

TRANSPORTATION  
RESEARCH RECORD 846

# Pavement Management



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*TRANSPORTATION RESEARCH RECORD* 846

# Pavement Management

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# Effect of Pavement Roughness on Vehicle Fuel Consumption

FREDERIC R. ROSS

Several published research reports have shown that vehicle fuel consumption increases as pavement roughness increases. The existence of such a relation is today of particular interest to state departments of transportation for use in cost-benefit analysis of potential highway improvement projects. For a variety of reasons, however, the results of the earlier studies are not readily usable in benefit calculations. Therefore, in 1980 the Wisconsin Department of Transportation conducted a local field study that sought to define, for practical application, the relation between automobile fuel consumption and pavement roughness. Three different automobiles traveled on five pavement sections that, collectively, represented a wide range of roughness. For these pavements, roughness was expressed in terms of serviceability index, as measured by Wisconsin's electronic road meter. Fuel consumption was measured by a specially designed fuel meter mounted in each test vehicle. To minimize the influence of variables other than pavement roughness on fuel consumption, test conditions and procedures were highly controlled. The collected data suggest that there is in fact a quantifiable, but very modest, increase in automobile fuel consumption as pavement roughness increases. Over the range of roughness typically associated with state trunk highways (serviceability values between 1.5 and 4.5), this increase was (a) about 1.5 percent, (b) linear, and (c) not related to vehicle size. For the entire range of roughness included in this study (serviceability values between 0.9 and 4.4), the increase in fuel consumption was about 3.0 percent, which suggests that the overall relation between fuel consumption and pavement roughness may be nonlinear.

Several research investigations in the past decade have indicated an apparent relation between vehicle fuel consumption and the roughness of the traveled pavement surface (1-3). Specifically, these investigations have shown that fuel consumption increases as pavement roughness increases. The existence of such a relation has aroused considerable national attention because of the increasing costs of motor fuel and the interest in conserving energy.

Two of these investigations, because of their thoroughness and their potential usefulness, are especially important. One was conducted in 1971 by Claffey under the auspices of the American Association of State Highway and Transportation Officials (AASHTO) and the Federal Highway Administration (FHWA) (1); the other was conducted in the late 1970s in Brazil by the Texas Research and Development Foundation (2). Unfortunately, there is substantial disagreement in the findings of these two studies. Using normalized data from the two sources, Zaniewski and others (4) have reported an indicated increase in fuel consumption between a broken asphalt pavement and a smooth surface of 30 percent for Claffey's results compared with only 10 percent for the Brazil results. Obviously, a difference of this magnitude creates serious problems for an agency interested in practical application of the findings.

Despite such disagreement, Claffey's results have formed the basis for a number of publications that seek to dramatize the effect of deteriorated pavements on fuel consumption. As an example, it has been reported that driving a car at 40 mph on a deteriorated paved surface can increase fuel consumption by 56 percent compared with driving on a pavement in good condition (5). Conceivably, this figure represents an extreme situation--some "worst possible" case. But there is good reason to believe it considerably overstates the effect of pavement roughness on fuel consumption for the expected range of roughness on state trunk pavements. Obviously, an abandoned, disintegrated pavement would be inappropriate for practical fuel consumption studies.

There are other difficulties in applying Claffey's results. Only two levels of roughness were included in his study, and these were not defined by objective criteria that would allow systemwide extension of the findings. Finally, it is not certain that the automobile Claffey used--a 1964 sedan--responded to pavement roughness in the same way as do today's smaller automobiles.

Because of the apparent limitations in earlier investigations, the Wisconsin Department of Transportation (DOT) in 1980 contracted with Claffey to act as a consultant for an updated study that sought practical definition of the relation between fuel consumption and pavement roughness. This paper reports the results of that study.

## FIELD STUDY

### Test Pavements

Test pavements for the study had to be straight, level, lightly traveled, uniformly rough or smooth, all of one pavement type (in this case, bituminous concrete) and a minimum of 5000 ft in length (including a 4000-ft test segment and 500-ft approach segments at either end). Following extensive searching, five 2-lane bituminous concrete pavements were located that satisfied these requirements and, in addition, had suitably varied levels of roughness, as given below:

| Test Site | Serviceability Index     |                          |
|-----------|--------------------------|--------------------------|
|           | Northbound/<br>Eastbound | Southbound/<br>Westbound |
| 1         | 4.4                      | 4.4                      |
| 2         | 3.6                      | 3.5                      |
| 3         | 2.1                      | 2.2                      |
| 4         | 1.9                      | 1.9                      |
| 5         | 0.9                      | -                        |

Rod and level measurements, taken at 200-ft intervals along the pavement surfaces, indicated that the selected sites were acceptably level. The maximum variation in any 200-ft interval at any site was slightly more than 1 ft, and the maximum variation over the 4000-ft test segment was 8 ft.

Pavement roughness was measured in terms of serviceability index (SI). Although there are more sophisticated measures of pavement profile, the advantage of SI is its universality. Furthermore, for a state such as Wisconsin, where periodic SI measurements are obtained on the entire system of state trunk highways, the existence of a relation between fuel consumption and SI would enable systemwide application of the results. For all five sites included in this study, serviceability measurements were made in each lane with Wisconsin's electronic road meter prior to fuel consumption testing. As the table above indicates, directional SI differences were minimal. As a check on these initial readings, repeat serviceability measurements were made midway through the study. In both instances, the assigned SI values were based on multiple readings. Values determined on the two dates were essentially the same for all sites.

### Equipment

Three different sizes of automobiles were equipped with manual fuel meters constructed from schematic drawings prepared by Claffey. Details of the test vehicles are given in Table 1.

As Figure 1 shows, the fuel meters consisted of several buret tubes, an electric fuel pump, shut-off valves, and different lengths of rubber fuel-line hosing. This apparatus was mounted on 0.75-in plywood and attached to the automobile, as shown in Figure 2. Each test car was equipped with a tachometer and a vacuum gauge, mounted in the driver's line-of-sight for ease of reading (see Figure 3). Other essential equipment included a thermometer (suspended in one of the burets) for measuring fuel temperature and a stopwatch for recording driving

Table 1. Test vehicle data.

| Item                           | Vehicle      |                 |               |
|--------------------------------|--------------|-----------------|---------------|
|                                | 1            | 2               | 3             |
| Manufacturer                   | Chevrolet    | American Motors | Chevrolet     |
| Model                          | Chevette     | Concord         | Impala        |
| Body type                      | 4-door sedan | 4-door sedan    | Station Wagon |
| Year                           | 1980         | 1980            | 1979          |
| Engine size (in <sup>3</sup> ) | 98           | 151             | 305           |
| Number of engine cylinders     | 4            | 4               | 8             |
| Rear-axle ratio                | 3.70         | 3.08            | 3.08          |
| Normal fuel pressure (psi)     | 4            | 4.5             | 7             |
| Tire type and size             | 13 in radial | 14 in radial    | 15 in radial  |
| Nominal vehicle test weight    | 2550         | 3240            | 4560          |
| Speedometer reading            |              |                 |               |
| Start of testing               | 5600         | 8890            | 16 200        |
| End of testing                 | 8450         | 11 700          | 19 400        |

time (i.e., average travel speed) over the test sections. An observation station, equipped with a thermometer and a recording anemometer, was established at each test site.

### Test Procedures

Operation of the fuel meter was quite simple. Well in advance of the test section, the driver stopped the automobile and the observer filled the two burets with gasoline from the automobile's fuel pump. After the level of gasoline in the test buret was noted and recorded, the auxiliary fuel pump was turned on, the hose from the automobile's fuel pump was shut off, and the three-way valve was adjusted to dispense gasoline from the reservoir buret. The driver started the car and accelerated to the 55-mph test speed before reaching the approach segment. The purpose of the approach segment was to stabilize vehicle performance at test conditions. On reaching the beginning of the test segment (plainly marked by traffic cones on both sides of the pavement), the observer quickly switched the three-way valve so that gasoline was dispensed from the test buret. At the end of the test segment, the valve was switched back to the reservoir buret. Finally, the driver brought the car to a stop, and the observer recorded the fuel remaining in the test buret. Fuel used over the 4000-ft test length was calculated by deducting the initial fuel reading from the final reading. The observer also recorded the temperature of the fuel and the elapsed time for each run. After these measurements, the driver turned the car around and the process was repeated in the opposite direction.

Numerous runs were made by each car at each site. As recommended by Claffey, testing continued until fuel consumption readings from at least six runs in each direction were clustered within several

Figure 1. Wisconsin-type fuel meter.

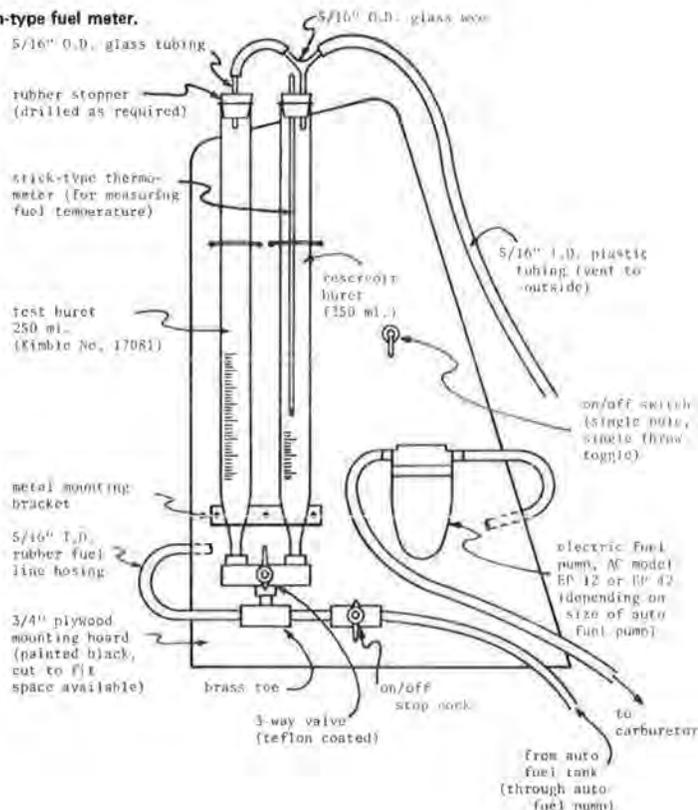


Figure 2. Fuel meter mounted in test vehicle.



Figure 3. Mounting of tachometer and vacuum gage in test vehicle.



milliliters of one another and within several milliliters of the observed readings in the opposite direction, or until variations due to wind or driver fatigue became excessive (in which case, testing was abandoned).

#### Field Testing

Actual field testing proved far more demanding than had originally been expected. Rigid controls were placed on test conditions and procedures to minimize or eliminate the effect of variables other than pavement roughness. The most rigorous controls were placed on wind velocity. Claffey initially suggested that testing be conducted only when winds were less than 3 mph, as measured by the recording anemometer and by cloth indicator flags posted at 200-ft intervals along the test sections. However, it soon became apparent that acceptable results (as gaged by apparent directional differences in fuel consumption) could only be obtained during near-zero winds. Since such conditions were a comparative rarity, testing extended over a six-week period.

Other variables measured and/or controlled during testing were fuel and air temperature, tire pressure, engine vacuum and speed, elapsed test time (vehicle speed), vehicle test weight, and engine operating condition. In the original design of the study, an allowable air-temperature range of 10°F

was stipulated. But when it became apparent that this criterion and the wind criterion could not be met simultaneously, the temperature controls were eliminated. Instead, air temperatures were routinely recorded, along with fuel temperatures, on the presumption that they both could be statistically standardized after testing was completed. For the other variables mentioned, it was possible to meet the established controls. All testing was conducted at a nominal speed of 55 mph, as indicated by elapsed time. For the Impala, individual test runs in which the travel speed deviated from the mean speed for all runs by more than 1 mph were eliminated. For the other two cars, the allowable deviation was  $\pm 0.5$  mph. Constant monitoring of the tachometer and vacuum gage was required to ensure uniformity of travel speed and engine performance. Runs in which the engine vacuum varied by more than 1 in were eliminated. Tire pressure for all cars was maintained at  $32 \pm 0.5$  psi, as determined by readings before, during, and after testing at each site. Fuel tanks were maintained above the one-half level. Unleaded fuel from a commercial source was used throughout the study. To the extent feasible, a single brand of fuel was used. The vehicle test weights given in Table 1 are nominal weights, as indicated. Actual test weight for each car varied from test to test and from run to run due to variations in the quantity of fuel in the gasoline tanks and, in several instances, to changes in crew personnel. However, efforts were made to keep these variations to a minimum, and in no case did they exceed 100 lb.

As originally planned, a specific driver and observer were assigned to each car. Extensive training was conducted to minimize possible idiosyncratic variations and to standardize procedures. Particular emphasis was placed on standardizing driving techniques. Drivers were warned against even the slightest acceleration or deceleration in the test sections and were instructed to monitor engine performance gages constantly. For a variety of reasons, several crew changes were necessary during the course of testing. However, these changes were exclusively transfers from one car to another; in no case were untrained technicians used. In the final tally, a total of 18 tests were considered acceptable (as used here, a "test" refers to the complete series of test runs by one car at one site on one date); the originally assigned crews were responsible for 14 of these.

The engines of all test cars were precisely tuned before any testing started, and no adjustments were made thereafter. No record testing was conducted until the engines were warmed up by driving at least 20 miles at highway travel speed. As an additional precaution against cold engines, and to ensure that test procedures were stable before the start of record runs, the initial runs of every test were eliminated.

#### Testing Considerations

There were eight separate test dates, as indicated in Table 2. Note that in several instances multiple tests were made by a particular car at a particular site. Although it was not completely intentional, it is apparent that the three cars and five sites are distributed quite randomly over these dates. Thus, it is improbable that time produced a systematic variation in the results. Testing was attempted on several other days, but these eight were the only ones for which wind conditions were judged acceptably calm.

The study plan called for testing to be conducted in both directions at each site. Because the se-

Table 2. Test dates and test conditions.

| Test Date | Time (a.m.) | Wind (mph) | Air Temperature (°F) | Chevette      | Concord | Impala        |
|-----------|-------------|------------|----------------------|---------------|---------|---------------|
| Oct. 9    | 7:00-9:00   | 0-3        | 34-46                |               |         |               |
| Oct. 20   | 6:00-7:30   | 0-1        | 47-51                | Site 1        | Site 2  | Site 2        |
| Oct. 21   | 6:30-8:00   | 0          | 32-42                | Site 4        | Site 4  |               |
| Oct. 22   | 6:30-10:00  | 0-1        | 18-38                | Site 2/Site 5 | Site 1  | Site 3/Site 4 |
| Oct. 29   | 6:00-7:30   | 0-1        | 12-21                | Site 3        | Site 5  |               |
| Nov. 5    | 6:00-8:30   | 0-1        | 18-32                |               | Site 1  | Site 1        |
| Nov. 11   | 6:00-8:00   | 0-2        | 20-28                |               | Site 3  | Site 5        |
| Nov. 18   | 6:00-9:30   | 0          | 10-25                | Site 1/Site 5 |         |               |

lected sites were all level and the winds at time of testing virtually calm, it was felt that systematic grade or wind variations would be minimal. Moreover, averaging directional results would tend to neutralize any variation produced by these sources. On four of the five sites, it proved to be possible to test in both directions. On the fifth site, because of an intersecting highway near the eastern terminus, it was possible to test only in the eastbound direction.

The test burets indicated fuel levels to the nearest milliliter, and it was possible, with practice, to read the burets this precisely. The three-way valves proved effective in abruptly starting and stopping the flow of fuel from the burets. Presumably, since the valves were manually operated, some variation was introduced in the readings due to variations in the observer's response time, but these would tend to be neutralized by averaging the multiple runs at each site.

In the operation of the fuel meter, the electric pump delivered fuel from the burets to the carburetor in surges. Therefore, a particular reading could theoretically vary according to the size of the surge. But these surges were small (measured less than 2 mL), and, again, the effect of multiple runs would be to neutralize the variation.

