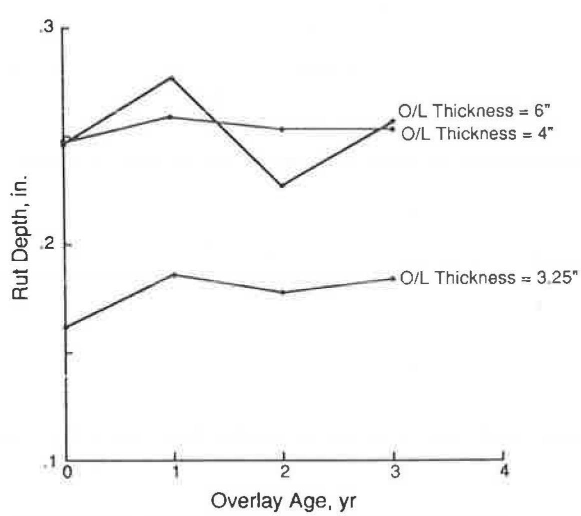

FIGURE 4 Overlay rutting history (Jefferson County sections).

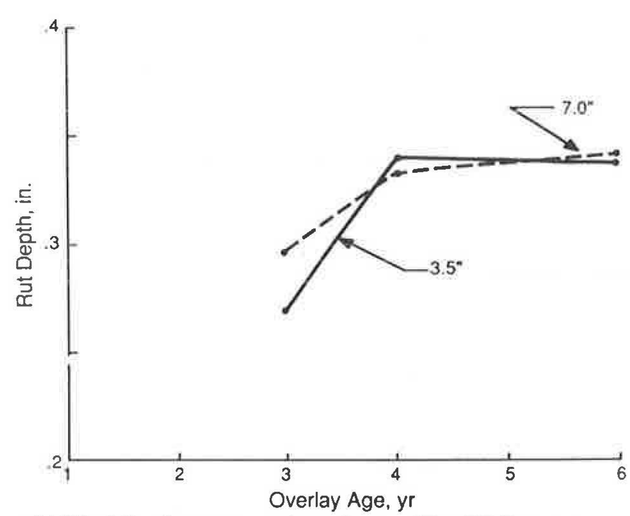

FIGURE 5 Overlay rutting history (Fall-McLennan County sections).

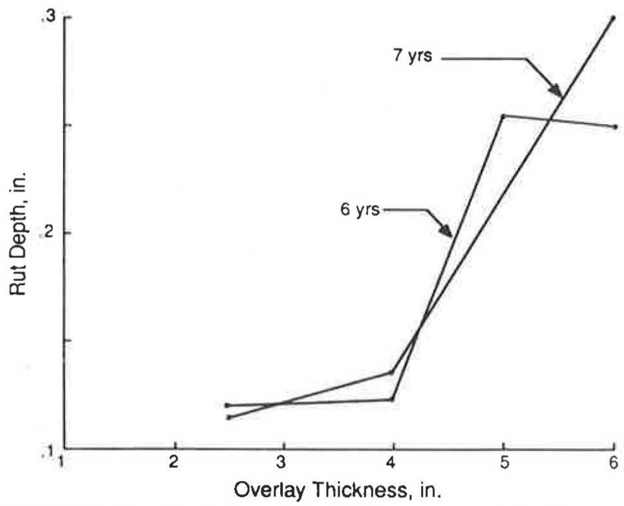

FIGURE 6 Effect of overlay thickness on rut depth (Walker County data).

3. Rut depth increases with an increase in overlay thickness.
4. The materials of construction and construction-related items may control the amount of rutting in overlays.

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A Microcomputer Program To Evaluate Cost-Effective Alternatives for Concrete Pavement Restoration

WAHEED UDDIN, R. FRANK CARMICHAEL III, AND W. RONALD HUDSON

A methodology for evaluating cost-effective alternatives for rehabilitation of pavements that was developed for microcomputer applications is described. The life-cycle cost–1 (LCC1) microcomputer program is designed for comprehensive economic evaluation of competing alternatives provided by users. The LCC1 program is unique for life-cycle cost analyses because of its flexibility and the options it offers users: it creates and saves multiple input files and provides default data, manipulates input data without going through an entire session, offers seven available optimization options for rank ordering the strategies, and considers multiple maintenance and rehabilitation treatments. The user inputs an array of design strategies (for initial construction or rehabilitation design). Several cycles of maintenance and rehabilitation actions can be included in a single strategy. Peripheral cost items like moving guide rails and adjusting drainage structures are also considered. The LCC1 methodology is capable of computing user operating costs and added user costs due to traffic delays during rehabilitation and reconstruction. The present worth or the annualized equivalent annuity method can be used to establish ranking of alternatives. Applications of the LCC1 program to the analysis of various alternative strategies for concrete rehabilitation are presented in this paper.

Several computer programs for life-cycle analysis of pavements are found in the literature for project-level application in surface or rehabilitation type selection *(1–5)*. Most of these programs, however, rely on predictive models for generating alternative design strategies. Moreover, these programs are generally not capable of considering multiple rehabilitation activities. A life-cycle cost analysis involves modeling for several years the performance of a particular structure exposed to a given set of conditions, including expected environment, forecast traffic loadings, selected maintenance treatments, and selected rehabilitation strategies. The analysis is used to evaluate several different rehabilitation or maintenance actions.

Life-cycle cost (LCC) analysis of pavements enables pavement management decision makers to optimize the expenditure of available funds by evaluating the cost-effectiveness of competing rehabilitation strategies. The development of a comprehensive procedure for comparing project-level designs of new or existing pavements based on an optimum life-cycle cost was the primary objective of this study for the Pennsylvania Department of Transportation (PennDOT). The following criteria, established by the sponsors at the outset of the study *(6, 7)*, were used in the development of the methodology: *(a)* The alternative strategies will be user inputs. Performance or distress prediction models will not be used to generate design strategies. *(b)* Multiple options for maintenance and rehabilitation treatments, compatible with standard methods used by PennDOT, will be considered. *(c)* Agency cost models will include peripheral cost items like guide rail relocation, drainage structures, and related shoulder work. *(d)* A model for added user costs due to traffic delays during rehabilitation work must be included in the life-cycle cost procedure. *(e)* Present worth analysis will be used for economic evaluation unless a better and more versatile method is identified.

In this paper the LCC1 microcomputer program that incorporates the developed methodology is described and example applications are presented.

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METHODOLOGY

The LCC methodology developed in this study *(7–9)* considers alternatives for resurfacing existing pavements, concrete pavement restoration, maintenance, rehabilitation cycles, and peripheral activities. The thrust of this LCC methodology is a straightforward economic evaluation. The competing design alternatives are user inputs.

For a new pavement project, the main issues related to initial structural design are surface type and thickness design. The LCC methodology considers several types of pavements for new pavement design or reconstruction. The available options are *(a)* four surface types including asphaltic concrete pavement (ACP), plain jointed concrete pavement (JCP), jointed reinforced concrete pavement (JRCP), and continuously reinforced concrete pavement (CRCP); *(b)* eight types of base course materials including crushed aggregate, dense-graded crushed aggregate, bituminous aggregate, cement aggregate, lean concrete, aggregate lime pozzolan, bituminous concrete, and cement concrete; and *(c)* granular and stabilized subbase materials.

Some of the factors considered in selecting rehabilitation alternatives for a given pavement section are structural capacity; roughness; type, extent, and degree of pavement distress; skid resistance; type of facility; traffic characteristics; subgrade characteristics; and adjoining roads. Resurfacing is a major rehabilitation option. Determining resurfacing thickness requires an evaluation of the existing pavement and the material properties of new overlay or recycled materials.

Maintenance and Rehabilitation Alternatives

Several types of maintenance treatments and rehabilitation actions are considered in the LCC methodology. For life-cycle

TABLE 1 MAINTENANCE TREATMENTS CONSIDERED IN THE LCC METHODOLOGY

Treatment	Applicable to
Cleaning and sealing joints	PCC
Spall repair	PCC
Subsealing	PCC
Slab jacking	PCC
Rigid patching	PCC
Crack sealing	BF/BR
Bituminous patching, manual	BF/BR
Bituminous patching, mechanized	BF/BR
Seal coat/surface treatment	BF/BR
Base repair	PCC/BF/BR

NOTE: PCC = portland cement concrete (rigid pavement), BF = bituminous surfacing over flexible base (flexible pavement), and BR = bituminous surfacing over rigid base (composite pavement).

costing, maintenance and rehabilitation scheduling is provided by users. The selection of these alternatives is primarily based on *(a)* type of existing pavement, *(b)* concrete pavement rehabilitation techniques recommended by PennDOT *(10)*, and *(c)* maintenance treatments considered in the Systematic Technique to Analyze and Manage Pennsylvania Pavements (STAMPP) system of PennDOT *(11)*. Selected maintenance treatments are given in Table 1. Table 2 gives various rehabilitation alternatives considered in the LCC methodology. These include overlays, concrete pavement restoration techniques, and reconstruction. The methodology is designed to handle a combination of rehabilitation techniques under a single strategy.

A survey of PennDOT experts working in various areas (maintenance, construction, design, and administration, for example) was performed in this study. Expected life data for various rehabilitation alternatives were collected for bituminous pavements, portland cement concrete pavements, and bituminous pavements with rigid bases. The results, summarized by Uddin et al. *(8)*, can be used for scheduling maintenance and rehabilitation activities in a design strategy.