#### Elimination of Unacceptable Runs

In the final tally, the number of directional runs for any test ranged from 9 to 15, depending on the variables cited previously (wind speed, consistency of results, time available, etc.). When testing was completed, the raw data were examined and unacceptable runs were eliminated as noted above. Specifically, this process resulted in the elimination of

1. All runs labeled "abort" at the time of testing (such as for deer in the road or for trucks passing),
2. All first runs in both directions (these were considered "warm-up" runs),
3. All runs in which a fluctuation in engine vacuum greater than 1 in was noted,
4. All runs in which driving time varied from the mean time for each car by  $\pm 0.5$  mph ( $\pm 1$  mph for the Impala), and
5. All "deviant" runs (deviancy here is defined as any lone high or lone low fuel consumption reading that varied from the next highest or lowest reading by more than 1 mL).

After this process, 257 of the original 355 individual runs remained: 100 of 122 for the Chevette, 86 of 133 for the Concord, and 71 of 197 for the Impala. The number of directional runs for any test ranged from 4 to 13.

#### Standardization for Differences in Fuel and Air Temperatures

The processed data were submitted to the University

of Wisconsin-Madison Statistical Laboratory for analysis. The first step was to adjust the observed fuel consumption readings to some standard fuel and/or air temperature, since analysis clearly indicated the existence of a statistically significant relation between these variables. Two methods were considered: (a) adjusting for fuel temperature via volumetric corrections developed by the Society of Automotive Engineers and then adjusting statistically for air temperature and (b) adjusting statistically for fuel and air temperatures simultaneously. These methods were attempted for the three cars individually and for the three cars collectively. The method finally selected is one that simultaneously adjusts for fuel and air temperatures for the three cars collectively and standardizes all readings to the average fuel and air temperature. The mathematical model is as follows:

$$f_s = f_o + 0.10963(t_f - 71.4) + 0.06744(t_a - 28.4) \quad (1)$$

where

- $f_s$  = standardized fuel consumption for any run,
- $f_o$  = observed fuel consumption for any run,
- $t_f$  = observed fuel temperature for any run, and
- $t_a$  = observed air temperature for any run.

The average fuel and air temperatures are 71.4°F and 28.4°F, respectively.

This method generally yielded a high degree of consistency among the readings. Credit for this consistency properly goes to the equipment and the field operations, however, since the temperature adjustments were proportionately small. For all 257 readings, the average adjustment was 1.14 mL and the standard deviation was 0.67.

#### Directional Difference in Fuel Consumption

Despite efforts to eliminate systematic variations in the readings, as described earlier, examination of the data revealed systematic directional differences in average standardized fuel consumption. These differences are summarized below ("difference" is taken as the average standardized fuel consumption for the eastbound or northbound direction minus the standardized fuel consumption for the westbound or southbound direction; \* = statistical significance at the 0.05 level):

| Site | Directional Differences (mL/4000 ft) |         |        |
|------|--------------------------------------|---------|--------|
|      | Chevette                             | Concord | Impala |
| 1    | -0.36                                | +0.25   | -2.71* |
| 2    | +2.43*                               | -1.64   | -2.95* |
| 3    | +0.49                                | -0.02   | +2.43* |
| 4    | +0.26                                | -0.25   | -4.43* |

Because of the difference in sign, these variations cannot be ascribed to directional variations in the road (e.g., in roughness or gradient). Conceivably, they are related to prevailing wind, although winds were virtually nonexistent during

Figure 4. Comparison of fuel consumption versus SI for three test vehicles.

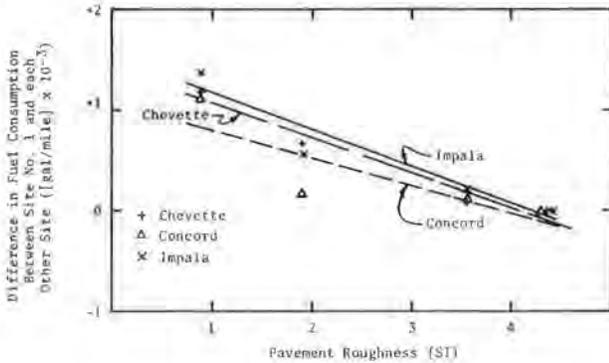
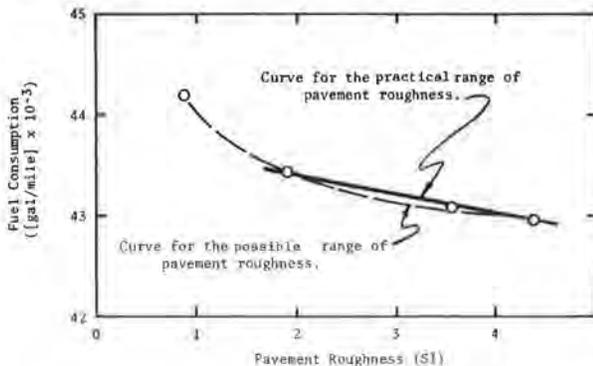


Figure 5. Standardized fuel consumption versus pavement roughness for car traveling at 55 mph on Wisconsin state trunk highways.



record testing. But in fact these variations posed no major difficulties in treating the data, since, in accordance with Claffey's recommendation, the directional averages were themselves averaged to produce a single figure for each site.

Fuel Consumption Versus SI Readings

Average standardized fuel consumption readings were converted from milliliters per 4000 ft, as given in the table above, to the more common units of gallons per mile. To compare the results from the three cars directly, the difference in fuel consumption between site 1 (the smoothest surface) and each of the four other sites is given below:

| Car      | Difference in Fuel Consumption<br>[(gal/mile) x 10 <sup>-3</sup> ] |        |        |        |
|----------|--|--------|--------|--------|
|          | Site 2   | Site 3 | Site 4 | Site 5 |
| Chevette | +0.077   | -0.056 | +0.673 | +1.189 |
| Concord  | +0.098   | -0.188 | +0.154 | +1.099 |
| Impala   | +0.206   | -0.293 | +0.548 | +1.395 |

Note that positive signs in this table indicate that fuel consumption for the particular site and car was greater than for the smoothest surface; negative signs indicate lesser fuel consumption than for the smoothest surface.

It is evident at once that a similar pattern exists for the three cars, the principal features of which are the following:

1. Substantially more fuel was consumed on site 5 (the roughest pavement) than on the other sites.
2. Less fuel was consumed on site 3 than on any other site.
3. For sites 1, 2, 4, and 5 (i.e., from the

smoothest to the roughest pavement), there is a progressive (although irregular) increase in fuel consumption.

Site 3 presents analytical problems. For all three cars it runs counter to the pattern of fuel consumption indicated by the other sites, and it differs markedly from site 4, although measured roughness levels at the two locations are quite close. In seeking an explanation for the apparently anomalous readings at this site, one could look at (a) the assigned SI value, (b) the recorded air or fuel temperature, (c) the calculated temperature adjustment factors, (d) the possibility of peculiarities in the pavement surface, or (e) the fuel consumption measurements themselves. Since there are difficulties with all these explanations, it seems wisest simply to recognize the site as anomalous, to omit it from further analysis, and not to attempt a rationalization at this time.

The data given in the table above for sites 1, 2, 4, and 5 are plotted versus SI in Figure 4. Linear regression analysis using the plotted points indicates slopes of -0.000 347 2, -0.000 269 3, and -0.000 368 7 for the Chevette, Concord, and Impala, respectively. Undoubtedly, better fit could be obtained with some nonlinear expression, but the straight-line model is immediately useful for comparing the results of the three cars.

The consistently negative signs of the slopes and the general agreement in their size are significant; that is, fuel consumption for each car increased as roughness increased (or as SI decreased), and the individual increases are of the same order. Clearly, there are differences in the pattern of data points from car to car that cannot be explained by differences in vehicle size, since the lowest indicated slope is for the middle-sized car (Concord). Thus, for the three cars included in this study, the effect of pavement roughness was not an apparent function of vehicle size.

When the three data points at each of the four sites are averaged, the results are as plotted in Figure 5. The dashed curve represents the indicated relation between SI and fuel consumption for automobiles at 55 mph over the range of pavement roughness included in this study. The data in this curve support the findings in earlier studies that fuel consumption increases as pavement roughness increases. For SI levels above about 2.0, this increase appears quite modest; at lower levels it is more significant. Between the smoothest and roughest surfaces (SIs of 4.4 and 0.9, respectively), the indicated increase is about 3 percent. As a practical matter, however, SI levels below 1.5 are rarely encountered for project-length segments of the Wisconsin state trunk system. Thus, it would seem that the solid line, which closely approximates the dashed curve over the SI range of 1.5 to 4.5, reasonably depicts the apparent practical relation between fuel consumption ( $F_s$ ) and pavement roughness (SI). The regression model for this line is as follows:

$$F_s = 0.043 771 - 0.000 187 9 \times SI \tag{2}$$

where  $F_s$  is in gallons per mile.

Using this relation gives an indicated increase in fuel consumption of about 1.5 percent between SIs of 4.5 and 1.5, under the conditions of this study. For practical application, it appears that the slope of this line, 0.000 187 9, is appropriate for translating differences in pavement roughness into differences in automobile fuel consumption.

It should be noted that, even if one allows normalizing results from different sources, the in-

crease in fuel consumption on rough pavements reported here is considerably less than the increases reported in some earlier studies. But since the curve in Figure 5 is distinctly nonlinear over the possible range of roughness, comparisons such as this may be misleading. For example, assignment of an improper objective roughness level to Claffey's "badly broken and patched" surface could be of major consequence. Differences in objective roughness scales, as between SI and Zaniewski's quality index, could also be of major consequence. Therefore, comparisons of reported relations between fuel consumption and pavement roughness require great care.

#### CONCLUSIONS

Based on an analysis of data collected in this study, and within the limits implied or expressed, the following conclusions are made:

1. The effect of pavement roughness on automobile fuel consumption, because it is proportionately small, can be overwhelmed by the effect of more significant variables--e.g., travel speed, road gradient, driving habits, and wind velocity. In particular, wind can pose great difficulties in conducting field tests on fuel consumption. For this study, reliable data were obtained only during virtual dead-calm conditions.

2. To detect the relatively small variations in fuel consumption produced by pavement roughness, equipment capable of very accurate measurement is essential. The manual fuel meter developed for this study and described in this paper proved capable of measuring fuel consumption accurately to 1 mL, which for the 4000-ft test segments used here is considered minimum acceptable accuracy.

3. The collected data indicate that automobile fuel consumption increases as pavement roughness increases, where roughness is measured in terms of SI. Between the smoothest and roughest pavement included in this study (SIs of 4.4 and 0.9, respectively), the indicated increase was about 3 percent, which appears to be considerably less than has been reported by other investigators. However, since the relation between fuel consumption and pavement

roughness determined in this study is distinctly nonlinear, comparison with results from other studies must be done with great caution.

4. Although the relation between fuel consumption and pavement roughness appears nonlinear over the range of roughness included in this study, for the pavements normally encountered on Wisconsin state trunk highways (SIs ranging from 4.5 to 1.5), the relation can be approximated by a straight line that has a slope of 0.000 187 9 gal/mile/SI. This study indicates that, for the conditions described here, an automobile traveling on a paved surface that has an SI of 1.5 consumes about 1.5 percent more fuel than it would consume traveling on a paved surface that has an SI of 4.5.

5. For the three automobiles used in this study, the relation between fuel consumption and pavement roughness was not an apparent function of vehicle size.

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## Rational Seasonal Load Restrictions and Overload Permits

BILLY CONNOR

Seasonal load restrictions have been enforced in Alaska since the first paved road in 1950. The time frame and level of such restrictions have historically been based on the experience and judgment of maintenance personnel. This results in a lack of continuity from region to region. A rational load-restriction policy has been developed based on the load-damage relations on a pavement structure. Deflection data are used to monitor the strength of the embankment and thus to provide the information on which to base the time frame and level of restrictions. A policy on overweight-vehicle permits is presented based on the ability of the roadway system to carry the load and the load-damage relations. The policy uses the philosophy that the user pays for any damage in excess of that which would be incurred by legal loading.

The relation between vehicle weights and the performance of the pavement structure has been recognized for many years. Because of the large decrease in pavement life due to heavy vehicle weights and

the rapid rise in maintenance costs, strict control of excessive truck weights has become imperative.

The problem of controlling excessive truck weights becomes crucial when the stress in the asphalt layer exceeds the tensile strength of the material. When this occurs, failure is immediate. Failure due to high tensile stress is not likely to happen under normal conditions. However, for highways in cold climates, this may well be the case if the pavement structure is weakened during the spring-thaw period. To protect these highways during the thaw-weakened period, a decision must be made concerning the level and duration of spring load restrictions.

In Alaska, as in most other states, such decisions have typically been made by maintenance engi-

neers based on their experience and judgment. Studies based on deflection testing have shown that these decisions are sometimes inaccurate, which often results in premature pavement failure. In other cases, load restrictions have been imposed on roadway sections unnecessarily. A rational policy of seasonal load restrictions should be established to protect weakened roads. At the same time, the trucking industry should not be unduly burdened by unjustifiable load restrictions.

Unfortunately, the problem of excessive truck weight is not limited to spring-thaw periods. Too many of the trucks traveling the nation's highways exceed state weight limits. The key reasons generally cited are lack of enforcement and fines that are too small to be a deterrent. The cost and difficulty of obtaining overweight-vehicle permits vary greatly. In Alaska, as in many other states, the cost is based on administrative costs and not on the damage to the roadway. The damage to the roadway surface by overweight vehicles is borne by all road users (1).

#### EFFECTS OF WEIGHT ON PAVEMENT PERFORMANCE

The effect of axle weight on the performance of pavements has become a common topic of conversation among transportation engineers. Because of limited highway and airport funding, pavement management has become imperative. The primary aim of any pavement management method is to optimize the life of a pavement. Because the major factors affecting pavement life are the size and frequency of the loads imposed on the pavement structure, there is concern about the regulation of axle weights. Since the weight of the automobile is relatively small, the major portion of fatigue damage can be attributed to truck traffic. In fact, the proportion of pavement damage that can be directly attributed to trucks has been estimated to be as high as 90 percent (1).

Because heavy axle loads are responsible for such a large portion of the damage to the nation's highways, their effect on pavement performance must be understood. Many empirical and theoretical studies have been undertaken to describe these relations. Because of the importance of weight-damage relations in seasonal load restrictions and overweight-vehicle permit policies, a detailed discussion is given here.

#### Development of Weight-Damage Relations

Several studies have been undertaken to delineate the relations between load and damage on pavement structures. Most of these studies use an equivalent axle-load factor (F), which is defined as the ratio of the damage caused to a pavement by a single pass of a given vehicle to the damage caused by a single pass of a standard vehicle. This relation can be mathematically represented as follows (2):

$$F = D_x/D_s \quad (1)$$

where  $D_x$  is the damage caused by a single pass of a specified vehicle and  $D_s$  is the damage caused by a single pass of a standardized vehicle.

The American Association of State Highway Officials (AASHO) Road Test formula, which predicted this ratio, is

$$F = [(W_2 + 1)/(W_1 + 1)]^{4.79} \quad (2)$$

where  $W_2$  is the axle weight of the vehicle in question (kips) and  $W_1$  is the axle weight of the standard vehicle (kips).

Typically, a single-axle load of 18 000 lb is defined as a standard because it was the legal maxi-

mum load in most states at the time of the AASHO Road Test (1958-1960). A 33 000-lb tandem axle load is usually assumed to have an equivalency factor of 1.0. Figure 1 compares the equivalency factors derived from the AASHO Road Test with those from the Pennsylvania Department of Transportation (PennDOT) Test (3). The agreement is quite good.