TABLE 2 REHABILITATION ALTERNATIVES

Alternative	Applicable to
Milling	Bituminous pavements
Leveling course	Bituminous pavements
Recycling	Bituminous pavements
Scratch course	Bituminous or PCC pavements
Joint rehabilitation	PCC pavements
Spall repair	PCC pavements
Subsealing	PCC pavements
Slab jacking	PCC pavements
Slab replacement	PCC pavements
Diamond grinding	PCC pavements
Recycling	PCC pavements
"Do nothing"	All pavement types
Asphalt concrete overlay[a]	All pavement types
Continuously reinforced concrete overlay[a]	All pavement types
Plain jointed concrete overlay[a]	All pavement types
Jointed reinforced concrete overlay[a]	All pavement types
Reconstruction	All pavement types

[a]Overlays can be considered with or without bond breaker layers.

Peripheral Activities

The LCC methodology also considers a number of peripheral items:

- Construction, maintenance, and rehabilitation of unpaved, asphaltic concrete, surface treated, or portland cement concrete shoulders.
- Moving guide rails, shifting and repairing drainage structures, and three miscellaneous items specified by the user.

COST MODELS

The following cost models are included in the LCC methodology: construction costs, maintenance costs, rehabilitation costs (including overlay and reconstruction costs), peripheral costs (including maintenance and rehabilitation costs), cost of maintenance and protection of traffic, user costs due to traffic delays, user operating costs, salvage value, and value of extended life. All types of unit costs and fixed cost components are user inputs. However, default values for these cost components are provided in the computer program.

Construction Costs

The initial construction cost for a new pavement is the sum of the cost of right-of-way, the cost of engineering and surveying, the mobilization cost, the cost of subgrade preparation, the cost of subbase and base construction, the cost of surfacing, and peripheral costs. The cost for asphaltic concrete surfacing consists of the costs for prime coats, tack coats, and asphaltic concrete. The cost for a portland cement concrete pavement includes the placement cost, steel reinforcement cost, longitudinal joint cost, and cost of transverse joints (if jointed concrete pavement is specified).

Maintenance, Rehabilitation, and Peripheral Costs

The maintenance cost is a function of a fixed unit cost and a variable unit cost. The periodically updated computer printouts of costs, based on maintenance performance standards and annual maintenance expenditure summaries prepared by an agency, are excellent sources of unit costs for various maintenance activities. The methodology also considers annual routine maintenance cost as a function of surface type. Examples of this type of unit cost are $825 per lane-mile per year for rigid pavements and $1,825 per lane-mile per year for flexible pavements.

The cost model for rehabilitation activities (excluding overlays and reconstruction) is also a function of a fixed unit cost and a variable unit cost. The overlay cost model is a function of the type of overlay. It is a sum of the site establishment cost, the surface preparation cost, the overlay placement cost, the steel reinforcement cost, and the cost of joints. The cost of bond breaker construction is calculated if a bond breaker is required to retard reflective cracking of overlays on PCC pavements.

In the case of an asphaltic concrete overlay on an existing rigid pavement, the cost of locating the existing transverse joints, saw cutting into the new overlay, and sealing is also considered. This is a standard practice in Pennsylvania. If the user does not wish to include this item in his overlay alternatives, he can simply ignore it by specifying a zero value for the unit cost of this item.

Two components are used in the cost model for reconstruction: *(a)* demolition cost and *(b)* reconstruction cost using the initial construction cost model. The cost model for peripheral items is a function of a single unit cost and includes maintenance and rehabilitation costs.

Maintenance and Protection of Traffic

Maintenance and protection of traffic during resurfacing, rehabilitation, and reconstruction projects are treated as a lump sum cost item, expressed as a percentage of the total cost for overlay, reconstruction, and concrete pavement restoration (CPR) work. For maintenance treatments, the fixed cost of each item should reflect the traffic-handling cost.

Traffic Delay Cost

Overlay placement, reconstruction, or CPR has a definite impact on traffic. The excess user costs associated with this impact are estimated using a traffic delay cost model. The model used was originally developed by Scrivner et al. *(12)* for overlay construction. Five types of traffic detour models are used in the model, which has been updated and modified by other investigators *(13)*. The model used in the LCC1 program *(8, 9)* handles the following activities: bituminous concrete or PCC overlays, CPR work, other rehabilitation activities (milling, leveling course, surface treatment, recycling), and reconstruction.

The model first predicts the delay time incurred by each vehicle as it passes through the restricted zone of work. This is calculated using the production rate and quantity of work associated with a specified rehabilitation action. Daily distributions of traffic for rural and urban areas are user inputs. Incremental user delay costs per unit time are built into the model. These, along with the user-specified traffic volumes and periods during which the delays will occur, are used to determine the traffic delay cost.

User Operating Cost

The option of determining the user operating cost associated with the performance history of pavements is also provided in the LCC methodology. Basically, the user operating cost model calculates operating costs due to a decrease in the present serviceability index (PSI). The consumption rate tables developed by Zaniewski et al. *(14)* are used for vehicle operating cost computations.

In the LCC methodology, an initial running speed of 55 mph is assumed for a pavement in ideal condition. The procedure for assigning various speed adjustment parameters for speed change and stop cycles is based on the FHWA's HPMS program *(15)*. Performance history of the pavement during its entire analysis period is estimated by calculating a PSI for each year. A linear relationship is used for pavement deterioration from an initial PSI (P1) to a terminal PSI (PT). The PSI at a given time is readjusted for overlay, reconstruction, CPR work (diamond grinding, slab jacking and subsealing, slab replacement, and spall repair), and recycling. PSI-values for future years are recalculated using linear deterioration rates based on the new PSI (after rehabilitation) and the expected lives of rehabilitation activities. An example of built-in expected lives for rehabilitation of portland cement concrete pavements is given in Table 3.

Salvage Value and Value of Extended Life

Salvage value is the residual value of the pavement or its reusable materials, or both, at the end of service life. Consideration should also be given to the value of extended life

TABLE 3 BUILT-IN DATA USED BY THE LCC1 PROGRAM TO PREDICT THE PSI HISTORY OF A GIVEN STRATEGY (portland cement concrete)

Alternative	Code	Expected Life (yr)[a] Low Traffic (ADT < 30,000)	High Traffic (ADT ≥ 30,000)	PSI
Construction/	1–10			
reconstruction	11–20			
< 8 in.		20	15	P1 (construction)
≥ 8 in.		20	20	PCON (reconstruction)
Thin overlay (< 2 in.)	21–40	7	4	POV[b]
Thick overlay (≥ 2 in.)	21–40	12	9	POV
Spall repair	45	9	5	0.8 (POV)
Subsealing	47	9	5	0.8 (POV)
Slab jacking	48	9	5	0.8 (POV)
Slab replacement	49	20	15	0.8 (POV)
Diamond grinding	50	9	6	0.8 (POV)
Recycling	51	15	15	0.9 (POV)

[a]Based mostly on the survey of PennDOT's engineers.
[b]POV = serviceability value after overlay or other types of rehabilitation.

related to the unequal serviceability levels of various alternatives at the end of the analysis period. As is shown in Figure 1, the two strategies result in different values of extended life. In the proposed LCC methodology, both the value of extended life (YVEXL) and the salvage value (TSALV) are considered. These are either negative costs or zero values.

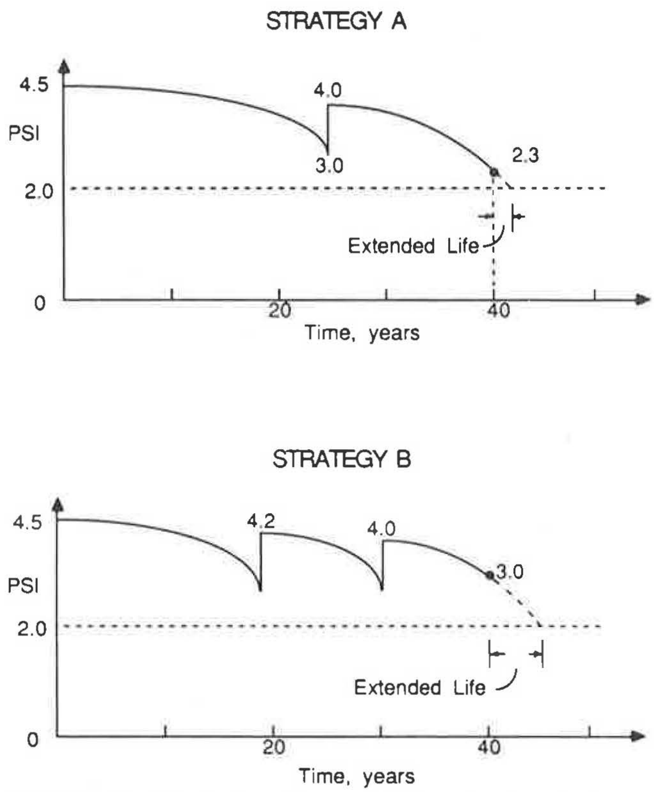

FIGURE 1 Illustration of expected lives for two strategies.

OPTIMIZATION

The LCC methodology uses a simple rank-ordering scheme to optimize the analyzed strategies. Several available options enable users to assess the impact of various economic considerations on rank ordering of the strategies. All options include future overlay costs. The program output will show the strategies ranked in order of ascending costs. These options are *(a)* total life-cycle costs, *(b)* initial construction costs (only for new pavements), *(c)* total life-cycle costs excluding maintenance costs, *(d)* total life-cycle costs excluding user costs, *(e)* total life-cycle costs excluding salvage value, *(f)* construction and rehabilitation costs, and *(g)* total of construction, maintenance, and rehabilitation costs.

PROCEDURES FOR ECONOMIC EVALUATION

Present worth (PW) analysis remains the best general method for LCC evaluations. The PW and equivalent uniform annual cost (EUAC) methods should not be regarded as separate or mutually exclusive options. Options for both methods are provided in the LCC methodology. The following discussion is largely drawn from Wilkes and Harrison *(16)*. Using the present worth method, the analysis period (also referred to as horizon length) should be the same for all alternative design strategies *(17)*. The present worth of costs for a given alternative can be calculated by discounting various cost streams (initial capital cost of construction, future rehabilitation and maintenance costs, user costs) by the chosen discount rate.

The EUAC method, known alternatively as the annual equivalent annuity (AE) method *(17)*, will give answers consistent with a benchmark PW when the decision-making environment is not complex. For example, EUAC can be used for a straight choice between alternatives in the absence of inflation and when anticipated inflation is uniform (meaning constant over time, across the alternative maintenance strategies, and for all sources of cost). EUAC is not precise under conditions of differential inflation (where the guaranteed consistency with PW is lost) but would be expected to give serviceable approximate results.

A discount rate, which is used to adjust future costs or benefits to present-day value, should not be confused with an interest rate, which is associated with the actual cost of borrowing money. In practice, the discount rate used will always contain judgmental elements. The validity of PW and AE is demonstrable when actual (nominal) cash flow and actual (nominal, not real) discount rates are used. Decisions based on real rates will be correct only if they are consistent with those obtained using nominal rates. If the real rate is defined appropriately, this consistency is guaranteed. For a given nominal rate of interest $100r\%$ the equivalent real rate is $100e\%$ given by

$$e = [(1 + r)/(1 + i)] - 1$$

where the appropriate rate of inflation is $100i\%$.

A real rate can be used only with cash flows expressed in base year prices (i.e., uninflated costs). Similarly, a nominal rate can only be used in conjunction with the actual cash flow expected.

The inflation rate to use is that which is relevant to the determination of actual cash flow (i.e., a rate that is drawn from changes in materials, equipment, labor, and other related costs and not a consumer price index). As with the discount rate, an element of judgment enters. An inflation rate using a published index could be worked out in the following manner. Highway maintenance and operation cost trends *(18)* indicate that

Year	*Index*
1982	160.04
1983	166.28

The current annual rate of inflation of these costs is

$$(166.28/160.04 - 1) * 100\% = 3.9\%$$

If present circumstances are thought likely to persist, the figure so obtained (rounded to 4 percent) can be used as the inflation figure. If federal policy is thought likely to reduce inflation significantly, judge a lower figure—say 3 percent. It should be noted that use of a lower figure in these circumstances would not represent financial imprudence. These figures are used to make a correct decision between alternatives. Use of an artificially high inflation figure may well lead to incorrect selection of maintenance strategy and consequently higher-than-necessary costs.

DEVELOPMENT OF THE MICROCOMPUTER PROGRAM

A microcomputer program, Life-Cycle Cost Analysis for Pavement Management, Version 1 (LCC1), was developed using the proposed LCC methodology for operation on an IBM-PC microcomputer using the MS-DOS operating system. The program requires a minimum of 200 k random-access memory (RAM). In addition, the following equipment is needed: at least one disk drive for double-sided, double-density floppy disks; monochrome video monitor; printer; and GWBASIC software. The execution of the LCC1 program, operating procedure, and input guide are treated in detail in the LCC1 user's manual *(9)*. Salient features of the LCC1 program are that it

- Generates an audio signal whenever the user makes an unacceptable entry.
- Features user-friendly data input sessions.
- Enables the user to update or modify data in a given category at any time during an input session.
- Has a built-in set of default input data.
- Checks the value of each input variable entered by the user against built-in maximum and minimum values.
- Allows the user to prepare an input data file in a given session by *(a)* creating an entirely new data file by entering all new data, *(b)* using the default data file with modifications to input default values in the desired categories, or *(c)* modifying any of the existing input files from the previous sessions without going through a complete input session.
- Allows the user to save changes made in the existing input data files for later use.
- Examines alternative strategies for new as well as existing pavements.
- Considers various types of pavements: *(a)* asphaltic concrete (bituminous) pavements, *(b)* continuously reinforced concrete pavements, *(c)* jointed reinforced concrete pavements, *(d)* jointed plain concrete pavements, and *(e)* bituminous pavements with rigid base.
- Considers multiple maintenance and rehabilitation activities.
- Calculates initial construction and future cost streams.
- Calculates user cost relative to traffic delays because of overlay construction.
- Includes user operating cost as an optional feature.
- Analyzes various economic scenarios using the PW or AE methods.
- Varies analysis periods.
- Performs full economic analysis at three different interest and inflation rates during a single session.
- Ranks up to a maximum of nine design strategies in order of ascending discounted life-cycle costs (PW or AE methods) using one of the seven available optimization options.

The LCC1 program consists of a series of program units and data files based on the capabilities of microcomputers. Figure 2 shows the job control system flow of the LCC1 program. The input data component is programmed in BASIC language. The analysis component has been programmed in FORTRAN 77. Communication between the BASIC and FORTRAN components is effected within the LCC1 program using user instructions in a friendly interactive mode.

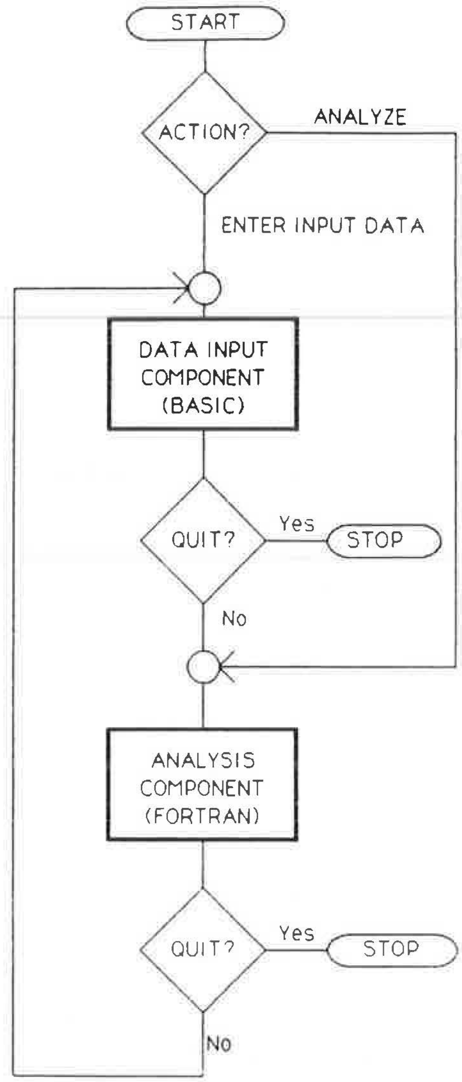

FIGURE 2 Job control system flow of the LCC1.

BASIC Component

The primary functions of the BASIC component are to generate the input data file, which is read and used by the FORTRAN component; to create an entirely new input data file or edit or modify an existing input data file; to load the default input data whenever the program finds that the specified file name does not exist; to edit and modify the default data file; to overwrite the modified values on the current input file (being edited) or create another file; to save the modified values or changes in the file specified by the user; to modify the value of a specific input variable in a current input file without going through the whole file; to create and modify several input data files in one session before starting the analysis part; and to check the input data in a current session for errors and unreasonable values wherever necessary.

The LCC1 program can store input data from every session in separate data files. The following menus are used for entering input data interactively.

Menu	*Description*
A	General design information,
B	Project data (including traffic data),
C	Roadway cost items,
D	Base and subbase placement costs,
E	Surface placement costs,
F	Maintenance unit costs,
G	Rehabilitation unit costs,
H	Peripheral maintenance and rehabilitation unit costs,
I	Traffic delay cost parameters,
J	User operating cost parameters,
K	Road structure information (associated with design strategies),
L	Overlay structure information (associated with design strategies),
M	Design strategies, and
N	Exit program.

The user is permitted to make changes in any of the menus described in the main menu without going through the entire session. Default data are provided for every variable in each of these menus. The default unit costs are based primarily on a review of tabulations of 1985 bids prepared by PennDOT.

To improve the efficiency of data entry for design strategies, it is recommended that the users enter up to 10 construction and reconstruction alternatives in Menu K (with information on layer types and thicknesses, shoulder type, width, thickness, etc. for each alternative). Similarly, up to 20 overlay alternatives can be entered in Menu L. Menu M is then used to enter up to 9 design strategies and associated analysis periods. The user can edit or review any of these design strategies before exiting from a given input data session.

FORTRAN Component

The FORTRAN component asks the user the name of the input data file for LCC analysis; reads the input data from the specified file; prints the input data; calculates life-cycle costs for every strategy; performs the economic analysis using the specified option (PW or AE methods, or both) and the first set of interest and inflation rates; ranks the strategies in order of ascending discounted life-cycle costs using one of the seven specified options; prints the results; calculates life-cycle costs; ranks and prints results for the second and third sets of interest and inflation rates; and analyzes additional input data files before exiting, if desired by the user. The FORTRAN component of the LCC1 program calls several different subprograms to perform the pertinent analyses, rank and print the input data, and produce the final output.