Recently, computer modeling has made use of mechanistic analysis to predict load-related damage. Since tensile strain at the bottom of the pavement layer is usually associated with fatigue cracking, the following equation was developed by Deacon to compute equivalency factors (2):

$$F_i = (e_i/e_b)^c \quad (3)$$

where

- $F_i$  = equivalency factor,
- $e_i$  = maximum tensile strain due to the axle-load configuration,
- $e_b$  = maximum tensile strain due to an 18-kip single-axle load, and
- $c = 5.5$ .

Note that Equations 2 and 3 are similar; indeed, it has been shown that they compare favorably.

#### Multiple Axles

Most of the work in weight-damage studies has assumed single axles and tandem axles only. In 1979, Havens, Southgate, and Deen (3) used a modified Chevron N-Layer computer program to predict damage to pavement from multiple axles. The following conclusions were reached:

For most highway vehicles, the deflections caused by a set of dual tires will be influenced by the dual tires on the opposite end of the axle. Similarly, the addition of another axle has a modifying influence on the deflection bowl of the single axle. In a three-axle group, maximum deflection will occur beneath the inner tire on the center axle. However, fourth and/or succeeding axles are located far enough from the "center" axle of the triaxle group as to have almost no effect on the magnitude of the deflection, but such additional axles do affect the horizontal dimension of the deflection bowl. Thus, the total load on a given group divided by the number of axles indicates that, for four or more axles, the total load can be increased by approximately 83.5 kN (18 800 lb) for each additional axle.

Figure 2 (3) shows the damage factors for various axle groupings based on the ratio of work strain at any given load to work strain for the 18-kip axle load. It is obvious that, whenever possible, the use of additional axles should be encouraged, especially in lieu of overweight permits.

Havens, Southgate, and Deen stress that the steering axle is a major source of damage. This damage has been shown to be especially large on pavements less than 2 in thick, such as those typically used in Alaska. The excess load on the steering axle should therefore be kept to a minimum.

Because damage caused by increased axle loadings is not linear, as has been shown, loads should be distributed over as many axles as practical. For example, 34 000 lb carried on a tandem axle may cause only approximately one-twentieth the damage of the same load carried on a single axle.

The steering-axle weight should be kept as low as practical because of the magnitude of the damage caused by heavy steering axles, especially on thin

Figure 1. Equivalency factors from AASHO and PennDOT Road Tests.

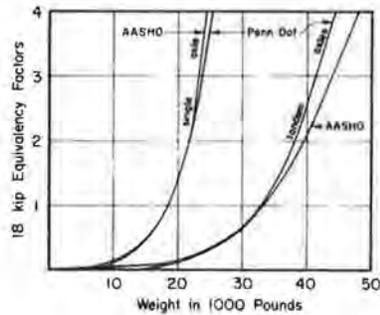
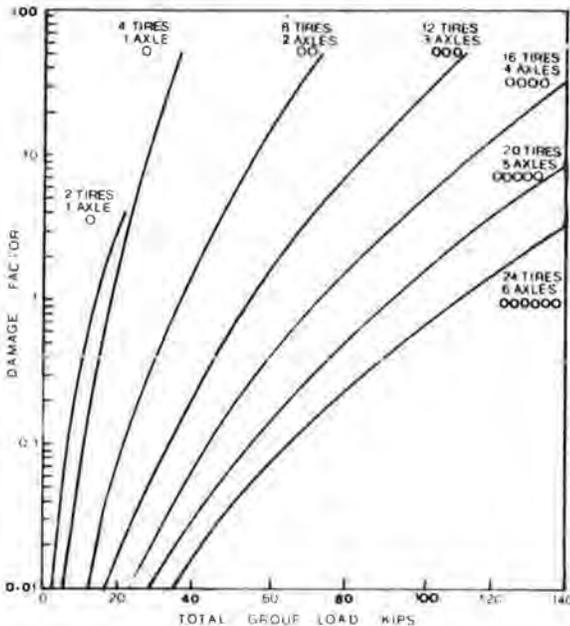


Figure 2. Damage factors for various weights on different axle configurations.



pavements. As Figure 2 shows, the damage factor for a legal 20 000-lb axle load on a two-tire front axle is twice as great as the same load on a four-tire rear axle. To be comparable, the legal front axle loading should be limited to 18 000 lb.

#### SPRING LOAD RESTRICTIONS

Structural damage done to pavements during spring thaw can result in very high maintenance costs. Spring load restrictions are often imposed during this period to limit damage to the roadway, which results in higher shipping charges to the public. When load restrictions reach a level of about 50 percent of the legal axle load, the route is effectively closed to most truck traffic. A balance must therefore be sought.

Most state agencies currently base the level and duration of seasonal load restrictions on the experience and judgment of maintenance engineers. However, quantitative methods of establishing load restrictions can and have been developed by using measurements of vertical pavement surface movements under a standard load, commonly termed "deflection test data". These methods allow each roadway section to be analyzed to minimize economic losses to all parties.

#### Current Seasonal Load-Limit Policy in Alaska

Currently, spring-thaw load limits are set by the

regional maintenance engineer based on the engineer's judgment and experience. To an increasing extent, these decisions are influenced by deflection testing, most of which was accomplished for research programs. In the past two or three years, the use of deflection testing has become more common on urban routes; however, rural routes have no testing other than that performed by researchers.

Unfortunately, the maintenance engineer must give the trucking industry ample notice before setting load restrictions. Under the present policy, the maintenance engineer is forced either to wait until load restrictions are required before setting limits or to try to outguess the weather. However, as will be discussed later, this can be overcome by using deflection test histories for different roadway sections.

#### Current National Seasonal Load-Limit Practices

A review of the literature shows that 42 states and Canadian provinces experience seasonal freezing of roadways (4). Of these, 17 impose spring load restrictions. Ten states rely solely on experience and judgment, 5 use Benkelman deflection testing, 1 state uses plate load testing and the Benkelman beam, and 1 uses visual inspection and the Dynaflect deflection test method.

From the above summary, it is evident that procedures to establish spring-thaw load limits continue to be based on experience, even though quantitative procedures for assessing pavement behavior under load during the spring-thaw period have been available since the 1969 publication of NCHRP Report 76 (5). Most procedures use pavement deflection and/or curvature to determine the seasonal variations in pavement response. By using these procedures, seasonal load restrictions can be determined simply and economically.

#### Establishing a Spring Load-Limit Criterion

Hardcastle and Lottman (4) stated an appropriate philosophy for implementing load restrictions:

The maximum axle load allowed on a pavement while it is in a spring-thaw weakened condition should be restricted to the load that produces the same horizontal tensile strain in the asphalt stabilized surface or base layer as is produced by the maximum axle load allowed on the pavement during the previous summer-fall (normal) period.

The accomplishment of this objective can be divided into three comprehensive tasks:

1. Define the pavement structure and determine its response to maximum legal axle loads during noncritical periods (summer and fall),
2. Determine the response of the pavement structure to loading during critical periods (spring thaw), and
3. Determine the maximum allowable axle load that can be applied during weakened periods without exceeding the horizontal tensile strains that would be induced by the maximum allowable axle loads during noncritical periods.

#### Determination of Pavement Response to Axle Loads

Tasks 1 and 2 require the determination of the response of the pavement structure to the axle loads imposed. This can be accomplished by determination of horizontal strains in the pavement but requires sophisticated computer modeling techniques and a

considerable amount of data about the materials properties. These properties must be determined for the period in question.

Deflection data allow the in situ measurement of the pavement response without direct measurements of materials properties. Because of the ease with which deflection data can be obtained, deflection testing has become quite widespread. Three basic methods for obtaining deflection data are (a) static rebound (Benkelman beam), (b) vibratory response (Dynalect and Road Rater), and (c) impact loading. Since the static-rebound methods have become standardized, the relations presented in this paper will be based on this method. Although correlations have been somewhat limited, standard practice has been to relate other methods to the static-rebound method.

#### Maximum Allowable Load During Critical Periods

Protection of roadways from premature failure is the end goal of a procedure for establishing load limits. However, this is no easy task. The maximum allowable axle load during critical periods must be determined to accomplish this task. If the philosophy adopted is that of limiting damage to what would normally be expected, the damage that could be expected both in the spring-thaw-weakened condition and in the summer-fall condition for various loads must be established.

As explained earlier, equivalency factors between different axle weights have been established by both theoretical and empirical approaches. Since the agreement between approaches has been good and since computers are not readily available to maintenance engineers, an empirical approach, based on deflection measurements, will be developed. Implied in this approach is the concept that the pavement deflection under a wheel load represents the strength of the pavement structure and that the strength of the pavement structure is directly related to pavement performance (4).

Figure 3 shows the relation between equivalent axle loads (EALs) per day, maximum annual deflection, and pavement life with no load limits as developed in Ontario, Canada (6). As expected, the pavement life decreases at a fixed number of EALs per day, commonly termed design traffic number (DTN), as the deflection increases. If, for example, the DTN is 800 EALs/day and the deflection level is 0.023 in, the life expectancy would be 20 years. Should the deflection level be increased to 0.031 in during the spring thawing period, the life expectancy would be reduced to 10 years. From Figure 4, a means of maintaining the 20-year design life at the 0.031-in deflection level would be to reduce the DTN to 390 EALs/day during spring thaw. Two methods are available to accomplish this: (a) reduce the number of axles that traverse the pavement or (b) reduce the maximum legal axle weight. The first method may cause hardships, since some vehicles will not be allowed to travel. The second method reduces for the freight industry the economic attractiveness of hauling over the highway system. When applied to spring load restrictions, the second method can be used most fairly and easily.

The reduced maximum axle load must now be determined. The ratio of the DTN at the higher deflection level to the desired deflection level is equal to the equivalency factor discussed earlier. Mathematically, this is represented as

$$F = \text{DTN (lower)}/\text{DTN (desired)} \quad (4)$$

By equating this to the equivalency factor described by the AASHTO Road Test, the following relation is formed:

$$\text{DTN (lower)}/\text{DTN (desired)} = [(W_2 + 1)/(W_1 + 1)]^{4.79} \quad (5)$$

Rearranging the equation and solving for  $W_2$ ,

$$W_2 = \{ [\text{DTN (lower)}/\text{DTN (desired)}]^{0.209} (W_1 + 1) \} - 1 \quad (6)$$

Continuing the previous example,

DTN (lower) = 390 EALs/day.

DTN (desired) = 800 EALs/day.

$F = 390/800 = 0.49$ .

Assume that  $W_1 = 18$  kips and  $W_2 = (0.49)^{0.209}(19) - 1 = 15.4$  kips, the maximum allowable single-axle load.

Load restriction =  $15.4/18$ , or 86 percent of the maximum legal axle load.

The technique used above allows the calculation of axle weights that will cause (on weaker pavements) the same damage as the heavier loads on stronger pavements. As has been shown, a single pass of a 15.4-kip load on a weakened pavement structure that has a Canadian Good Roads Association deflection of 0.031 in is equivalent to an 18-kip load on a pavement that deflects at 0.023 in.

This can be expanded to spring-thaw conditions. If a pavement has a summer deflection level of 0.023 in and the maximum legal axle weight is 18 kips, what load restriction should be imposed if the spring deflection increases to 0.031 in? If one applies the equal-damage philosophy, the maximum legal axle weight should be lowered to 15.4 kips, or 86 percent of the normal. The results of this procedure are graphically shown in Figure 4. The percentage reduction in axle weight can easily be determined by using Figure 4 and knowing the normal summer deflection level and the present deflection.

Because deflection data on Alaskan highways have been measured on only 150 miles, an alternative method of determining the acceptable deflection level must be provided until such data are available. Figure 5 is an adaptation from the California design procedure and shows the acceptable deflection levels for various pavement thicknesses and traffic indexes (TIs). If one knows the TI and the thickness of the constructed asphalt concrete pavement, the appropriate deflection level can be determined. For example, a route with a TI of 8 and a pavement thickness of 2 in would have an acceptable deflection of 0.034 in. Again, it is better to have measured the normal summer deflection.

To this point, the procedure implies that all roadways should be posted during spring thaw if the deflection level increases above the normal summer level. This is not practical nor necessarily desirable. It is therefore necessary to establish a procedure by which routes that do not need load restrictions can be determined.

Hardcastle and Lottman suggest that any route that has a fall deflection greater than 0.023 in should be posted in the spring. However, several cases have been observed in Alaska where sections had very low fall deflections and very high spring deflections. Obviously, in such instances, a fixed fall deflection criterion would be unacceptable.

An alternative procedure requires that the design deflection be used. If the spring deflection exceeds the design or acceptable deflection level, then load limits should be enforced. If not, the roadway need not be restricted. From Figure 5, the maximum allowable deflection level for a pavement thickness of 2 in and a TI of 8 would be 0.034 in. Should the measured spring deflection exceed 0.034 in, the load restriction applied would be determined by referring to Figure 4 and knowing the normal or acceptable deflection level.

Figure 3. Design deflection curve for various vehicle loadings.

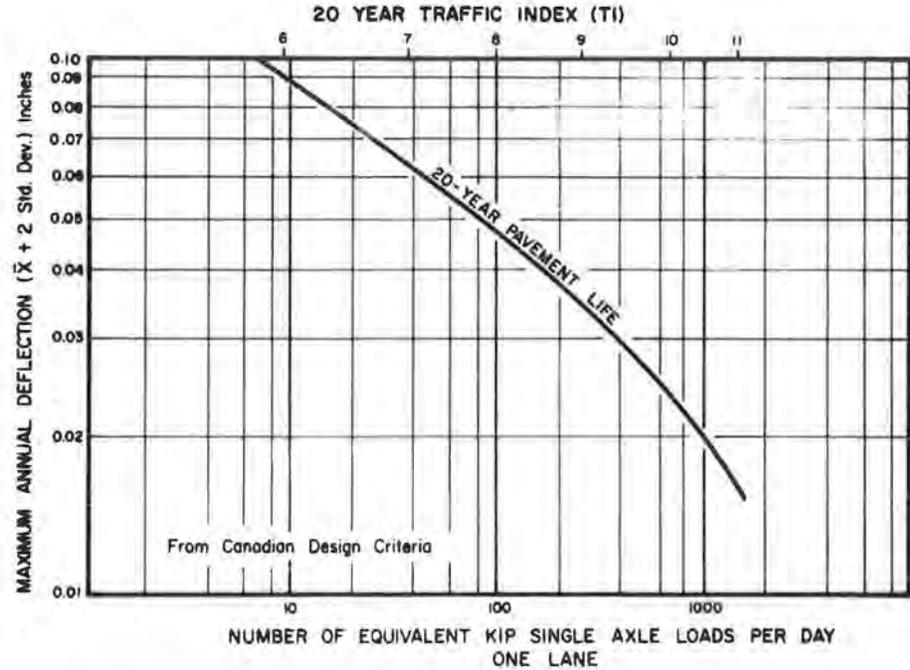


Figure 4. Load-limit percentages given measured deflections and normal summer deflection levels or acceptable deflection levels.

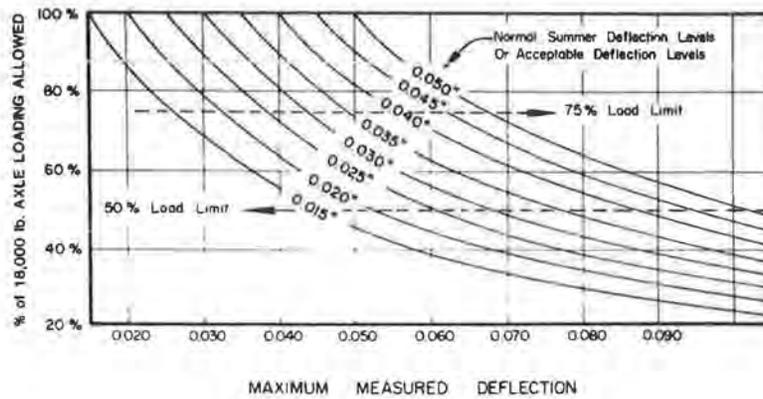


Figure 6 shows a flow chart that summarizes the procedure for establishing the level of spring load restriction. To illustrate its use, consider the following example.