APPLICATION

The LCC1 program has been used for life-cycle analysis of rigid and flexible pavement designs. Examples of the LCC analysis of alternatives for the rehabilitation of existing rigid pavements are presented here. Table 4 gives several design

TABLE 4 STRATEGY FOR LIFE-CYCLE ANALYSIS OF CONCRETE PAVEMENT REHABILITATION

Time	Activity
Bituminous Overlay	
1 year	Bituminous overlay (minimum 3 ½ in.) saw and seal joints
	Adjust guide rail and drainage structures
	Type-7 paved shoulders
	Maintenance and protection of traffic
	User delay
5 years	Seal coat shoulders
	Clean and seal 25% of joints
10 years	1% full-depth patching
	60-psy scratch course
	1 ½-in. ID-2 overlay, saw and seal joints
	Type-7 paved shoulders
	Adjust guide rail and drainage structures, if necessary
	Maintenance and protection of traffic
	User delay
15 years	Seal coat shoulders
	Clean and seal 25% of joints
20 years	1 ½-in. cold milling (recycling)
	3% full-depth patching
	45-psy scratch course
	1 ½-in. ID-2 inlay
	Seal coat shoulders
	Maintenance and protection of traffic
	User delay
25 years	Seal coat shoulders
	Clean and seal 25% of joints
30 years	Same as 20 years, except patching same as 10 years
35 years	Seal coat shoulders
	Clean and seal 25% of joints

TABLE 4 *continued*

Time	Activity
Concrete Pavement Restoration	
1 year	CPR (using restoration techniques)
5 years	Seal coat shoulders
	Clean and seal 25% of joints
10 years	Concrete patching—25% of initial quantity
	Spall repair—25% of initial quantity
	Subsealing—25% of initial quantity
	Grinding—25% of initial quantity
	Clean and seal 25% of joints
	Seal coat shoulders, if Type 6 or 7
	Maintenance and protection of traffic
	User delay
15 years	Seal coat shoulders, if Type 6 or 7
	Clean and seal 25% of joints
20 years	1% full-depth patching
	Clean and seal 25% of joints
	60-psy scratch course
	3 1/2-in. ID-2 overlay, saw and seal joints
	Type-7 paved shoulders
	Adjust all guide rail and drainage structures
	Maintenance and protection of traffic
	User delay
25 years	Seal coat shoulders
	Clean and seal 25% of joints
30 years	3% full-depth patching
	60-psy scratch course
	1 1/2-in. ID-2 overlay, saw and seal joints
	Type-7 paved shoulders
	Adjust guide rail and drainage, if necessary
	Maintenance and protection of traffic
	User delay
35 years	Seal coat shoulders
	Clean and seal 25% of joints
Cement Concrete Overlay	
1 year	Plain cement concrete overlay (using the Corps of Engineers method for design)
	Patch existing pavement
	Spall repair and grinding
	Concrete shoulder
	Adjust guide rail and drainage structures
	Maintenance and protection of traffic
	User delay
5 years	Clean and seal 25% of longitudinal joints, including shoulders
	Reseal 5% of roadway transverse joints, 0% if neoprene seals are specified
10 years	Concrete patching—10% of quantity as determined in Year 0 (based on field measurement)
	Spall repair—10% of quantity as determined in Year 0 (based on field measurement)
	Diamond grinding—10% of quantity as determined in Year 0 (based on field measurement)
	Clean and reseal 25% of longitudinal joints, including shoulders
	Reseal 10% of roadway transverse joints, 0% if neoprene seals are specified
	Maintenance and protection of traffic
	User delay
15 years	Clean and seal 25% of longitudinal joints, including shoulders
	Reseal 10% of roadway transverse joints, 0% if neoprene seals are specified
20 years	CPR project, same as 10 years except double patching, grinding, and sealing quantities
	Reseal 5% of transverse joints if neoprene seals are specified
25 years	Clean and seal 25% of longitudinal joints, including shoulders
	Reseal 10% of roadway transverse joints, 5% if neoprene seals are specified
30 years	2% full-depth patching
	Clean and seal all joints
	60-psy scratch course
	3 1/2-in. ID-2 overlay, saw and seal joints
	Type-7 paved shoulders
	Adjust all guide rail and drainage structures
	Maintenance and protection of traffic
	User delay

TABLE 4 *continued*

Time	Activity
Cement Concrete Overlay	
35 years	Seal coat shoulders Clean and seal 25% of joints
Reconstruction	
1 year	Reconstruction after removal of existing concrete pavement Maintenance and protection of traffic User delay cost
5 years	Clean and seal 25% of longitudinal joints, including shoulders Reseal 10% of roadway transverse joints, 0% if neoprene seals are specified
10 years	Same as 5 years
15 years	Clean and seal 25% of longitudinal joints, including shoulders Reseal 10% of roadway transverse joints, 0% if neoprene seals are specified
20 years	Concrete patching—2% of area Spall repair—0.5% of area Subsealing—25% of the joints, minimum Diamond grinding—100% of roadway Clean and seal 25% of longitudinal joints, including shoulders Clean and reseal all transverse joints Reseal 5% of roadway transverse joints if neoprene seals are specified Maintenance and protection of traffic User delay
25 years	Clean and seal 25% of longitudinal joints, including shoulders Reseal 10% of roadway transverse joints, 5% if neoprene seals are specified
30 years	2% full-depth patching Clean and seal all joints 60-psy scratch course 3 ½-in. ID-2 overlay, saw and seal joints Type-7 paved shoulders Adjust all guide rail and drainage structures Maintenance and protection of traffic User delay
35 years	Seal coat shoulders Clean and seal 25% of joints

NOTE: For high-volume roadways, the resurfacing interval should be reduced to between 5 and 8 years.

strategies for a new highway facility. Some important design and economic variables used in this analysis follow. The present worth analysis (user operating cost not considered) optimization code is 1 (all costs).

Location = rural
Lanes = four
Project length = 1 mi
Design strategies = four
Discount rate = 10.0%
Inflation rate = 4.0%
Base year for costs = 1985
Base year for analysis = 1986
Initial ADT = 10,000
Initial ADT year = 1985
Design year ADT = 15,000
Design year = 2005
Percentage trucks = 5%

A summary of results is shown in Figure 3.

SUMMARY AND RECOMMENDATIONS

The LCC1 microcomputer program is designed for detailed economic evaluation of an array of feasible strategies for design and rehabilitation of pavements. The program consists of BASIC and FORTRAN components and provides a user-friendly and flexible input data entry and modification subsystem. The LCC1 program is designed for execution on an IBM-PC or any compatible microcomputer.

The LCC1 program offers the options of using the present worth or the annual equivalent annuity method for economic evaluation. The LCC1 analyses can be performed at three different sets of interest and inflation rates. The provision of a comprehensive default data file and the ability of the user to examine the default values of various input variables before entering new values are other features. The program can be used to examine the cost-effectiveness of restoration alternatives compared with resurfacing or reconstruction alternatives for concrete pavement rehabilitation.

ACKNOWLEDGMENT

The authors acknowledge the contributions of Len Moser and other individuals of Texas Research and Development Foundation. The authors also gratefully acknowledge useful discussions provided by John Hunt, Wade Gramling, and Barry Falkenstine of PennDOT during this research study.

LIFE CYCLE ANALYSIS FOR PAVEMENT MANAGEMENT DECISION MAKING

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION

LCCI
SUMMARY OF RESULTS
USING OPTIMIZATION CODE 1

TITLE: TEST FILE 1 (PCC P. REHAB) DATE: 05/20/86
LOCATION: PENNSYLVNIA(EXAMPLE) DISCOUNT RATE: 10.00
USER: WAHEED UDDIN INFLATION RATE: 4.00

RANK	1	2	3	4
STRAGEGY #	1	2	4	3
ANALYSIS PERIOD (YR)	40	40	40	40
PRESENT WORTH COSTS ($ ×1000)				
CONSTRUCTION	6.	4.	2.	123.
MAINTENANCE	87.	367.	63.	242.
REHABILITATION	383.	1319.	2113.	2250.
PERIPHERAL MAINT.	28.	22.	6.	34.
PERIPHERAL REHAB.	0.	0.	0.	0.
USER VEH. OPERATING	0.	0.	0.	0.
TRAFFIC DELAY	17.	54.	143.	46.
SALVAGE DUE EXTENDED LIFE	0.	0.	0.	−7.
PAVING MATERIAL SALVAGE	0.	−4.	−3.	−5.
TOTAL PRESENT WORTH	521.	1762.	2325.	2683.
ANNUAL EQUIV. ANNUITY ($)	34.	114.	150.	173.
RANK COSTS ($ ×1000)	521.	1762.	2325.	2683.

FIGURE 3 Summary of results generated by the LCC1 program.

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A Systems Approach for Design and Evaluation of Alternatives for Concrete Pavement Rehabilitation

B. FRANK MCCULLOUGH AND WAHEED UDDIN

To design and evaluate concrete pavement rehabilitation alternatives effectively it is necessary to consider the existing condition of pavements and to establish decision criteria for triggering the necessary pavement maintenance or rehabilitation actions. A methodology based on a systems approach to designing and evaluating cost-effective alternatives for concrete pavement rehabilitation is presented. The Pavement Rehabilitation Design System (PRDS-1) computer program incorporates this methodology. The program uses mechanistic analysis to generate numerous rehabilitation design alternatives and perform economic evaluation. The evaluation technique is sensitive to both the performance and the cost of the competing alternatives. Resurfacing alternatives include bituminous concrete, jointed portland cement concrete, and continuously reinforced concrete. The PRDS-1 program was used to evaluate typical concrete pavements for overlay thicknesses and associated life-cycle costs. A comparison of resurfacing alternatives with several restoration alternatives indicates that in certain cases restoration alternatives are more economical.

A large portion of the highway infrastructure of the United States is currently in place. Heavy traffic, environmental effects, and material properties and their interactions are primary causes of deterioration of the existing highway systems, the result of which is future need for significant pavement rehabilitation. Pavement rehabilitation is closely associated with pavement maintenance practices and costs. Rehabilitation alternatives for concrete pavements can be categorized as overlay of existing surface, restoration of the pavement structure, recycling, and reconstruction. To select cost-effective pavement rehabilitation strategies it is necessary to evaluate the existing condition of the pavement design and present and future overlay and maintenance requirements and make economic assessments of various design strategies.

In a recent study for the Pennsylvania Department of Transportation (PennDOT), the systems approach was used to develop a comprehensive computer program for the rehabilitation design of flexible and rigid pavements. The primary objectives were to identify and review alternate resurfacing strategies, examine restoration techniques, and develop a method of selecting cost-effective alternatives. The development and implementation of a computer program, Pavement Rehabilitation Design System, Version 1 (PRDS-1), for rehabilitation design of flexible and rigid pavements is described. An economic comparison of resurfacing and restoration alternatives is also included.

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REVIEW OF ALTERNATIVE REHABILITATION STRATEGIES

An extensive literature review to examine the state of the art in alternate resurfacing techniques and the use of new and nonconventional materials *(1)* showed that rehabilitation strategies for concrete pavements may be separated into four broad categories: overlay of existing surface, reconditioning and recycling of existing surface, restoration of the pavement structure, and reconstruction.

Constructing an overlay on the existing pavement is the traditional approach to rehabilitation. The addition of a new layer of material, depending on its thickness, can be considered a major structural improvement. Bond breakers, construction fabrics, and special overlay materials have been used to eliminate or reduce the severity of reflective cracking.

Reconditioning the existing surface involves reworking the pavement surface by cold milling, heater planer, or heater scarifier to remove surface irregularities for a smoother riding surface, to improve drainage, or to correct local skid resistance problems. Because of a combination of economic and material availability factors, a great deal of attention has recently been placed on recycling pavement materials. A large variety of recycling techniques has evolved *(2, 3)*.

Restoration of concrete pavements is another promising way to improve the structural condition and extend the life of a pavement. Pavement restoration is the application of a set of special procedures to repair all visible and known manifestations of distress. For the purpose of this paper, concrete pavement restoration (CPR) is defined as a rehabilitation methodology that does not include any overlay, resurfacing, or reconstruction. Examples of restoration techniques are joint sealing and repair, undersealing and slab jacking, diamond grinding, cold milling, concrete and asphalt patching, and concrete slab replacement. Typical kinds of rigid pavement distress that can be treated with concrete pavement restoration techniques are surface defects (scaling, raveling, polishing, depression, and punch-outs); cracking (longitudinal, transverse, and block); and joint-related distress (faulting, spalling, D-cracking, and blowups).

The fourth option to resurfacing is to remove the pavement structure and reconstruct. This alternative may be applicable to severely distressed pavement sections and used to correct drainage problems or to correct or change the pavement's geometrics or grade line. In reality, complete reconstruction of a pavement requires the design and construction of a new pavement structure and is therefore beyond the scope of this study.

PRDS METHODOLOGY

The systems approach to pavement rehabilitation design, developed at the University of Texas, formed the methodology for the evaluation and analysis procedure *(4–6)* developed in the present study *(1)*. The systems methodology provides a user with criteria for selecting an optimal pavement rehabilitation strategy. Recent research at the University of Texas has provided a computer model, the Rigid Pavement Rehabilitation Design System (RPRDS-1) *(6, 7)*. This model uses elastic layer theory to compute stresses and strains in the various pavement layers. These data are then used to evaluate expected pavement life. Life-cycle costs are computed and reduced to present worth for economic comparison of alternative strategies. In the present study, this program was extended to include flexible pavement analysis. The revised program is titled the Pavement Rehabilitation Design System (PRDS-1).

Extensive test runs were made on the PRDS-1 program to verify the analysis of concrete pavements. The sensitivity of the design selection to traffic delay cost variables and values of Young's modulus was studied to verify the reasonableness of the model. The results of these runs were used for implementation.

Figure 1 shows the general framework of the PRDS-1 program. The key elements of the PRDS-1 program are data input (structural variables, cost parameters, and constraints); structural analysis (define critical stress or strain location and compute stresses and strains with elastic layer theory); strategy selection (adequate structural life and write file of feasible strategies); present worth analysis for all feasible strategies (initial cost, maintenance cost, user delay costs, and salvage value); sorting strategies with the lowest present worth; and printing outputs.

Figure 2 shows the types of overlay alternatives considered by the PRDS-1 program. Note that as many as two overlays may be considered during the analysis period. The surfacing used in these overlays may be asphaltic concrete pavement (ACP), continuously reinforced concrete pavement (CRCP), or jointed concrete pavement (JCP).

Prediction of Pavement Response

The PRDS-1 program can analyze a multitude of possible overlay strategies, which requires an even larger number of pavement responses for predicting overlay life. Because it is not feasible to calculate and recalculate responses for every option, a computer subroutine, CONRSP, is used in the PRDS-1 program to make appropriate decisions about what responses are required for each strategy, prepare the necessary inputs to calculate each response, call appropriate routines to calculate responses, and store responses for later use.

Linear elastic layered theory *(1)* is used for computing stresses, strains, and displacements in flexible pavements. Layered theory cannot predict pavement responses at discontinuities such as the pavement edge or joint in a rigid pavement. Therefore, a stress adjustment factor is input to the PRDS-1 to modify interior stresses estimated with linear elastic theory to

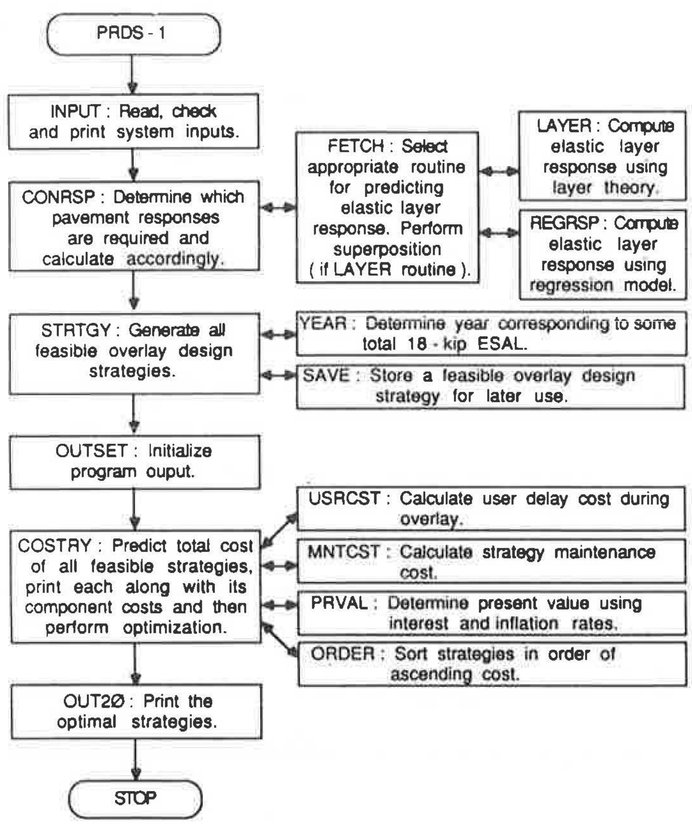

FIGURE 1 General flow diagram of the Pavement Rehabilitation Design System computer program *(1)*.

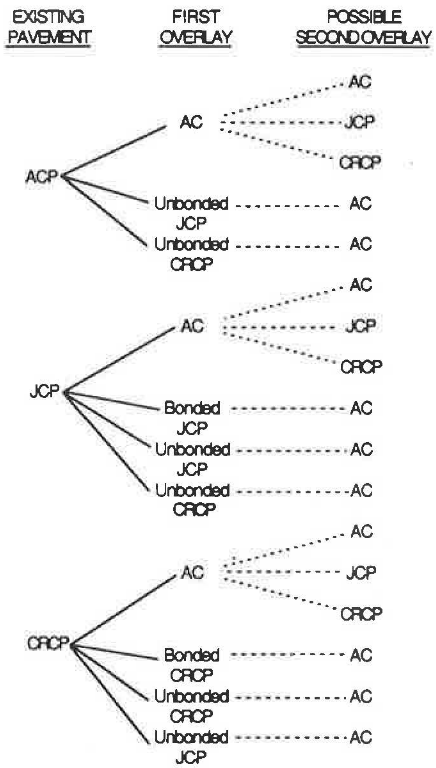

FIGURE 2 Overlay design strategies available in PRDS-1 with regard to existing types of pavement and overlay.

represent the critical stress near an edge or corner of rigid pavements. The assumed loading and pavement configurations for calculating these stresses are shown in Figure 3.