The peak spring deflection of a roadway section has been measured to be 0.045 in. The pavement thickness is 2 in and the TI is 8. The normal summer deflection is unknown. Determine the level of load restriction that will be required. Since the normal summer deflection is unknown, the allowable deflection level for a TI of 8 and a pavement thickness of 2 in from Figure 6 is found to be 0.034 in. The measured peak spring deflection of 0.045 in is greater than the allowable deflection (0.034 in). Therefore, load restrictions will be required. From Figure 5, the level of restriction required for a peak deflection of 0.045 in and an allowable deflection of 0.034 in would be 80 percent.

Economic Impact of Load Restrictions on the Trucking Industry

Whenever load restrictions are imposed, the economic impact on the trucking industry is substantial. However, any additional damage done by trucks during this period is in effect subsidized by the general

public. Because the general public is also the consumer, it ultimately pays for additional costs to the freight haulers. A balance must therefore be sought between the costs to the trucking industry and the costs of repairing and prematurely replacing roads.

Whenever load restrictions are in effect, trucking agencies can add additional axles, split loads, or divert shipments to rail or air carriers. Whenever additional axles are added, fuel consumption increases approximately 5 percent/axle, which adds about 4¢/mile to the cost. The cost of mounting an additional axle is about \$50. These axles are often added during spring breakup, when loads that cannot be broken down must be hauled.

Shipment by rail is less expensive than trucking any time of the year. The major complaint is that it takes two or more days longer to move freight from Anchorage to Fairbanks by rail than by truck. For communities not in the rail belt, trucks are the only means of freight transport.

Air shipment is much too expensive for heavy freight movement. However, in some instances the cost may well be justified. Again, communities that do not have air strips capable of handling heavy aircraft must depend on trucks.

Figure 5. Acceptable deflection levels for various pavement thicknesses.

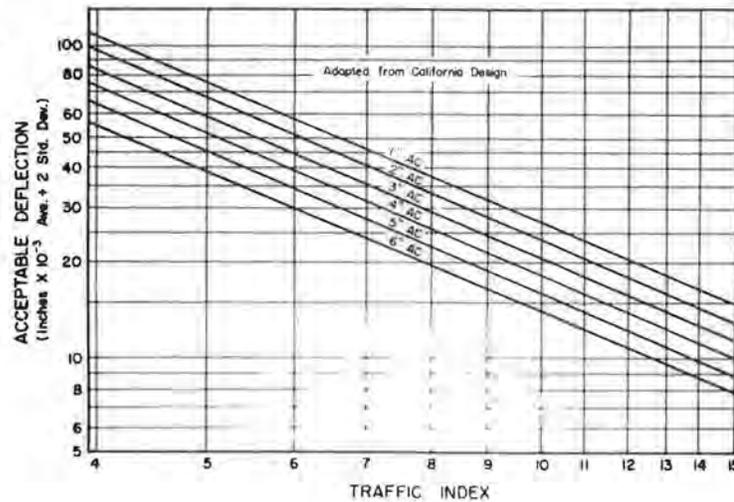
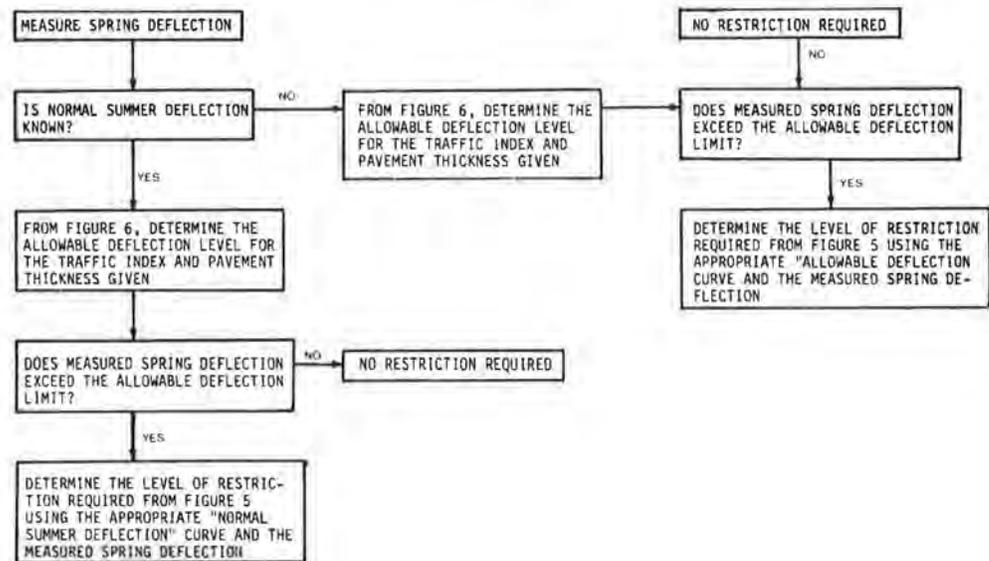


Figure 6. Flowchart for establishing spring load restrictions.



Interviews with several trucking firms operating within Alaska provided cost information for use in this study. These data are summarized below:

| Area                                | Condition         | Total Cost (\$/truck mile) |
|-------------------------------------|-------------------|----------------------------|
| South of Fairbanks                  | Paved             | 1.62                       |
| North of Fairbanks                  | Gravel            | 2.56                       |
| Alcan Highway, Seattle to Fairbanks | Paved and unpaved | 1.21                       |

These costs include only operating costs--i.e., gas, oil, tires, and driver. The less tangible costs, such as safety, are not estimated here because no uniform method of estimation has been developed. When load restrictions are imposed, added trucking costs include the costs for additional trucks, trailers, and axles and the costs of transferring part of the load to the additional trailers. Figure 7 shows the relation between the percentage load restriction and payload for a 3-S2 tractor-trailer combination. Notice that, when the road is posted for 30 percent of legal axle load, no freight can be moved. The cost relations are shown in Figure 8. Understandably, most trucking agencies stop hauling

when the load limits reach 50 percent because the cost becomes prohibitive. Similar plots for other truck types could easily be derived.

To amplify the magnitude of these costs to the trucking industry, consider the total daily cost increases incurred during the period when 75 percent axle restrictions are imposed. The average truck traffic on the Parks Highway for March and April of 1979 was about 140 trucks/day. Assuming that all of these trucks are traveling full, the increased cost would be \$33 000/day on the Parks Highway alone. The costs for the major routes within Alaska are summarized in Table 1, where they average out to be \$66/day/mile.

Cost to State If Load Restrictions Are Not Applied

Whenever a fully loaded truck travels a highway that is in a thaw-weakened condition, excessive damage occurs just as if it were overweight. The damage to the pavement can be estimated by redefining the "legal" load. For example, if restricting axle loads to 75 percent of the legal maximum is necessary to protect the pavement (using Equation 2), an 18-kip single axle would have an equivalency factor

Figure 7. Relation between payload and percentage restrictions for 3-S2 truck.

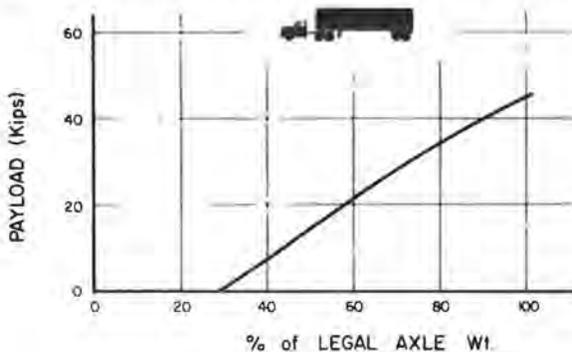


Figure 8. Cost per 1000 lb per mile with decreased axle weight for 3-S2 truck.

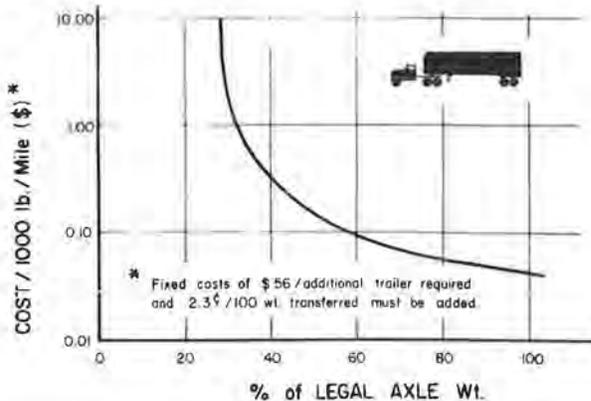


Table 1. Cost to trucking industry when 75 percent highway load restrictions are applied.

| Route                    | Length (miles) | No. of Trucks per Day | Cost per Day (\$000s) |
|--------------------------|----------------|-----------------------|-----------------------|
| Parks                    | 360            | 140                   | 33                    |
| Sterling                 | 139            | 181                   | 17                    |
| Seward                   | 129            | 228                   | 19                    |
| Glenn                    | 189            | 107                   | 13                    |
| Richardson               |                |                       |                       |
| Fairbanks-Delta Junction | 97             | 68                    | 4                     |
| Delta-Glennallen         | 151            | 35                    | 4                     |
| Glennallen-Valdez        | 115            | 61                    | 5                     |
| Alaska Highway           |                |                       |                       |
| Border-Tok               | 93             | 12                    | 1                     |
| Tok-Delta                | 108            | 22                    | 2                     |
| Tok Cutoff               | 125            | 22                    | 2                     |
| Total                    |                |                       | 100                   |

of 3.65. If a 50 percent restriction is required, the equivalency factor would be 21.64. If one knows the number of equivalent EALs, the cost to the state can be computed as follows.

Based on construction costs in interior Alaska, the present cost (1980) of a 2-in hot asphalt overlay 24 ft wide in Alaska is approximately \$13/linear ft. A 2-in minimum overlay has been suggested in a recently completed study by McHattie, Connor, and Esch (7). The average peak deflection on the major routes within the state has been found to be 0.036. By using a 20-year design life and Figure 5, the current pavement cost to the state for each EAL-mile would be  $c = \text{cost/mile/EAL life}$ . From Figure 3, EAL life = 1 124 200. Therefore,  $c = 13(5280)/1\ 124\ 200 = \$0.061/\text{EAL-mile}$ . A single pass of a legally

loaded 3-S2 truck would cost \$0.183/mile. If the same vehicle were to pass when 75 percent axle-load limits were required, the cost to the state would be \$0.668/mile. The incremental cost would then be \$0.485/mile/pass. Table 2 summarizes the cost to the state, which averages out to be \$106/day/mile. The benefit/cost ratio for applying 75 percent load restrictions is then 1.6. It seems prudent, therefore, to apply load restrictions for the entire period required.

#### Sampling Frequency

Because of the expense involved and limited availability of personnel and equipment, testing an entire system annually would be impractical. A more practical method would be to determine those sections that are representative of a route through a deflection inventory. These sections could then be monitored during the spring thaw to determine when load restrictions should be applied. Care should be taken to select sections in each climatic region to ensure that load restrictions are not removed prematurely.

The inventory could also be used to establish the level of restriction required for each route. This would require management to establish what level of protection would be required for the routes in question. For example, it may be decided that load restrictions should be based on the deflection level below which 95 percent of all roadway sections fall.

#### Summary

Although the mechanism for thaw weakening is reasonably well understood, prediction of the magnitude and the time frame still must be measured in the field if accurate information is required. The use of deflection data allows an in situ measurement of the strength of the pavement structure without direct measurements of materials properties.

Relations between deflection levels and pavement life have been developed by a number of agencies. By combining these relations with the weight-damage relations found in the AASHTO Road Test, the level of restriction can be ascertained. The underlying philosophy behind this determination is that damage allowed during the weakened period should be no more than the damage that would have occurred during the normal summer months.

By monitoring the deflection levels of carefully selected roadway sections, the time frame for load restrictions can easily be determined. Since the benefit/cost ratio is in favor of applying load restrictions, it is suggested that load restrictions should be maintained until it has been determined that the embankment has regained its strength. Since trucking agencies know within a few weeks the time when load restrictions will be enforced, they should be encouraged to schedule around them.

#### OVERWEIGHT-VEHICLE POLICIES

Although it is recognized that overweight vehicles are necessary, they must be closely regulated to prevent pavement damage. Nationally, it has been found that illegally overloaded vehicles travel the highways with little fear of enforcement (1). As a result, pavement life is being drastically reduced without compensation to the state governments. This requires not only effective enforcement but also a fair permit policy.

Any good permit policy should have three basic qualities:

1. It must strongly encourage legal loads whenever possible.

**Table 2. Cost to state when 75 percent highway load restrictions are not applied.**

| Route                    | Length (miles) | No. of Trucks per Day | Cost per Day (\$000s) |
|--------------------------|----------------|-----------------------|-----------------------|
| Parks                    | 360            | 140                   | 30.6                  |
| Sterling                 | 139            | 181                   | 32.1                  |
| Seward                   | 129            | 228                   | 37.5                  |
| Glenn                    | 189            | 107                   | 25.8                  |
| Richardson               |                |                       |                       |
| Fairbanks-Delta Junction | 97             | 68                    | 8.4                   |
| Delta-Glennallen         | 151            | 35                    | 6.7                   |
| Glennallen-Valdez        | 115            | 61                    | 9.0                   |
| Alaska Highway           |                |                       |                       |
| Border-Tok               | 93             | 12                    | 1.4                   |
| Tok-Delta                | 108            | 22                    | 3.0                   |
| Tok Cutoff               | 125            | 22                    | 3.5                   |
| Total                    |                |                       | 158.0                 |

2. Enforcement must be adequate to discourage movement of overweight loads without a permit.

3. The fines must be sufficiently large to reduce the economic attractiveness of avoiding permits.

Although there is little disagreement on the first requirement, lack of enforcement, low permit costs, and inadequate fines have been cited as primary reasons why so many overweight vehicles are operating on the nation's highways.

The overweight-vehicle policy described in this paper is based on the "user pays" philosophy. The permit cost is based on the estimated cost to the state for the overweight vehicle, including costs for pavement damage and administrative costs. The economic evaluation of whether to break the load down, add more axles, or pay for overweight loads is borne by the trucking company.

Current Alaska Policy on Overweight Vehicles

Under Alaska statutes (Section 19.10.060), the Commissioner of Transportation and Public Facilities is authorized to regulate the maximum size and weight of all vehicle operations on highways under his jurisdiction. The Alaska Administrative Code (17 AAC 25.0) stipulates the maximum vehicle weights and dimensions (7). This section also states that "Police officers or specifically authorized employees of the Department of Highways are delegated the authority to enforce these regulations."

The maximum axle loadings are given below:

| Axle No. or Group | Maximum Weight (lb) | Minimum Axle Spacing (ft) |
|-------------------|---------------------|---------------------------|
| Single            | 20 000              | 10.0                      |
| 2-axle tandem     | 34 000              | 3.5                       |
| 3-axle tandem     | 42 000              | 3.5                       |
| 4-axle tandem     | 50 000              | 3.5                       |

The maximum gross vehicle weights are determined by the following equation:

$$W = 500 \{ [LN/(N - 1)] + 12N + 36 \} \tag{7}$$

where

- W = maximum gross vehicle weight to the nearest 500 lb,
- L = distance between the extreme axles (ft), and
- N = number of axles.

The weight on any tire may not exceed 500 lb per inch of tire width.

The maximum single-trip permit limits that may be issued by the district permit offices are given below:

| Axle No. or Group | Maximum Weight (lb) |
|-------------------|---------------------|
| Single            | 30 000              |
| 2-axle tandem     | 50 000              |
| 3-axle tandem     | 60 000              |
| 4-axle tandem     | 70 000              |

Tire loading is 750 lb/in, and gross vehicle weight (GVW) is 150 000 lb. Maximum GVW is 25 percent above the lowest posted weight limit on bridges.