To develop these stress factors, solutions based on the finite element method were used to adjust interior stresses to critical stresses for each type of rigid pavement–overlay combination permitted by the PRDS-1 program. A list of the critical stress factors is included in the PRDS-1 user's manual *(8)*. The SAP-2 finite element computer program *(9)* was used to develop these factors using the following procedure:

- Interior responses were determined using SAP-2,
- Critical responses near an edge or corner were determined with SAP-2, and
- The critical stress adjustment factor was computed as the ratio between the critical stress and the interior stress.

Prediction of Fatigue Life

Fatigue equations for load-associated cracking are used in the PRDS-1 program to determine the number of equivalent single axle load (ESAL) applications that a pavement structure can carry before it reaches some limiting failure criterion. This is accomplished by relating the critical stress in a rigid pavement, or critical strain in a flexible pavement, to the number of 18-kip ESAL applications needed to reach a specified level of cracking. Structural capacity of pavement is calculated in terms of remaining life, which requires the assumption that Miner's damage hypothesis is valid for highway pavements.

PCC Fatigue Equations

Two fatigue equations (Figure 4) are used in the PRDS-1 program. These equations were developed from an analysis of AASHO Road Test data and data from condition surveys of rigid pavements in Texas *(10–12)*. These equations are an improvement over the original portland cement concrete (PCC) fatigue equation developed for the Federal Highway Administration by ARE, Inc. *(13)*.

The equation for PCC surface layers uses a failure criterion of a cracking index of 50 ft per 1,000 ft^2. In its development seasonal variation in subgrade strength and a simulation of actual axle loadings used at the AASHO Road Test were considered. For overlaid PCC pavements, the second PCC equation is used. This equation is an adjustment of the first equation based on results that showed that overlaid pavements in Texas were far exceeding their predicted fatigue life *(14)*. The adjusted equation is used in the PRDS-1 program for predicting the life of overlaid rigid pavements with PCC shoulders.

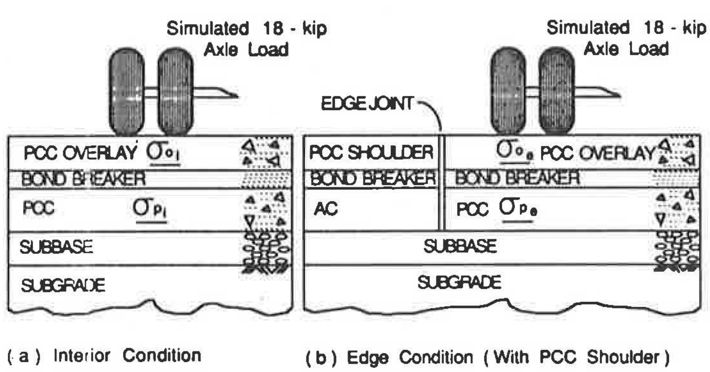

FIGURE 3 Interior and edge stress conditions for one of the overlay strategies considered in the PRDS-1 (an existing PCC pavement with an unbounded PCC overlay with PCC shoulders); the ratio of edge to interior stress is used for adjusting stress predicted by elastic layer theory *(7)*.

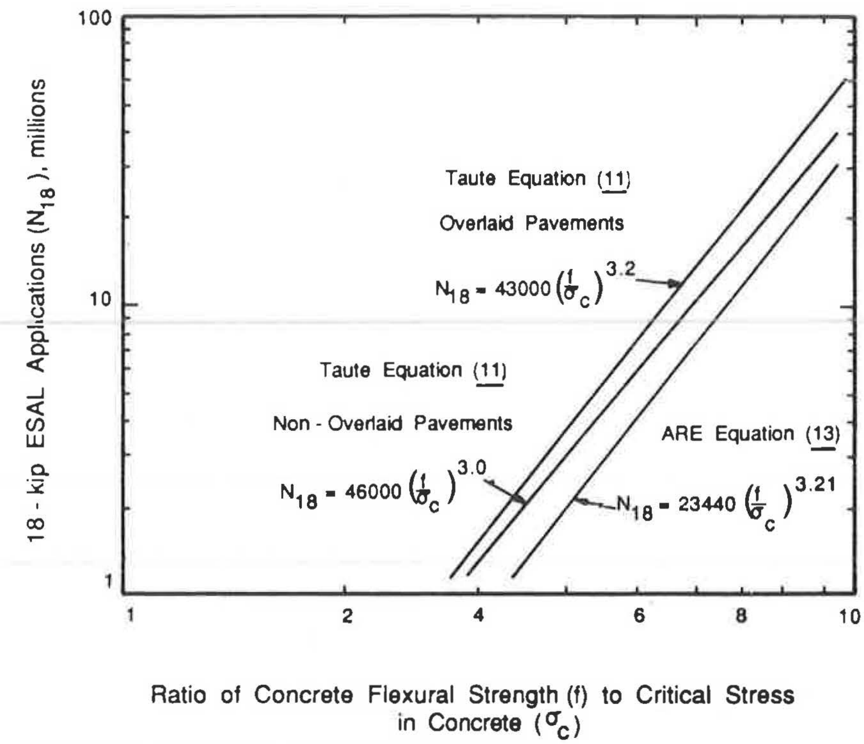

FIGURE 4 Comparison of PCC fatigue equations.

Asphalt Concrete Fatigue Equation

The equation used in the PRDS-1 program to predict fatigue life was developed from laboratory fatigue tests and adjusted with data obtained from the AASHO Road Tests for asphalt concrete pavements with less than 10 percent cracking in the wheelpath area *(15, 16)*. This fatigue equation is expressed as

$$\log N_f = 15.947 - 3.291 \log (e/10^{-6}) - 0.854 \log (E^*/10^3)$$

where

N_f = number of 18-kip ESAL applications to "failure,"
e = maximum asphalt concrete tensile strain, and
E^* = asphalt concrete complex modulus (psi).

Year Corresponding to Overlay Placement

Overlay placement times are specified in the PRDS-1 program in terms of percentage remaining life of the pavement structure. To compute the time required to reach a specific level of remaining life, the program first computes the allowable number of applications to failure using the appropriate fatigue equation. The amount of traffic carried by a pavement to a level of remaining life is computed by multiplying the allowable number of applications by the remaining life value. The time required to apply this amount of traffic is computed using the traffic growth model and the initial traffic level and growth rate specified.

Design Constraints

The PRDS-1 program limits the number of thickness design alternatives by omitting those that provide excessive lifetimes. When a feasible thickness design has been generated, no further designs in which the overlay thickness is greater are considered. The primary design constraints used in the PRDS-1 to determine feasible overlay strategies are length of analysis period, minimum time between overlays, maximum period of heavy maintenance, and maximum total overlay thickness.

Cost Models

The PRDS-1 program includes the following models for cost analysis of feasible strategies: traffic delay cost, overlay construction cost, distress and maintenance cost, value of extended life, and salvage value. All costs are discounted to present worth to allow for economic comparison of alternatives.

Traffic Delay Cost

The traffic delay model computes user delay time and traffic delay costs associated with traffic passing through an overlay zone. The model used in the PRDS-1 program was originally developed by Scrivner et al. *(17)*. The model first predicts the delay times incurred by each vehicle as it passes through the restricted overlay zone. Daily distributions of traffic for rural and urban areas as well as the incremental user delay costs per unit time are built into the routine. These, along with the user-specified traffic volumes and periods during which the delays will occur, are used to determine the total overall traffic delay cost. The cost is then converted to a per-square-yard basis and discounted to present worth.

Overlay Construction Cost

The components of the overlay construction cost model are site establishment cost, surface preparation cost, overlay and shoulder placement cost, and steel reinforcement cost. Each of these cost components is determined on a per-square-yard basis and summed to get total overlay construction costs.

Distress and Maintenance Cost Model

The PRDS-1 program predicts distress using rates of distress development specified by the user. The life of a pavement overlay strategy is divided into several periods as shown in Figure 5. The first is the zero distress rate period between 0 and 20 percent of the overlay life. Typically, few severe distress manifestations will occur during this period. The second, between 20 and 60 percent of the overlay life, is the initial distress rate period during which distress develops at a low but significant rate. The third period, between 60 and 100 percent of the overlay life, represents a secondary rate of distress development, which will occur up to the end of the overlay fatigue life. From this point to the maximum allowable number of years of heavy maintenance, the periods consist of 1-year intervals during which distress development should increase geometrically. The user, then, must define values for the initial distress rate, the secondary distress rate, and the distress rate for each year up to the maximum allowable number of years of heavy maintenance. The user also specifies the cost of repairing a typical manifestation of distress or defect.

With this information, the program increments its way through the life of a strategy, multiplying the number of defects that occurs during a given year by the cost of repairing them. These costs are then discounted to present worth. When the end of the analysis period is reached, these yearly costs are accumulated to give the total maintenance cost of the strategy.

Value of Extended Life

This quantity represents a future return cost that results from the additional years of service past the end of the analysis period that some strategies will provide because they are "overdesigns." Provision is made in the PRDS-1 for the user to assign some value on a per-square-yard basis to each additional year of service.

Salvage Value

Salvage value accounts for the value of the overlay structure after it reaches the end of its life. This does not necessarily

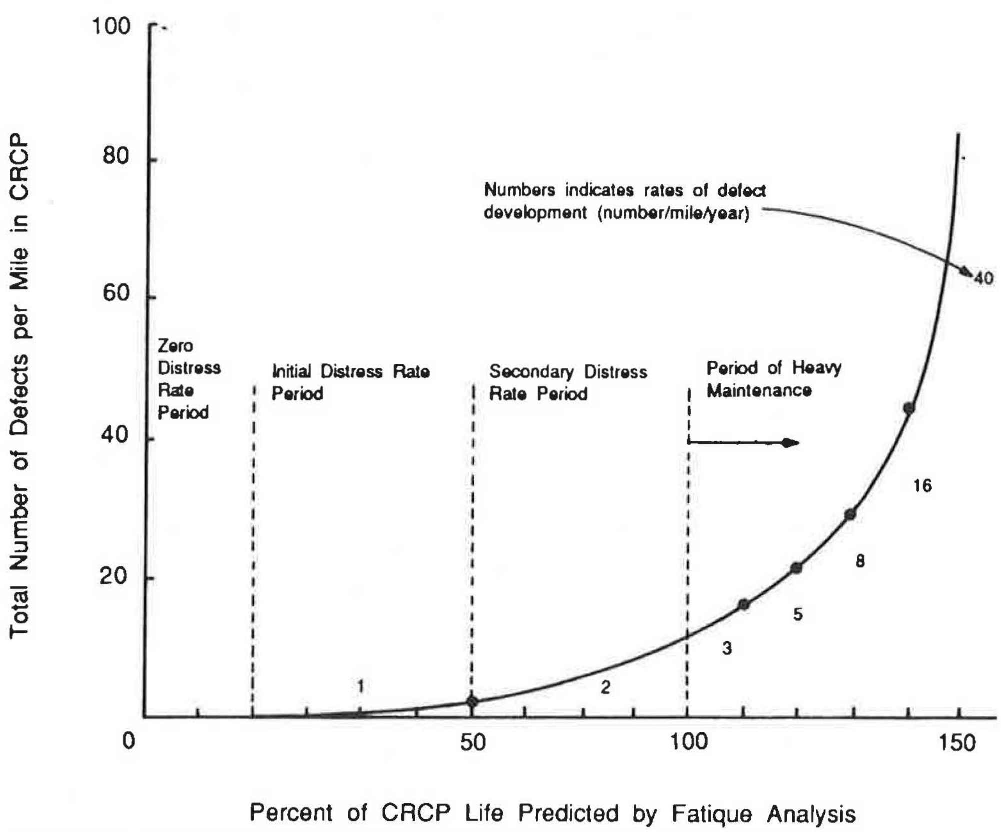

FIGURE 5 Normalized graph of the development of CRCP defects in Texas *(6)*.

correspond to the end of the analysis period because some strategies may last significantly longer. Salvage value is computed by multiplying the total cost of overlay construction by a percentage specified by the user and discounting to present worth. Only the value of overlay construction cost is considered. Original pavement construction cost need not be considered because it is the same for all strategies.

Economic Analysis and Ranking

The net present value model is used to account for the time value of money. The discount rate is used to calculate present worth of future costs. This rate is somewhat less than the prime interest rate offered for investments and represents the "true" growth in the value of money. The PRDS-1 program sorts the top 20 strategies in order of increasing present worth of total cost.

IMPLEMENTATION OF THE PRDS-1 PROGRAM

Input values were selected for Pennsylvania conditions and alternative strategies were analyzed for several cases of flexible and rigid pavements. Details of various sets of runs of the PRDS-1 program are presented in project reports *(1, 9)*. These analyses provided valuable information on the behavior of the model for a variety of conditions. This information was used to make a detailed study of alternative overlay strategies for four pavement types: continuously reinforced concrete pavements (CRCP), jointed concrete pavements (JCP), flexible pavement with an asphalt-stabilized base, and flexible pavement with a granular base.

A four-lane highway was considered for these examples. The results of CRCP and JCP examples are presented in this paper. A factorial design was used to investigate the influence of traffic volume and subgrade types. The user of this program should take care in the assignment of 18-kip ESAL applications. The AASHTO traffic equivalency factors for tandem axles can be appreciably higher on rigid pavements than are those for axles of the same weight on flexible pavements. Subgrade modulus was analyzed at three levels: low (8,000 psi), medium (14,000 psi), and high (20,000 psi).

CRCP Analysis

A summary of results for the subgrades of 14,000 psi is given in Table 1. At low traffic levels, AC overlays are generally the most cost-effective, followed by PCC overlays with flexible shoulders. There is a significant cost difference factor of approximately 2.5 between AC and PCC overlay strategies at low traffic levels. The cost is generally lower when the first overlay is placed at 0 percent remaining life.

At medium traffic levels, AC overlays are still generally the least expensive. The difference between AC and PCC overlays is still significant but to a lesser degree than was the case for low traffic levels. A cost ratio of 1.78 was obtained at the low subgrade level. The timing of the first overlay is not particularly significant. If overlay placement is postponed because of the effect of the discount rate on the cost of additional overlay thickness, AC overlay thickness is sensitive to both the timing of the first overlay and the subgrade conditions; PCC overlay thickness is not sensitive to these two variables.

User delay cost is not very significant at medium traffic levels. At high traffic levels, user delay cost overrides all other cost considerations. Hence, the optimal strategy is to build an overlay to last the entire period at the start of the analysis period.

JCP Analysis

At all traffic levels, AC overlays are generally significantly cheaper than PCC overlays with AC-to-PCC cost ratios between 2 and 4. At low traffic levels, the influence of both subgrade condition and time of first overlay placement is not significant for AC overlay thickness. The timing of the first overlay, however, has a significant effect on the total rehabilitation cost. An increase of 79 percent in total overlay cost can result if the overlay is placed at 20 percent remaining life instead of 0 percent because costs are discounted over a longer period. Delay costs are not significant at low traffic levels.

An example of overlay thickness versus remaining life for a low subgrade modulus is shown in Figure 6. At medium traffic levels, both subgrade condition and timing of first overlay placement have some influence on the selected AC overlay thickness. Postponing the overlay to remaining life values lower than the initial condition can result in an increase in total overlay cost of up to 23 percent.

Delay costs are more significant when the interaction between subgrade and overlay timing results in increased overlay thickness and can be up to 17 percent of the total overlay cost. At 0 percent initial remaining life, subgrade condition does not have a significant effect on overlay thickness because of the required increase in thickness. The poor condition of the existing pavement causes the user delay costs to be fairly significant. At high traffic levels it is generally cheaper to place the first overlay as soon as possible; increased overlay thickness and even higher traffic levels result in higher total overlay cost if the overlay is delayed.

Cost-Effective Overlay Strategies

The cost-effective overlay strategies for Pennsylvania conditions are given in Table 2 for continuously reinforced concrete pavement and in Table 3 for jointed concrete pavement. These recommendations are based on the conditions and assumptions used in the analysis. Strategies for specific projects should be based on an analysis of existing conditions for the project.

CONCRETE PAVEMENT RESTORATION AS A REHABILITATION STRATEGY

Concrete pavement restoration (CPR) alternatives to resurfacing can be based on economic or policy considerations. The PRDS-1 program cannot directly consider life-cycle costs for restoration without an overlay. Criteria for considering CPR as

TABLE 1 FINAL OVERLAY ANALYSIS OF CRCP WITH 40 PERCENT REMAINING LIFE

Traffic Level (ADT)	Type of Overlay	Value	Remaining Life at Placement			
			40%	20%	10%	0%
10,000	AC	Total cost ($)	7.94	6.33	5.86	5.43
		Delay cost ($)	0.11	0.09	0.09	0.08
		Thickness (in.)	2	2	2	2
		Placement (yr)	0	7.4	10.6	13.6
	PCC with flexible shoulders	Total cost ($)	22.54	16.16	14.65	13.02
		Delay cost ($)	1.38	1.18	1.08	1.00
		Thickness (in.)	6	6	6	6
		Placement (yr)	0	7.4	10.6	13.6
	PCC with rigid shoulders	Total cost ($)	29.09	21.10	18.46	16.28
		Delay cost ($)	1.38	1.18	1.08	1.00
		Thickness (in.)	6	6	6	6
		Placement (yr)	0	7.4	10.6	13.6
30,000	AC	Total cost ($)	10.64	10.36	10.64	10.87
		Delay cost ($)	0.49	0.62	0.82	1.12
		Thickness (in.)	3	4	5	5
		Placement (yr)	0	2.6	3.8	5
	PCC with flexible shoulder	Total cost ($)	25.45	22.96	21.91	21.33
		Delay cost ($)	4.14	3.93	3.91	4.27
		Thickness (in.)	6	6	6	6
		Placement (yr)	0	2.6	3.8	5
	PCC with rigid shoulder	Total cost ($)	31.85	28.62	27.25	26.37
		Delay cost ($)	4.14	3.93	3.91	4.27
		Thickness (in.)	6	6	6	6
		Placement (yr)	0	2.6	3.8	5
60,000	AC	Total cost ($)	152.06	203.52	220.67	207.95
		Delay cost ($)	138.78	190.76	207.32	258.00
		Thickness (in.)	6	7	7	8
		Placement (yr)	0	1.3	2	2.6
	PCC with flexible shoulder	Total cost ($)	189.48	216.87	232.86	250.77
		Delay ost ($)	167.90	196.60	213.08	230.77
		Thickness (in.)	6	6	6	6
		Placement (yr)	0	1.3	2	2.6
	PCC with rigid shoulder	Total cost ($)	195.84	220.84	238.61	256.33
		Delay cost ($)	167.90	196.60	213.08	230.77
		Thickness (in.)	6	6	6	6
		Placement (yr)	0	1.3	2	2.6

NOTE: Subgrade modulus of elasticity = 14,000 psi. AC = asphalt concrete and PCC = portland cement concrete.