For overweight-vehicle permits that exceed the values in Table 1, approval is required by the Department of Public Safety through the State Permit Officer. There is no mention of designating routes that may be used. The fee schedule for overweight-vehicle permits is given below:

| Category  | Fee (\$) |
|---|----------|
| Single trip of 0-99 miles; less than 150 000 lb GVW             | 10       |
| For 30-day permits; not to exceed 15 percent over legal GVW     | 25       |
| Single trip of 100 miles or more; less than 150 000 lb GVW      | 25       |
| Single trip of 0-99 miles; for GVW of 150 000 lb or more        | 25       |
| Single trip of 100 miles or more; for GVW of 150 000 lb or more | 50       |

The total permit fee charged for any permit shall be the sum of the oversize and overweight fees as required.

Overweight-Vehicle Policies of Other States

A review of the overweight-vehicle policies of other states showed a wide variety of permit schedules. The policies can be summarized as follows: Eleven states have free permits, 22 have fixed fees, and 17 have permit fees based on excess weight (4). Of the 17 states that have graduated fees based on excess weight, 6 vary the fees with both excess weight and mileage.

Many philosophies exist concerning overweight-vehicle policies. If overweight loads are considered a normal part of a roadway's function, then permits may be free. Other states feel that the permit cost should pay for the administration, usually \$5 to \$10. If it is felt that the cost of the damage incurred should be borne by the hauler, the cost will probably be based on weight and distance moved. Still others feel that permit operations should yield revenue, in which cases the fees may be quite high.

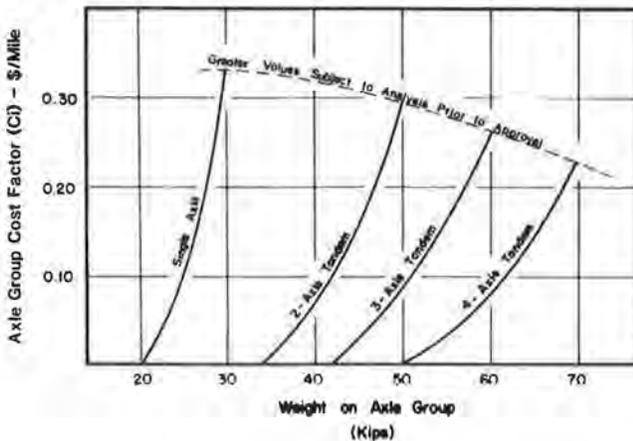
There is, however, a general consensus that permits should be issued only when (a) the load is not readily reduced, (b) there exists no other reasonable alternative mode of transportation that will not be excessively costly, (c) a reasonably large sector of the economy will benefit, or (d) damage to the pavement structure will not likely be any greater than any normally permitted moves.

Enforcement varies from almost nonexistent to very strict, as do the fines that are assessed. Legal weights vary widely from state to state, which causes problems with the interstate trucking industry. In some states, the legal load varies between Interstate and state highways, and this causes further confusion. Because of the confusion caused by such wide variation, every study reviewed recommended uniformity of legal weights and permit policies.

Rational Overweight-Vehicle Policy

Whenever an overweight vehicle passes over a roadway

Figure 9. Cost per axle mile for overweight-vehicle permits.



section, damage is done to the surface that exceeds the damage that has, in principle, been paid for in fuel and licensing taxes. If the owner of the vehicle is not charged for this excess damage, then he or she is in effect being subsidized by all the users of the roadway network. It seems reasonable, therefore, to base permit costs on this excess damage.

As was shown earlier, the average cost per EAL-mile within Alaska is approximately \$0.06. By using equivalency factors, the excess damage for any truck can be determined in equivalent EALs. By combining these two factors, the incremental cost to the state of an overweight axle group can be determined by the resulting equation:

$$c_i [(W_{1+i}/W_{2+i})^{4.79} - 1] k \tag{8}$$

where

- $c_i$  = incremental cost for a specified axle group per mile of travel,
- $W_1$  = weight of the specified axle group (kips),
- $W_2$  = legal weight of the axle group (kips), and
- $k$  = \$0.06 (cost/EAL-mile).

By summing the cost of all axle groups and multiplying times the distance, the permit fee can be determined as follows:

$$P_c = \sum_{i=1}^n (c_i d) + A_c \tag{9}$$

where

- $P_c$  = permit cost,
- $n$  = number of axle groups,
- $d$  = distance traveled (miles), and
- $A_c$  = administrative cost.

The administrative cost may include bridge structural analysis and, in some cases, even pavement structural analysis. Because there is an upper limit to the load a pavement structure can tolerate without exceeding its fatigue strength, it is suggested that axle groups that have an equivalency factor greater than 6 be referred to the maintenance engineer or other persons in authority for approval.

Figure 9 graphically shows Equation 9 for various axle groups. To illustrate the use of Figure 9, consider the following example.

A permit is requested for a 3-S2 commercial vehicle with the following axle loadings: front axle,

18 000 lb; drive axles, 47 000 lb; and trailer axles, 50 000 lb. The load will be moved from Anchorage to Fairbanks (360 miles). Determine the permit cost, assuming an administrative cost of \$25.00. From Figure 9, the cost per mile for each axle group is as follows:

| Axle Group    | Cost per Mile (\$) |
|---------------|--------------------|
| Front axle    | 0.00               |
| Drive axles   | 0.21               |
| Trailer axles | 0.30               |
| Total         | 0.51               |

From Equation 9,  $P_c = \$0.51 (360) + 25 = \$208.60$ .

This seems like a fairly high permit cost, but remember that, by using three-axle tandems, the permit can be avoided.

Summary

The overweight-vehicle policy suggested in this report adopts a "pay-as-you-go" philosophy. By using a current overlay cost of \$13/linear ft of two-lane highway and the deflection-life curves developed in Ontario, a cost of \$0.06/EAL-mile can be derived. By relating this cost to the weight-damage relations developed in the AASHO Road Test, a cost per mile permit cost can be computed. The resulting costs are shown in Figure 9.

Of course, if the system is to work, enforcement will be a key factor. For this reason, enforcement will need to be strengthened.

CONCLUSIONS

Seasonal Load Restrictions

1. A rational method of establishing load restrictions has been developed in this paper based on deflection data. It is recommended that Figure 4 be used in establishing load restrictions in Alaska.
2. Since extensive deflection data are required to determine the level of restriction, a deflection inventory is desirable. This inventory could also be used to predetermine problem areas to be monitored during the spring-thaw period to establish the time frame for load restrictions.
3. Monitoring of thaw depths can be used to determine when to begin deflection testing.
4. It has been shown that load restrictions are justified, since costs of damage to pavement will exceed the additional shipping costs during the restricted period. They should therefore be posted as long as required based on deflection testing.

Policy on Overweight-Vehicle Permits

1. Weight-damage relations dictate that overweight vehicles must be controlled to prevent unnecessary reductions in pavement life.
2. In order to recover the cost to the state for damage caused by overweight vehicles, it is recommended that permit fees be based on EAL-miles traveled in excess of legal loads plus a fixed cost for administration.
3. Whenever possible, additional axles should be required to avoid overweight vehicles.
4. For obvious reasons, no overweight-vehicle permits should be issued during the thaw-weakened period.

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## State-Level Pavement Monitoring Program

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A long-term pavement monitoring program consists of the identification of specific data elements to collect, their measurement (including equipment), the sampling frequency both over time and within a pavement network and project, the efficient storage and retrieval of large amounts of data, its analysis and evaluation, and the use of the data in daily policy-level decisions and project programming. This paper describes (a) data needs for state-level management, planning, and design functions; (b) temporary data-collection activities by various states; (c) sampling strategies and data-processing strategies; and (d) key issues and problems to be considered in planning a long-term monitoring program.

There are three levels of pavement data monitoring. They correspond to three principal activities in the process of managing a state pavement network. State program management is concerned with overall planning, budgeting, and the equitable distribution of funding to political subdivisions of the state. It also includes special case studies and research needs, such as for cost allocation and improvement of design procedures. State project programming is concerned with selecting specific projects for an annual maintenance and rehabilitation program as well as for development of a projected future work plan. Project design is concerned with selecting the proper materials, layer thicknesses, and treatments for a specific project. Because the latter activity occurs on a project-by-project basis, it is not usually considered as part of a state's long-term monitoring program and will not be considered further here.

Long-term pavement data monitoring serves several important functions in state program management:

1. It provides current information on pavement condition and documents performance history on all functional systems for administrative, planning, operational, and research purposes.
2. It provides data for the projection of future funding requirements for various possible scenarios, including (a) keeping the pavement network in its current condition, (b) improving the condition of pavements on selected road networks, (c) determining

the impact on network condition of a budget cut, and (d) keeping the pavement network condition stable in the event of changes in legal sizes and weights of trucks.

3. It provides accurate information to the legislature on current pavement conditions, costs, and accidents.

4. It provides information for the equitable allocation of funds to all subdivisions of the state weighted by the traffic, functional class, and occasionally the economic impact of specific road networks such as those used for hauling important products and commodities.

5. It provides information for a variety of special needs, such as studies of cost allocation or truck weight, requests from the public, state and national research projects, and data for use on the national level by the Federal Highway Administration (FHWA).

Long-term data monitoring assists in project programming in the following ways:

1. The determination of current condition and other information about specific sections of pavement to allow prioritization of and estimation of costs for required maintenance or rehabilitation and

2. The collection of condition and other data on a sufficient number of randomly sampled sections of pavement to provide the estimates and special studies that are required for state program management.

These multiple uses of a monitoring system require several types of information to be gathered and to be updated frequently. The data need to be gathered efficiently, by using sampling techniques both in time and along the length of various functional classes of highways. The purpose of sampling is to reduce to a minimum the effort that is required to collect the data. Although sampling is an essential element of a long-term monitoring program,

the method of sampling must be dictated by the purposes for which the data will be used (as discussed later in this paper). In all cases, the data must be collected by reliable methods, a large number of which are currently in use in a number of states. The data must be efficiently processed, analyzed, and stored so as to be easily accessible for the variety of purposes for which they will be used.

#### DATA NEEDS FOR STATE PROGRAM MANAGEMENT

In general, the following types of data will be required for state program management:

1. Definition of the highway system, including designations of the functional classes of highways, a division of the highway network into standard uniform sections, and a determination of the mileage, age, surface type, and pavement depth distributions within each functional class and other subdivisions (districts, counties, etc.);
2. Traffic and capacity information, including the distribution of average daily traffic, 18-kip equivalent single-axle loads (ESALs), and volume/capacity (V/C) ratios for each functional class of highway;
3. Climatic variables, including moisture and temperature data;
4. Surface condition data, primarily the distribution of skid number by functional class;
5. General accident rates from existing accident record systems;
6. Past histories on accident rates, inflation rates, cost indexes, and unit cost distributions of maintenance and rehabilitation; and
7. Pavement condition rating distributions by functional class, including riding quality [or present serviceability index (PSI)] and significant distress types (including composite distress index).

The distributions of data should include a mean, a standard deviation, and the type of distribution (normal, lognormal, Beta, Poisson, etc.). This statistical information can be determined by evaluation of results by a suitable sampling scheme.

Special studies may require that additional detailed information be collected on a selected sample of test sections. These special studies may include the development of damage models for cost-allocation needs, pavement design improvement, impact of quality control on pavement performance, and other research needs. The development of special relations using multiple regression analysis is discussed in a report by Rauhut and others (1).

#### DATA NEEDS FOR PROJECT PROGRAMMING

Most of the same categories of information must be gathered for the purpose of project programming. However, the data must be collected on specific sections of pavement rather than on a random sampling among sections. When this specific information is stored and retrieved in a data base management system, it should remain identified by the pavement section from which it came.

The types of data needed for project programming include the following:

1. Definitions of the network by uniform sections of similar design, which should be categorized by functional class and traffic level;
2. Traffic and capacity information;
3. Skid condition data;
4. Accident rate history;
5. Pavement condition, including riding quality (or PSI) and distress; and

6. Average annual pavement maintenance expenditure.

Data items 2-6 must be updated periodically. An overall summary of these data, their level of importance, and sampling frequency is given in Table 1. The frequency of updating varies from item to item and also among pavement sections. Factors affecting the frequency include rate of change in the item, allowable maximum interval between observations, and allowable minimum value.

#### SYNTHESIS OF CURRENT DATA-COLLECTION MONITORING PROGRAMS

##### Distress

Distress monitoring includes distress type, severity, and amount. Significant distress types for flexible pavements include rutting, alligator cracking, transverse and longitudinal cracking, block cracking, potholes, bleeding, raveling and weathering, and swells and depressions. Significant distress types for rigid pavements include: slab cracking, D-cracking and reactive aggregate deterioration, pumping, joint deterioration, and blowups. Several states have developed distress identification guidelines; however, a standard distress rating manual should be used throughout the states to achieve consistency. The FHWA Highway Pavement Distress Identification Manual (2) is recommended for this purpose.

Distress is currently monitored by several states for state-level program studies, project programming, and project design. The State of Washington performs distress condition surveys on a portion of every highway section once every two years. The distress data are then used to arrive at a structural rating (3). The State of California also monitors selected distress on each highway section once every two years (4). The entire length of highway is surveyed and the data are used to determine the extent of structural damage (percentage of shattered slabs and percentage of fatigue alligator cracking). Each section of highway in Ontario is rated at least once every three years, utilizing distress identification manuals but more frequently as rehabilitation becomes imminent (5,6).

The State of Florida procedure for flexible pavements is based on selecting and surveying a typical 100-ft-long by one-lane-wide section as a representative sample of the total pavement section evaluated (7). Only three distress types are recorded: rutting, cracking, and patching. For concrete pavements, a rating team measures and records pavement distresses while driving at a reduced speed. The rating team, however, is required to stop the vehicle at a representative location of the rated section and determine the summation of faulting for five consecutive slabs (7).

A comprehensive distress survey procedure and distress identification manual was developed under NCHRP Project 1-19 (8) for plain jointed, reinforced jointed, and continuously reinforced concrete pavements (similar to the FHWA Manual for concrete pavements). A survey crew of two or three persons can survey a 2-lane-mile section in 0.5-1 h, depending on the amount of distress. A few distress types (such as swells and depressions and shattered slabs) are recorded along the entire project, but most are sampled in 0.1-mile sections randomly located within every mile. The data are recorded on specially prepared sheets that are coded for immediate keypunching and entering into a computerized data base.

The U.S. Army Corps of Engineers procedure

Table 1. Summary of data that should be collected for programming and project prioritization.

| General Data Item            | Level of Importance | Sampling Frequency (years) | Relative Cost Level |
|------------------------------|---------------------|----------------------------|---------------------|
| Description of pavement      | Normal              | - <sup>a</sup>             | Low                 |
| Traffic volume               | Normal              | 2-3                        | Medium              |
| Traffic loadings             | Normal              | 2-3                        | High                |
| Skid resistance              | High                | 3-5                        | Medium              |
| Accident rates               | High                | 2-4                        | Medium              |
| Distress                     | High                | 1-4                        | Medium              |
| Roughness and serviceability | High                | 1-4                        | Medium              |
| Maintenance costs            | Normal              | 1-2                        | High                |
| Rehabilitation costs         | Normal              | 1-2                        | Low                 |

<sup>a</sup>Initial and after major work.

considers all distress types and uses a comprehensive distress identification manual (9). Each pavement section is divided into sample units (about 100-200 ft long). The number of sample units to be inspected in each section is a function of the objective of the survey and the desired level of reliability (10).

Distress has been traditionally measured manually by traveling slowly along the pavement. New photographic equipment now provides for rapid (40-mph) recording of all visible distress within a 12-ft-wide lane on a strip photo and also rut depth and cross slope. The automated photographic system has significant advantages over the manual surveys, including cost, speed, and safety.