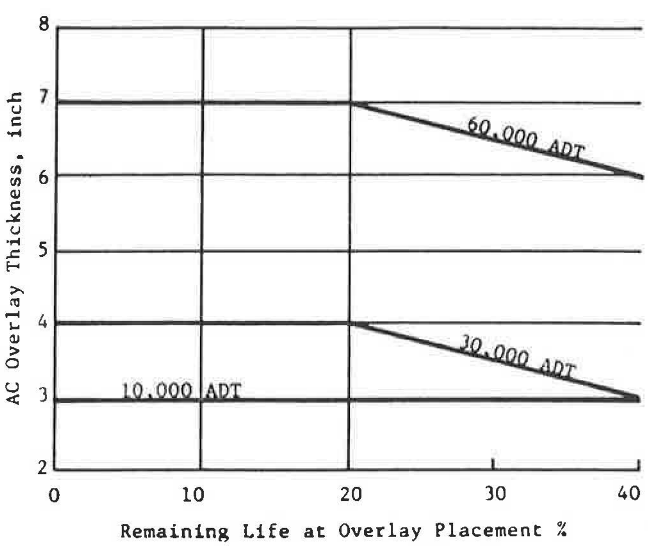

FIGURE 6 AC overlay thickness on JCP with subgrade modulus of elasticity of 8,000 psi.

a feasible rehabilitation strategy should be based on functional performance developed from an objective evaluation of condition survey information and other test results. An objective criterion for the application of CPR strategies can be based on a comparison of life-cycle costs. An example of life-cycle costs for CPR techniques is presented for a JCP using the conditions for the final analysis for JCP described earlier. The assumed unit costs for the CPR techniques are $0.50 per linear foot for joint reseal, $55.00 per square yard for full-depth slab placement, $1.80 per square yard for undersealing, and $3.10 per square yard for diamond grinding.

The cost of resurfacing alternatives predicted by the PRDS-1 program for a subgrade modulus of elasticity of 8,000 psi with a traffic level of 10,000 ADT is given in Table 4. The cost varied from $10.74 per square yard for immediate placement to $5.30 per square yard for a 19-year delay in overlay placement.

To make life-cycle computations for CPR strategies, the conditions described in Table 4 and the following assumptions were used: *(a)* the restoration work restores the pavement to an

TABLE 2 RECOMMENDATIONS FOR ECONOMICAL OVERLAY STRATEGIES OF CRCP BASED ON PRDS-1 ANALYSIS

Subgrade Condition (elastic modulus)	Traffic Level		
	Low	Medium	High
Low (8,000 psi)	AC overlay, intermediate placement delay	AC overlay, immediate placement	PCC overlay or AC overlay[a], immediate placement
Medium (14,000 psi)	AC overlay, long placement delay	AC overlay, moderate placement delay	AC overlay or PCC overlay, immediate placement
High (20,000 psi)	AC overlay, long placement delay	AC overlay, moderate placement delay	AC overlay, immediate placement

NOTE: Moderate placement delay—optimum strategy calls for routine maintenance for 5 years then placement of overlay; intermediate placement delay—optimum strategy calls for routine maintenance for 10 years then placement of overlay; long placement delay—optimum strategy calls for routine maintenance for 15 years then placement of overlay. Analysis included user delay costs. Initial remaining fatigue life of existing pavement was 40 percent.
[a]For high traffic levels AC and PCC overlay costs are very close. The overlay listed first had the lowest predicted present cost.

TABLE 3 RECOMMENDATION FOR MOST ECONOMICAL OVERLAY STRATEGIES FOR JOINTED CONCRETE PAVEMENTS BASED ON PRDS-1 ANALYSIS

Subgrade Condition (elastic modulus)	Traffic Level		
	Low	Medium	High
Low (8,000 psi)	AC overlay, long placement delay	AC overlay, moderate placement delay	AC overlay, immediate placement
Medium (14,000 psi)	AC overlay, long placement delay	AC overlay, intermediate placement delay	AC overlay, immediate placement
High (20,000 psi)	AC overlay, long placement delay	AC overlay, intermediate placement delay	AC overlay, immediate placement

NOTE: Moderate placement delay—optimum strategy calls for routine maintenance for 5 years then placement of overlay; intermediate placement delay—optimum strategy calls for routine maintenance for 10 years then placement of overlay; long placement delay—optimum strategy calls for routine maintenance for 20 to 30 years then placement of overlay. Initial remaining fatigue life of existing pavement was 40 percent. Analysis included user delay costs.

acceptable level of service for the duration of the analysis period, *(b)* delay costs computed for application of an immediate overlay by PRDS-1 are applicable for the restoration work, *(c)* all restoration work is performed immediately, and *(d)* user delay cost equals $0.16 per square yard. The following level of restoration work is assumed.

- There are three conditions for full-depth slab replacement: Case A, replace 2 slabs; Case B, replace 20 slabs; and Case C, replace 30 slabs.
- Joint resealing is required on 80 of the 88 joints.
- Undersealing is required for an area of 600 yd^2.
- Diamond grinding is needed to remove faulting on 50 percent of the joints. Assuming grinding on one side of joint over an area 4 ft × 24 ft, grinding area = 44 (4 ft × 24 ft)/9 ft^2 = 469 yd^2.

The computation of restoration cost is as follows:

Surface area of section = 1 mi (5,280 ft) × 24 ft/9 = 14,080 yd^2,
Joint resealing cost = 80 joints × 24 ft/joint × $0.5/ft = $960,
Undersealing cost = 600 yd^2 × $1.8/$yd^2$ = $1,080,
Grinding cost = 469 yd^2 × $3.1/$yd^2$ = $1,454, and
Cost for full-depth slab replacement for one slab = [(60 ft × 12 ft)/9] × $55 yd^2 = $4,400/slab.

In Case A (replacement of 2 slabs),

TABLE 4 SUMMARY DATA FOR LIFE-CYCLE COST EXAMPLE

Details of Overlay Strategy	Remaining Life at Overlay Placement			
	40%	20%	10%	0%
Total cost per square yard	$10.74	$ 7.10	$ 6.08	$ 5.30
User delay cost ($/$yd^2$)	0.16	0.13	0.11	0.10
Overlay thickness (in.)	3	3	3	3
Years until placement	0	10.4	14.7	18.7

NOTE: AC overlay; ADT = 10,000; E_{sub} = 8,000 psi; remaining life of pavement = 40%. Surface type = 10-in. JCP without concrete shoulder, number of lanes = 2 (one direction), lane width = 12 ft, project length = 1 mi, number of existing defects = 2 at 40% remaining life, and joint spacing = 60 ft.

$$\text{Total cost} = \text{restoration cost} + \text{traffic delay cost},$$
$$\text{Restoration cost} = [\$960 + \$1{,}080 + \$1{,}454 + 2(\$4{,}400)]/14{,}080 \text{ yd}^2 = \$0.87/\text{yd}^2, \text{ and}$$
$$\text{Total cost} = \$0.87/\text{yd}^2 + \$0.16/\text{yd}^2 = \$1.03/\text{yd}^2.$$

In Case B (replacement of 20 slabs),

$$\text{Restoration cost} = [\$3{,}494 + 20(\$4{,}400)]/14{,}080 \text{ yd}^2 = \$6.50/\text{yd}^2, \text{ and}$$
$$\text{Total cost} = \$6.50/\text{yd}^2 + \$0.16/\text{yd}^2 = \$6.66/\text{yd}^2.$$

In Case C (replacement of 30 slabs),

$$\text{Restoration cost} = [\$3{,}494 + 30(\$4{,}400)]/14{,}080 \text{ yd}^2 = \$9.62/\text{yd}^2 \text{ and}$$
$$\text{Total cost} = \$9.62/\text{yd}^2 + \$0.16/\text{yd}^2 = \$9.78/\text{yd}^2.$$

These assumptions and calculations are made only for illustration. The ranking of the resurfacing and restoration options for this example by cost is given in the following table.

Rank	*Cost ($/yd²)*	*Strategy*
1	1.03	Restoration Case A
2	5.30	3-in. AC overlay placed at 19 years
3	6.08	3-in. AC overlay placed at 14 years
4	6.66	Restoration Case B
5	7.10	3-in. AC overlay placed at 10 years
6	9.78	Restoration Case C
7	10.74	3-in. AC overlay placed immediately

Thus, depending on the level of restoration required for this pavement, restoration can be an economical alternative to resurfacing. For this analysis, only one period of restoration was performed at the start of the analysis period. Field conditions may necessitate future restoration and repair work that is not accounted for in this analysis.

SUMMARY AND RECOMMENDATIONS

The development and implementation of the PRDS-1 computer program that uses a systems approach to economic and structural evaluation of rehabilitation alternatives have been described. This computer model was exercised for rigid and flexible pavements using inputs representative of Pennsylvania conditions. Recommendations on economical resurfacing strategies for flexible pavements based on this analysis were presented. Costs for feasible overlay strategies were compared with the cost of restoration techniques.

PRDS-1 considers a wide range of cost variables in the economic analysis. Undoubtedly there are other cost variables that could be considered in the life-cycle analysis. Fatigue is used as the only limiting criterion in the program. The PRDS-1 program is written in a modular structure such that it may be readily modified when performance models for other types of pavement distress become available.

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