#### Roughness and Serviceability Index

Roughness is defined as irregularities in the pavement surface that adversely affect ride quality, safety, and vehicle maintenance costs. The PSI is highly dependent on roughness. It is computed directly from roughness data by several states.

There are several different indicators of roughness, including (a) the roughness index (sum of vertical deviations of surface or car body), (b) wavelength-amplitude characteristics (actual profile measurement), (c) slope variance, (d) vertical acceleration, and (e) subjective rating by a panel.

Roughness is generally considered a very significant parameter for state-level program management and state-level project programming. It is also of value at the project design level in determining the most cost-effective repair method.

A number of mechanical and electronic devices are used by states to measure roughness. Long-term monitoring requires that the measurement equipment be stable over many years. Several of these devices, particularly the car ride meters, change in roughness output over time as the car suspension system changes. Either careful year-to-year calibration procedures must be developed, or a device that measures the actual profile must be used.

Roughness data in California are measured with a car ride meter of the Portland Cement Association (PCA) type in order to identify when pavement corrective work should be undertaken to improve rideability (4). Critical levels of roughness were determined by use of rating panels. Raters were asked to judge whether the pavement provided a satisfactory ride or was so rough that it should be improved. A critical ride score was selected to identify those locations that should be considered for ride quality improvements. The ride score is computed separately for sections of pavement approximately 1 mile in length (this may vary from as short as 0.1 mile to more than 1 mile) and for bridge approach ride quality (approximately 100 ft approaching and leaving bridge ends). Roughness measurements are taken on every section of highway

(essentially every mile) every two years.

Texas uses roughness to calculate a PSI. Mays ride meters are located in each district and periodically run on pavement sections. The Mays meters are brought back to headquarters often, where they are run over a set of test sections for calibration. The General Motors (GM) Profilometer is used to measure the roughness on the calibration test sections periodically to ensure that they do not change. Functional relations have been developed between the profile measured with the GM Profilometer and a large panel of raters. This approach is felt to be an excellent method of obtaining uniform roughness data over the long term.

Minnesota develops an annual calibration or correlation between the PCA-type ride meters and a large panel rating. A potential problem with the approach is that the panel ratings may vary from year to year.

#### Surface Skid Condition

Skid resistance is usually determined based on the friction coefficient between the vehicle tires and the pavement surface. Methods for measuring the friction coefficient include trailers with locked wheels, trailers with unlocked wheels making a yaw angle with the direction of travel, trailers with rolling wheels in the slip mode, portable field instruments, and laboratory equipment. The locked-wheel trailer, which complies with American Society for Testing and Materials (ASTM) standards, is the method most commonly used by the states. Values are measured in terms of skid numbers (SN), which are measured at 40 mph or converted to equivalent 40-mph values and designated SN(40).

The State of California is using the locked-wheel-trailer method and has established procedures for a continuing skid-testing program. The frequency of the skid tests is such that all Interstate highways are tested each year, other freeways and expressways are tested every two years, and all other state highways are tested every four years.

#### Deflection

Pavement response to loading (or deflection) is routinely used by many states to assess the structural adequacy of pavements. The following general deflection devices are in use by states (11): (a) static deflection (e.g., Benkelman beam), (b) vibratory steady-state force (e.g., Dynaflect, Roadrater), and (c) impulse (e.g., Falling Weight Deflectometer). Deflection results are most commonly used at the project design level and not often used at the state monitoring level. However, special studies for cost allocation or design verification may require deflection testing on selected sections over various seasons throughout the year.

### Distress Composite Index

A meaningful pavement condition index (PCI) can be obtained by combining the effect of the individual distress types into a composite index. The index provides several uses at all three levels: program management, project programming, and project design. These uses include overall pavement condition rating, determination of needed level of maintenance and rehabilitation, project prioritization, performance modeling, computation of benefits for various repair alternatives, and budget optimization. The advantage of a distress composite index is that it indirectly combines in one number the weighted effects of important pavement condition indicators such as roughness, skid, and structural integrity. Such an index has been developed in various forms and implemented by states and agencies, including Washington, Texas, Florida, the U.S. Air Force, and the U.S. Army (3,5-7,10,12). For example, the PCI developed by the U.S. Army Construction Engineering Research Laboratory (CERL) (10) provides that the distress and level of severity be carefully and accurately defined to reduce field subjectivity. The distress deduct values are developed so that the computed PCI represents the collective judgment of experienced pavement engineers. This adds to the meaning and usefulness of the index.

### Traffic

Four types of traffic data are required for long-term pavement monitoring:

1. Volume data for the 13 standard vehicle categories are typically provided by states as average annual daily traffic (AADT) and average annual daily commercial truck traffic (AADTT), including a breakdown into the basic truck categories (buses; two-axle, six-tire single unit; three-axle combinations, etc.). Typically, states sample the traffic stream annually at key locations along the highway network. These data are published as maps or in summary tables.

2. Lane distribution of trucks is required, but states do not measure or report this information (except as part of a research study). This is a very important data element for use in pavement deterioration studies and must be included.

3. With regard to axle weight distribution of trucks, states typically collect only a minimal amount of data on truck axle load (e.g., one 8-h day per year at weigh stations). These limited data are used to prepare the "W-4 tables". W-4 tables can normally be obtained for a given weigh station or over an average of all stations on a functional classification of highways (e.g., IR, IU) each year. There are typically so few weigh stations available that the axle-load distributions are averaged over an entire functional class of highways. Data from the W-4 tables can be used to compute the mean load distribution factor (or mean 18-kip ESAL per truck). All states submit the required data to FHWA each year. Much improvement in this procedure of estimating axle-load distributions is needed for the long-term monitoring system.

4. V/C ratios are determined by dividing the vehicle count by the capacity computed from the Highway Capacity Manual (12).

### Accident Rates

For long-term pavement monitoring, the main emphasis regarding accident rates is to identify locations that have high rates and determine whether pavement condition might be a cause. Most states have

well-developed accident reporting system outside the highway department.

Many states have computerized roadway information files that can be matched with reported accident files to develop numbers of reported accidents by numerous roadway and traffic characteristics. This will allow the development of generalized accident rates for both programming and project development. These computerized accident and highway files can also be used to determine numbers of reported accidents by year for specific highway sections. By sampling large numbers of sections that have different pavement characteristics and numbers of accidents, accident rates (by severity) can be related to pavement characteristics. Such statistical studies can be used to provide predictive relations for analyzing the benefits of alternative pavement strategies. In addition to general statistical analyses, computerized accident files that can be matched with highway files can also be used to develop accident histories on each specific section of highway covered by the files. These data on specific sections can be used as inputs in specific pavement programming-level decisions. Data can also be developed for several years on specific accident types such as wet-weather accidents, by severity. Because of the extra burden of incorporating accident reporting into a pavement monitoring system, only accident rates developed from existing accident record systems should be used.

### Maintenance and Rehabilitation Costs

The details of systems for recording and reporting maintenance cost vary considerably from state to state. In an NCHRP study reported in 1977 (13), 11 states were surveyed and the following general characteristics were found. All states use automated data storage and reporting of routine maintenance cost information, although none record the lane in which the maintenance was done or the type of pavement (flexible or rigid). States record maintenance activities either by milepost, highway segments of variable length, highway, or district. In order to be useful for project programming, recording by either milepost or highway segment is essential. Any of these levels of reporting are adequate for program management purposes, but even here the more detailed systems are preferable.

Estimates of maintenance costs must be derived from unit costs of the various maintenance activities, the number of which varies between about 40 and 500. The aggregated maintenance costs in 1977 ranged between \$1000 and \$4500/lane mile or between \$10 and \$50/vehicle/year. The approximate cost of the recording and reporting of maintenance costs was about 0.5 percent. Table 2 (13) gives more detail on each state surveyed.

The costs of major maintenance, rehabilitation, and other such activities can be obtained more directly from bid summaries or from specific project codes. In any case, average costs are desirable in order to make projected estimates of project and networkwide funding needs.

### SAMPLING STRATEGIES

It is essential to have a carefully planned strategy for data collection for each of the two state-level long-term monitoring purposes. The minimum amount of data that is necessary should be collected each year in order to reduce the amount of personnel, equipment, and other resources used in this effort.

When a long-term monitoring program is initiated, there is a need to obtain detailed project programming data on 100 percent of the entire highway

Table 2. Typical state maintenance recording systems in 1977.

| State        | No. of Maintenance Activities | Location Identified        | Maintenance Cost (\$) |                      | Percentage of Maintenance Budget for Recording and Reporting |
|--------------|-------------------------------|----------------------------|-----------------------|----------------------|--|
|              |                               |                            | Per Lane Mile         | Per Vehicle per Year |  |
| California   | 470                           | Milepost                   | 3400                  | 11.40                | 0.4  |
| Hawaii       | 36                            | District                   | 4550                  | 20.90                | —  |
| Illinois     | 49                            | Highway segment            | 2100                  | 13.30                | 0.6  |
| Louisiana    | 124                           | Highway segment            | 2400                  | 41.80                | 0.4  |
| Minnesota    | Many                          | Highway segment            | 1800                  | 21.60                | 0.5  |
| Nevada       | 72                            | Milepost                   | 1300                  | 35.50                | 0.2  |
| North Dakota | 63                            | Highway segment            | 950                   | 29.30                | 1.0  |
| Pennsylvania | 218                           | District                   | 2660                  | 29.50                | —  |
| Tennessee    | 71                            | Numbered highway in county | 1600                  | 15.40                | —  |
| Washington   | 373                           | Highway segment            | 2400                  | 16.90                | 0.3  |
| Wyoming      | 46                            | Milepost                   | 1160                  | 51.00                | 0.5  |

network within a reasonably short period of time, such as two years. This means that approximately one-third or one-half of the network will be sampled in any one year. Once the initial condition of each section of the network is known, future sampling rates can be reduced based partly on numerical studies of sampling patterns and partly on experience, as discussed later. Under any circumstances, the level of effort that can be maintained over the long term will necessarily be controlled by the level of funding and the number of trained personnel available for the survey.

Once the initial condition data are collected, the sampling strategies for the two purposes become different. In general, the number of sections from which state program management data are drawn is considerably smaller than the number of sections needed for project programming. However, the state program management data must be collected in strict accordance with a carefully executed random, stratified, or cluster sampling plan since every effort must be expended to eliminate bias in these samples of data.

Data-Collection Strategies for Program Management

Data collection for program management purposes must be done systematically in accordance with a sampling plan. Careful attention should be applied to the design of the sampling process in order to gain an optimal compromise between the cost of collecting the data and the information gained from the survey.

A brief description and example of some of the available methods of survey sampling follow:

1. Simple random sampling provides that every sample has an equal probability of being chosen from a population. For example, if all highways in a given geographic area were divided into equal lengths (segments), then each highway segment would have an equal choice of being chosen for the required sample size.

2. Stratified random sampling is the sampling process whereby a population is divided into strata and then random samples are obtained within the described strata. For example, if a given state is divided into a number of highway department districts and data estimates were required for each district, then each district could be considered a stratum and individual highway segments could be randomly selected within each district.

3. In one-stage cluster sampling, elements within a population are first grouped together and then the

elements are randomly sampled. For example, if data estimates are required for a state, counties could be randomly selected throughout the state. Within each selected county, all highway segments would be sampled. The pavement segments surveyed are considered to be "clustered" within the selected counties.

4. Multistage cluster sampling (multistage sampling) is similar to one-stage cluster sampling but takes the process further. Multistage clustering allows for larger areas to be clustered together and then randomly sampled. The elements within these clusters are also randomly sampled. As in the previous example, if data estimates are required for a given state, counties within a district can be randomly selected and within those selected counties pavement segments may be randomly selected. This would constitute a two-stage cluster sample. A three-stage sample would involve randomly selecting highway department districts within a state, then counties within the selected districts, then pavement segments within the selected counties.

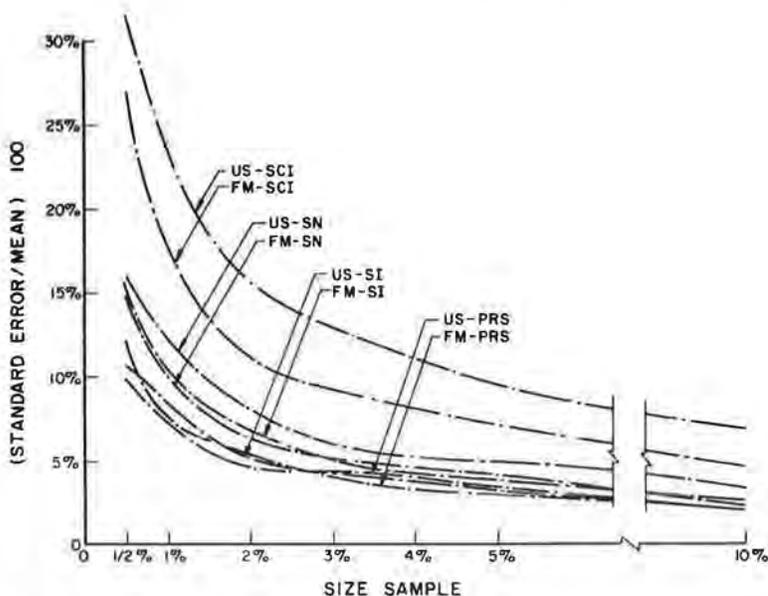
5. In systematic random sampling, every kth element of a set of data is sampled. The first element is selected at random between 1 and k. For example, data estimates are required for a state that has 100 counties. It is determined that 10 counties should be surveyed. The interval k is determined as  $100/10 = 10$ . The first county is selected at random between 1 and 10—say, 3. The counties to be surveyed are therefore 3, 13, 23, 33, ..., 93. Within each county selected, all highway segments would be sampled in the data-collection effort.

In addition, combinations of these five methods can be created.

Each sampling survey method has its own formulas for estimating means and standard deviations of the entire population or of specific functional classes of highways. The formulas can be found in standard reference books on sampling techniques (14,15). It is advisable to perform a study of each data type by using actual data to determine the precision of different sampling rates before selecting a specific sampling rate.

There are two approaches to determining the sample size. The first draws sample data from a small number of pavement segments in order to obtain a reliable estimate of the mean and standard deviation of each functional class of highway. Changes in the condition of these classes may be predicted by using regression models of distress and performance and a knowledge of the distribution of pave-

Figure 1. Coefficient of sample variation versus sample size for Texas highway district 21 (1975 data).



ment age and traffic within each class. This approach requires sample sizes on the order of 2-3 percent, as can be observed in Figure 1 (16), which is based on a detailed numerical study on PSI, SN, composite distress score or pavement rating score (PRS), and Dynaflect measurements (surface curvature index). As Figure 1 shows, virtually no increase in precision, as measured by the ratio of standard error to mean, can be expected with sample sizes greater than about 2 or 3 percent. The exception to this appears to be the Dynaflect data, which seem to reach a plateau at a sampling rate of about 5-10 percent.

The second method attempts to detect changes from one year to the next strictly by sampling without the use of regression equations as in the first method. In the second approach, the size of the sample is determined by the percentage change in pavement condition that it is desirable to detect and the level of confidence that is required of the result. The Highway Performance Monitoring System (HPMS) suggests that a 10 percent change in the mean value should be detectable at an 80 or 90 percent level of confidence (17). A simple formula for estimating the number of sections required is given by the following equations:

$$\eta_0 = 0.5z^2 / (p_1 - p_2)^2 \tag{1}$$

$$\eta = \eta_0 / [1 + (\eta_0/N)] \tag{2}$$

where

- $\eta$  = number of samples required to detect a change in proportions ( $p_1 - p_2$ ) in a functional class of pavement;
- $p_1$  = at time period  $t_1$ , the percentage of the total mileage of a functional class of pavement that has distress greater than an acceptable level, riding quality lower than a specified minimum, or SN lower than a minimum level;
- $p_2$  = same as above for time period  $t_2$ ;
- $(p_1 - p_2)$  = percentage change that can be detected;
- $N$  = total number of sections of pavement in a functional class; and

$p$  = normal variable for a specific level of confidence, as given below:

| Level of Confidence (%) | $z$  |
|-------------------------|------|
| 50                      | 0.00 |
| 70                      | 1.40 |
| 80                      | 1.29 |
| 90                      | 1.65 |
| 95                      | 1.96 |

For example, for a 10 percent change in the mean value of distress with 90 percent confidence,

$$\eta_0 = (0.5)(1.65)^2 / (0.10)^2 \tag{3}$$

Since  $z = 1.65$  for a 90 percent confidence level, then  $\eta_0 = 136$ . For a functional class that contains 500 sections of pavement, the sample size is  $\eta = 136 / [1 + (136/500)] = 107$ . If an 80 percent confidence level is acceptable, the sample size drops to 71 sections.

The more important functional classes of pavement, such as urban principal arterials, should require a 10 percent change to be detected with a 90 percent level of confidence. Other functional classes should require only an 80 percent level of confidence. This rule of thumb applies to all types of required data except climatic and traffic data.

Climatic data should be sampled from among the weather stations in each climatic region in a state but from no fewer than three stations. The process of traffic data collection should recognize the fact that the coefficient of variation of traffic counts varies inversely with the traffic volume. The HPMS system provides an approximate method of allowing for this variation of traffic variance with traffic volume, and that method is suggested as a practical means of determining the sample size of the sections where traffic data will be collected.

To summarize the two sampling methods described above, the first is concerned with making an accurate estimate of means and standard deviations, and the second is concerned with collecting sufficient data to detect a change in the mean value of a specified percentage. The first method only requires sample sizes of around 2-3 percent, whereas the second method, the HPMS method, may require

statewide sampling rates of around 5-10 percent (excluding locals). The first method requires the use of regression equations to predict future changes in pavement condition, but the effort required to develop such models may be recovered quickly in savings of personnel over the second method.

Some numerical experimentation with the actual data will probably result in a reduction in the total number of sections where data must be collected. The pavement condition data collected for the purpose of programming specific projects should provide more than enough data each year to make reasonable assessments of the changes that occur in each of the functional classes of pavement for the purposes of statewide program planning and fund allocation.

#### Data-Collection Strategies for Project Programming

Programming of individual projects requires accurate knowledge of the current condition of every pavement section in the entire network. In this case, sampling takes on a different meaning. The data-collection effort for project programming purposes must satisfy three requirements:

1. It must provide data for use in state program management.
2. It must systematically update the initial condition data collected at the start of the long-term monitoring effort.
3. It must collect current condition data on specific sections of pavement that are approaching the end of their satisfactory service life.

The first requirement must be met by sampling in accordance with a carefully planned sampling plan. The second requirement is one that will ensure that the information on each section of pavement is reasonably current. This means that every section should be inspected every two to five years, even though it remains in good condition. The third requirement is to permit prioritization of pavement maintenance or rehabilitation projects. The pavement sections that need to be inspected to satisfy the first two requirements can be designated on a map each year.

An approximate annual inspection schedule to meet the third requirement can be generated by using regression models of pavement distress, performance, and skid and projecting them forward in time to the point where unacceptable levels of these pavement condition indicators are predicted. Because of the scatter that is inherent in regression equations, it is considered wise to record condition data on each section of road in the network every two to five years, regardless of condition. This will permit more accurate and useful annual inspection schedules and can help in planning the manpower effort that will be required in the annual condition survey.

Pavement sections at locations where unusual numbers of accidents have occurred should be inspected as should those special sections of the network that are affected by a sudden increase of traffic or by a haul of economically important products and commodities.

Obviously, the percentage of the highway network that needs to be inspected each year will depend on the overall condition of the pavements in the network and the rate of deterioration. For a network in good condition, data may need to be recorded on no more than 20 percent of the sections. The percentage will increase as the overall condition of the network becomes poorer.

#### DATA PROCESSING, STORAGE, AND ACCESSIBILITY

The data that are gathered will have multiple uses in both state program management and project programming. Because of this, the data must be stored so that they are readily accessible in a variety of forms. In general, greater accessibility requires automation and a greater capital investment. Thus, the degree of accessibility should depend primarily on how frequently the data must be used. Data that do not change rapidly or are not used often may be stored in less accessible forms.

The following methods of storing data are listed in increasing order of accessibility:

1. Decentralized manual files,
2. Centralized manual files,
3. Punched cards,
4. Magnetic tape, and
5. Computerized data base.

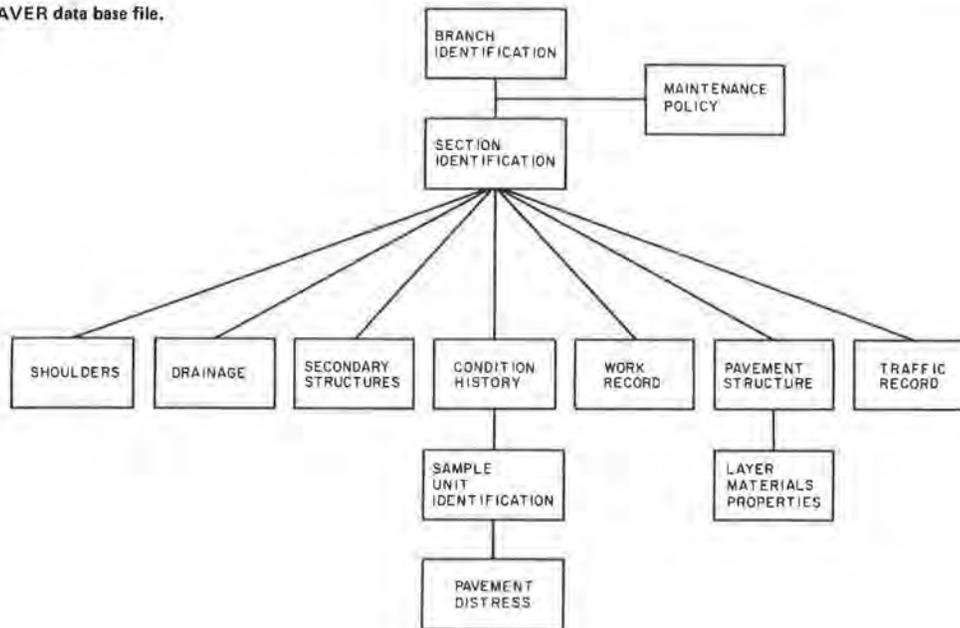
Because a substantial amount of financing and experience are required to build up and debug a computerized data base, it is best to regard the above list as the order in which a data storage system is developed. It must be realized, however, that the processing of large amounts of data on the state level as well as the accessing of the data by other agencies essentially require a computerized data base management system.

NCHRP Project 1-19, Concrete Pavement Evaluation System (COPEs) (8), used a hierarchical data base management system called SIR (17). The variables in the data base are grouped in records (historical data, distress, traffic, etc.), where each record owns many other records in a top-down or tree-like structure. Data are recorded on specially prepared sheets that can be directly keypunched into a computer file and then entered into the data base. Data retrieval and analysis are easily accomplished for SIR in either batch mode or interactive mode by using a remote computer terminal. The user sitting at a computer terminal can input and execute a set of SIR commands, retrieve data files in any desired format, conduct many kinds of analyses on the data by using statistical packages such as the Statistical Package for the Social Sciences (18), print out results, etc. Automated reports, such as a summary of pavement distress in a given district, can also be developed.

The PAVER system developed by CERL is one of the best developed and implemented pavement data base management systems (10,15). The PAVER system includes a sophisticated data base manager called System 2000, specially developed user interactive input-update-change routines, and specially developed COBOL and FORTRAN programs interfaced with the data base. The PAVER data structure is shown in Figure 2. The system currently offers more than 12 preformatted reports with practically unlimited sort items.

The Arizona Department of Transportation (DOT) recently developed a computerized pavement data base system called the Information Management System (IMS) (19). The data base contains eight segments: (a) route segment (location), (b) synthesized data, (c) common data (descriptive design information), (d) skid data (Mu Meter), (e) raw ride data (Mays Meter), (f) raw Dynaflect data (deflection), (g) raw cracking data, and (h) surface history (layers, date of construction, thickness, and type). The IMS file is hierarchical in structure. Remote computer terminals can be used to execute commands and obtain results. The data base is in place and being used. The Arizona DOT estimates that the computerization of the data base has resulted in large savings of manpower.

Figure 2. Structure of PAVER data base file.



#### SUMMARY OF KEY ISSUES AND PROBLEMS

Pavements are an essential part of the highway transportation system. The long-term monitoring of pavements at the state level is essential to the efficient management of this huge investment in public funds. This paper presents only a brief discussion of many issues and problems involved in the long-term monitoring of pavements at the state level. Many states have recognized its importance and are currently in the process of developing pavement management systems, which require a formalized monitoring system. Some of the key issues and benefits involved in a state-level pavement monitoring program are summarized as follows:

1. A long-term pavement monitoring program consists of the identification of specific data elements to collect, their measurement (including equipment), the sampling frequency both over time and within a network and project, the efficient storage and retrieval of large amounts of data, data analysis and evaluation, and finally the use of the data in day-to-day policy-level decision and project programming.

2. The long-term pavement monitoring program is a large and costly effort that will require a firm commitment for continued support on the part of the highest level of management in the state. Many data elements are currently being collected, and they must be coordinated and brought together into a common data base (or at least compatible data bases).

3. The data-collection effort can and must be reduced to a minimum by the carefully planned use of sampling surveys and regression equations. The program should be closely coordinated with the federal HPMS.

4. Long-term data collection and evaluation will permit better management of the overall state highway program. In-service information is needed for operational decisions concerning the many design, construction, and maintenance-rehabilitation aspects of a pavement system as well as for use in making reasonably accurate pavement performance predictions for planning, budgeting, and resource allocation purposes.

5. There are numerous special studies for which a

long-term pavement monitoring program will provide the required data, including cost-allocation studies, analyses of pavement deterioration to assess the relative damage attributable to traffic and environmental factors, and determination of the optimum time to rehabilitate.

6. Long-term monitoring data permit the use of optimization techniques to ensure that the most cost-effective funding and timing strategies are used on each section of pavement in the highway network.

7. Finally, a difficult question needs to be considered for which no definitive answer is now available: Can the expenditure of funds on manpower, equipment, and computer hardware and software for a long-term pavement monitoring program be recovered from real savings that are realized in a more cost-effective use of available funds? At the present time, intuition and some experience indicate that such a system will pay for itself in a reasonably short time, once it is adopted on a consistent statewide basis. Nevertheless, this question should be considered by top management in each state before the state embarks on the task of developing a state-level long-term monitoring program. It should be asked again once the program has been in operation long enough to show some tangible benefit. It is certain that a long-term pavement monitoring program can pay for itself only if it is integrated into an overall statewide pavement management process.

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## Data Requirements for Long-Term Monitoring of Pavements as a Basis for Development of Multiple Regression Relations

J. BRENT RAUHUT, MICHAEL I. DARTER, AND ROBERT L. LYTTON

A discussion is presented concerning how data from a broad, long-term pavement monitoring study can be used to provide improved models for predicting damage to pavements, maintenance requirements, costs, etc., as functions of significant variables defining the pavement structure, its environment, and the traffic loadings imposed on it. Specific items of data are recommended for collection, and sampling techniques and sample sizes are discussed. Special considerations related to pilot studies or other studies of limited size are discussed, and specific recommendations are offered. Results are presented for a study that had the following goals: (a) to identify distress types that either generate maintenance or cause loss of performance or safety; (b) to identify environmental and traffic factors, material and geometric properties, and other properties and conditions of a pavement structure that significantly affect performance and distress; (c) to describe multiple regression techniques for developing empirical relations and the characteristics of the data banks required for successful models; (d) to develop recommended approaches to establishing sample sizes for long-term pavement monitoring studies; and (e) to develop recommendations for a planned pilot study.

The Federal Highway Administration (FHWA) and various state departments of transportation (DOTs) are

considering the implementation of long-term pavement monitoring studies to support pavement management system data requirements and to produce data bases adequate for developing needed empirical relations. The relations needed include distress and performance prediction models and maintenance cost models for use in design, programming funds, project prioritization, and cost allocation. An initial pilot study is currently being implemented by FHWA in seven states. If this pilot study shows promise, funding for a much broader study may later be considered. The discussions that follow are generally in terms of comprehensive national or single-state long-term pavement monitoring studies, but compromises and recommendations specific to pilot studies or other smaller studies are also included.

The purposes of this paper are

1. To explore various approaches to long-term pavement monitoring;

2. To define the data needed for developing relations between (a) pavement condition, (b) axle-load weights and types and the number of each, (c) maintenance, (d) major repair, (e) environment, (f) construction costs, and (g) costs to maintain functional pavements;

3. To recommend specific data for collection to support development of multiple regression equations when long-term pavement monitoring studies are implemented; and

4. To make recommendations for planning and implementing limited pilot studies.

We have applied the experience gained in an ongoing FHWA research study, "Damage Functions for Cost Allocation" (for which we are the principal investigators), to the definition of the data needs that appear in this paper.

#### DEVELOPMENT OF RELATIONS AMONG PAVEMENT DISTRESSES, TRAFFIC LOADINGS, ENVIRONMENTAL FACTORS, AND OTHER IMPORTANT PARAMETERS

The common procedure for defining the relations among a group of variables for which ample data exist is a statistical analysis called linear multiple regression. Nonlinear multiple regression techniques may also be used if the linear techniques do not produce a model of sufficient accuracy. Although the use of multiple regression techniques is commonplace, their success depends heavily on the adequacy of the data base, consideration of the significant variables, and the functional form of the models used in the multiple regression. A succinct, but very thorough, discussion of the requirements for reliable predictive models may be found in a paper by Darter (1).

One purpose of proposed long-term monitoring of pavements is to produce adequate data bases to support the development of mathematical equations or models that explain the relations among the significant parameters identified in the introduction. One purpose of this paper is to identify significant variables that should be included in such data bases and to suggest means of ensuring that the data bases themselves are adequate. The significant variables are discussed in the next section of this paper. Once they have been selected, it will be critically important to ensure the adequacy of their measurement during the long-term monitoring process.

The data base that results from a long-term monitoring program must have the following characteristics if it is to provide reasonably accurate models for the relations desired:

1. It must include data for all variables that are significant to predictions of distress or performance, the maintenance or rehabilitation requirements that they generate, and the costs for that maintenance and/or rehabilitation. These variables will generally include (a) those required to define the geometry of a pavement section, (b) the engineering properties of the materials within the pavement structure, (c) the traffic and axle-load distribution imposed on the pavement, (d) the environmental conditions in which the pavement exists, (e) the time since initial construction and significant rehabilitation (such as overlays), (f) maintenance and rehabilitation histories, including identification of the distress or other parameters that generated the maintenance or rehabilitation, and (g) costs.

2. The selection of states to participate in national studies should be at least partly based on obtaining a representative sampling of the various environmental and geographic regions. The selection

of test sections should include a representative sampling of (a) urban and rural areas, (b) highway functional classes, (c) traffic levels, (d) types of pavements, and (e) distribution of ages since construction or last major overlay. However, more limited pilot studies may only practically include (a) rural areas, (b) traffic levels, and (c) typical flexible and rigid pavements.

3. The data collected should be reliable and uniform from state to state or from test section to test section. There would be some advantage to the selection of test sections for which historical data have been obtained in the past, but care must be taken before this information is used in a data base to ensure that it is consistent with the long-term measurements that are to be undertaken. Reliability of the new data can be promoted by (a) use of carefully developed procedures described in sufficient detail that individual biases are hard to introduce, (b) sufficient training of survey crews to gain consistency in data gathering, and (c) maintaining equipment and keeping it calibrated (for the same reason).

4. Sufficient data should be gathered to satisfactorily "explain" the relations to be developed from these statistical analyses. In determining the sufficiency of the data, one must consider the numbers of test sections, the lengths of pavements to be included in the test sections, and the numbers of measurements to be made within these lengths ("sample sizes" are discussed later in this paper). Darter (1) has also recommended some replicate data cases (pavements of identical construction, traffic, and climate) to gain an indication of "pure error", which is taken to be a combination of random variations and repeatable errors in test equipment or observation.

A number of states have accumulated substantial data banks in pursuit of pavement management systems or improvements in programming and in project planning and design. Such data banks may offer valuable opportunities (especially if data collection is continued for a sufficient length of time) for multiple regressions to develop meaningful models within the inference space provided. That is, they apply strictly to that state or general environment only and to the range of variables included in the collection effort. A data base that includes more than 300 flexible test sections in Texas has been used to develop distress and performance predictive equations, as have others for rigid pavements (2).

The following problems are common to available data bases:

1. Historical data are frequently not available over a sufficient time period or have been collected sporadically.

2. Significant data are often omitted.

3. Apparently identical items of data are recorded in totally different units or are defined differently. Examples of this include roughness measurements made with grossly varying procedures and equipment; cracking that may be measured in (a) square feet or square yards cracked in some area or length interval, (b) lineal feet of cracks, (c) percentage of total cracked, (d) percentage of slabs cracked, etc., and variation in definitions of rutting and in the length of straightedge used to measure it.

4. Traffic or loading data are inadequate.

5. Definitions of present serviceability index (PSI) differ. Among the methods used by different states are the American Association of State Highway and Transportation Officials (AASHTO) relation, modified AASHTO relations, rating panels, simple

equipment measurements correlated through the General Motors Profilometer to rating panels, and separate regression equations.

#### DATA NEEDS FOR DEVELOPING RELATIONS

There are two broad categories of data to be collected during long-term pavement monitoring studies. The first is basic inventory data, which includes those items that will remain constant over the monitoring period. The second is monitoring data, which includes those items that will change with time and will require periodic measurements or updating during the monitoring period.

The basic inventory data include those data necessary to (a) identify the test section, (b) describe the geometric details of its construction and the material properties of its structural constituents, (c) describe the environment in which the pavement test section exists, (d) identify the accumulated traffic and axle-load data prior to the long-term monitoring effort, and (e) identify construction costs and costs of subsequent maintenance and repair prior to the long-term monitoring effort. All of these data should remain constant throughout the monitoring period unless the pavement is resurfaced or rehabilitated during that period. If the pavement is resurfaced or rehabilitated, the test section becomes for practical purposes a new pavement structure with new surface conditions. The basic inventory data must therefore be revised to describe these new conditions while the original data are retained for reference and long-term cost analyses.

The monitoring data include distress and serviceability measurements, traffic and axle-load data, results of deflection testing, pavement maintenance costs, resurfacing costs, and restoration and rehabilitation costs during the monitoring year. These data are to be collected on an annual or other periodic basis to provide a historical data base for developing relations among distress, performance, traffic and axle loads, age, maintenance costs, and repair costs.

As discussed previously, these data must be both sufficient and reliable if satisfactory relations are to be obtained. In addition, special consideration should be given to variables that are already used as a basis for decisions by state highway agencies, providing that these variables offer sufficient statistical significance. As a further practical consideration, the number of items of data to be collected should be limited to that necessary; data that require an unusual amount of work effort or sophisticated testing should be avoided wherever possible. Specific data needs and the possibilities for limiting the items of data to be collected are discussed later in this paper.

#### Data That Have Significance in Multiple Regression Relations

The first logical step in the selection of items of data to be included in the proposed long-term monitoring program is to develop an organized list of all variables that could reasonably be expected to have statistical significance in the development of the multiple regression relations. Three such lists have been developed and appear below.

The following items of inventory data are used for test-section identification and/or have statistical significance for multiple regression relations:

1. Test-section identification--Highway number, urban or rural, lanes included, functional class, and location of test section;

2. Geometric details and general information--Width of highway, number of lanes, thickness of layers, years when overlays or reconstruction occurred, identification of materials used in overlay or reconstruction, adequacy of drainage, underdrains provided, extent and severity of rigid slab cracking prior to overlay, width of shoulders, year originally constructed, identification of layer materials, overlay thicknesses (or final layer thicknesses after reconstruction), year and details of roadway widened, joint spacing, dowel bar diameter, type of load transfer (aggregate interlock or dowels), and dowel bar spacing;

3. Environmental data--General type of environment (dry-freeze, wet-no freeze, etc.), number of freeze-thaw cycles per year, highest mean monthly temperature, lowest mean monthly temperature, Thornthwaite moisture index, lowest mean solar radiation, highest mean solar radiation, annual precipitation, freeze index, ~~cloud cover, and wind speed~~;

4. Accumulated traffic and axle-load data prior to long-term monitoring effort--Mean average annual daily traffic (AADT) for prior years, accumulated 18-kip equivalent single-axle load (ESAL) (AASHTO equivalencies), weighted mean of percentage trucks for prior years, accumulated number and distribution of tandem axles, and accumulated number and distribution of single axles;

5. Material properties--(a) For subgrade soil: soil type and classification, percentage passing no. 200 sieve, ~~moisture content (inner wheel path, sample in summer)~~, ~~modulus of subgrade reaction~~, plasticity index, dry density, ~~resilient modulus~~, and California bearing ratio (estimate from other data if not available); (b) for base and subbase layers (unbound): soil type and classification, ~~moisture content (inner wheel path, sample in summer)~~, ~~resilient modulus~~, dry density, percentage modified AASHTO compaction, and percentage binder (passing no. 40 sieve); (c) for base and subbase layers (stabilized): type of treatment (cement, lime, etc.), untreated soil type and classification, dry density, ~~resilient modulus~~, percentage of stabilizing agent, and percentage modified AASHTO compaction; (d) for asphalt concrete layers: asphalt grade, asphalt content, viscosity of asphalt, ~~temperature susceptibility of asphalt~~, original stability, ~~fatigue life potential~~, penetration of asphalt (at time of basic inventory data collection), initial air voids, type of coarse aggregate, ~~polish of surface aggregate~~, ~~permanent deformation potential~~, ~~Lottman tensile strength ratio~~, and dynamic modulus; (e) for rigid layers: ~~modulus of elasticity~~, percentage of steel in longitudinal direction, modulus of rupture (compute from compressive strength if not available), ~~thermal coefficient of concrete~~, and type of coarse aggregate; and

6. Construction costs prior to long-term monitoring effort--Cost of initial construction, cost of each past overlay, cost of each restoration or rehabilitation project, and accumulated pavement maintenance costs (if available separated from routine maintenance).

The following items of monitoring data have statistical significance for multiple regression relations:

1. Distress and performance measurements--(a) For flexible pavements (with or without overlays): alligator cracking (fatigue), rut depth, roughness, raveling, lane-shoulder separation, low-temperature transverse or longitudinal cracking, low-temperature block cracking, skid resistance (to monitor reductions), and flushing; (b) for rigid pavements: slab cracking, D-cracking, joint faulting, pumping, lane-

shoulder separation, skid resistance (to monitor reductions), roughness, blow-ups, and deterioration of transverse joints; and (c) for rigid pavements with flexible overlays: reflection cracking, rut depth, potholes in overlays, raveling, skid resistance (to monitor reductions), roughness, flushing, and lane-shoulder separation;

2. Traffic and axle loads--AADT, number and distribution of single-axle loads, number and distribution of tandem axle loads, 18-kip ESAL for year, accumulated 18-kip ESAL, percentage of trucks, and truck lane distribution;

3. Results of deflection testing--Mean maximum deflection under load, basin parameters, and coefficient of variation of maximum deflection; and

4. Pavement maintenance costs per square yard of test section (exclusive of routine maintenance such as mowing, salting, snow removal, etc.).

The following additional data items should be collected if resurfacing, restoration, or rehabilitation occur during the monitoring year:

1. Cost of overlay per square yard;

2. Description of overlay for addition to basic inventory data--Thickness of overlay, material in overlay, asphalt grade, viscosity and penetration of asphalt, type of coarse aggregate, ~~temperature susceptibility of asphalt, Lottman tensile strength ratio~~, cost of overlay (per square yard), asphalt content, stability of mix, initial air voids, initial skid number, ~~fatigue life potential, permanent deformation potential, and polish of surface aggregates~~;

3. Cost of restoration or rehabilitation per square yard;

4. Description of pavement structure resulting from restoration or rehabilitation--Identification of layer materials, joint spacing, thicknesses of layers, dowel bar diameter, and width of joint at dowel level; and

5. Material properties--Items 5b, c, and d in the first list above for material properties of new or revised layers above the subgrade level.

The selection of the items of data that appear in these listings (including those marked through) was based partly on the experience of the project staff and partly on information reported elsewhere (2-5). The experience of the project staff includes the results of multiple regression model developments by Lytton, Darter, and others that are quite similar to those proposed. These lists are intended to identify the entire range of variables that would reasonably be considered for inclusion in the long-term monitoring project. They in effect represent a "shopping list" from which a more limited set of variables may be selected as necessary to the generation of the data bank required.

In general, the variables in the first listing (inventory data), with the exception of material properties, will not be too difficult to obtain and must only be obtained once. However, some may not be absolutely required and may be omitted. The material properties listed include variables that would be duplicative for explaining relations through multiple regression analysis. For example, the fatigue life potential of an asphalt concrete is largely explained by asphalt content, initial air voids, and dynamic modulus. Both permanent deformation potential and original stability explain the tendency of an asphalt concrete to rut. It should be possible to eliminate some data where other data are sufficient. This is explored in another section of this paper.

#### Considerations for Limiting Data-Collection Requirements

The critical criterion for elimination of items of data from the monitoring study is the effect of their elimination on the sufficiency of the data bank or statistical base. It appears that all or most of the data that may be seriously considered for elimination primarily affect the development of relations between the occurrence of distress and other parameters. If this is so, decisions as to data for collection may be made on the basis of their significance to the prediction of distresses. Each of the authors has worked for many years toward development of relations between distresses and significant independent variables and have applied this experience in evaluating the effects of data elimination.

Table 1 lists as an example 13 of the 46 significant distresses and performance measures considered for flexible pavements and flexible pavements with flexible overlays and also the variables on which occurrence or prediction of these distresses depends. The intent in assigning these priorities was to recognize significance on the assumption that other data that explained much the same variance had not first been introduced into the multiple regression. Similar tables were also developed and studied for various types of rigid pavements.

It should be recognized that significance in the statistical sense is somewhat a function of the order in which the data are introduced into the regression analysis. For instance, permanent deformation potential would not generally be as significant to rutting distress if the stability of an asphalt concrete mix had already been included (due to expected correlation between permanent deformation potential and stability). Layer stiffnesses and other material properties would generally have only limited significance if they followed deflection measurements in a multiple regression analysis. Because of this, it is not enough just to decide that items with a priority of 3 may be eliminated and that those with a priority of 1 must be retained. Although data items with a priority of 3 for all distresses can probably be eliminated, some with a priority of 1 may also if other data explain similar variation (or correlate highly).

Since there may also be several different combinations of variables that may each adequately explain the variation in a relation, some discretion (and a lot of experience) must be applied in the selection of the variable combinations to retain and the variables to be eliminated. For instance, the test programs used to develop the fatigue life potential of asphalt concrete mixes are extremely sophisticated and costly. Therefore, data for other variables that are more common and easily obtained would be favored if they adequately explain the nature of the asphalt concrete mix and its effect on the occurrence of fatigue in pavements.

#### Data Recommended for Collection

The considerations discussed above have been used to reduce significantly the recommended data requirements for long-term pavement monitoring studies. The items marked through in the list of inventory data are those data not considered essential, and those items not marked through are the inventory data recommended for collection. None of the monitoring data listed were selected for elimination, but some data items can be eliminated from the list of additional data and these have also been marked through. It should be noted that those items of data eliminated from the third list are in all cases

Table 1. Significant variables and their relative importance to significant distresses and PSI for flexible pavements and flexible pavements with flexible overlays.

| Significant Variable        | Significant Distress |                         |           |                          |   | Raveling | Flushing | Reflection Cracking | PSI |
|-----------------------------|----------------------|-------------------------|-----------|--------------------------|---|----------|----------|---------------------|-----|
|                             | Fatigue Cracking     | Reduced Skid Resistance | Rut Depth | Low-Temperature Cracking | Roughness due to Subgrade Volume Change |          |          |                     |     |
| Layer thicknesses           |                      |                         |           |                          |   |          |          |                     |     |
| AC overlay                  | 1                    |                         | 1         | 1                        | 3                                       |          |          | 1                   | 1   |
| AC surface                  | 1                    |                         | 1         | 1                        | 3                                       |          |          | 2                   | 1   |
| Base                        | 1                    |                         | 1         |                          |   |          |          |                     | 1   |
| Subbase                     | 1                    |                         | 1         |                          |   |          |          |                     | 1   |
| Layer stiffnesses           |                      |                         |           |                          |   |          |          |                     |     |
| AC overlay with temperature | 1                    |                         | 1         |                          |   | 3        |          |                     | 1   |
| AC surface with temperature | 1                    |                         | 1         |                          |   | 3        |          |                     | 1   |
| Base                        | 2                    |                         | 1         |                          |   |          |          |                     | 1   |
| Subbase                     | 2                    |                         | 1         |                          |   |          |          |                     | 1   |
| Subgrade                    | 2                    |                         | 1         |                          |   |          |          |                     | 1   |
| 18-kip ESAL                 | 1                    | 3                       | 1         |                          | 3                                       | 3        | 2        | 2                   | 1   |
| Time since construction     | 3                    | 3                       | 3         | 1                        |   | 3        |          |                     | 2   |
| Time since overlay          |                      |                         |           | 1                        |   |          |          | 1                   | 2   |
| Annual precipitation        |                      |                         | 2         |                          | 1                                       | 1        |          |                     | 1   |

Note: 1 = essential data; 2 = moderately significant data; 3 = data have significance but explain only a limited amount of variance.

items of data eliminated from the first list also.

Whereas the resilient moduli of all other materials were eliminated, the dynamic modulus for the asphalt concrete layers was retained because this is a very important parameter that greatly affects the rutting potential and fatigue life of the pavement. In addition, it is not particularly difficult to obtain cores of this material when the inventory data are accumulated and to run dynamic indirect tensile tests on these cores to obtain the dynamic modulus. These tests should be run at a minimum of three temperatures that represent approximately the range anticipated. Tests should generally be conducted on the same specimens to limit variability, and testing should start with the cold temperatures first.

Data Not Recommended for Collection

It can be seen from Table 1 that less than half of the data considered was recommended for elimination from the collection effort. Although some of the data not eliminated may later be demonstrated to be statistically insignificant, it would not be appropriate to eliminate them at this time. Each of the items eliminated is discussed below.

Cloud cover and wind speed affect asphalt concrete pavement temperatures, which in turn have a strong effect on the material properties of the asphalt concrete. However, it is believed that the mean daily temperature for summer months and winter months plus the solar radiation will sufficiently explain the variations in asphalt concrete temperature for the purposes of the proposed studies.

The moisture contents for subgrade, subbase, and base layers were eliminated because these values would be very dependent on the time of year when the samples were removed and would be subject to continuing variation with time. Although moisture content has considerable effects on resilient modulus and permanent deformation potential, it is not considered practical to establish a requirement for a continuing sampling program on an annual or periodic basis. In addition, the effects of moisture content on the response of these layers to load will be represented satisfactorily by other material properties, annual precipitation, and periodic deflection measurements.

The resilient moduli for the subgrade soil, subbase, and base layers were omitted partly to avoid

sampling (assuming the other material property data are available from design and construction records), partly because the resilient moduli vary seasonally with moisture content, and partly because resilient moduli are also satisfactorily explained by other material properties and by the periodic deflection measurements. The modulus of subgrade reaction was also eliminated for essentially the same reasons.

The fatigue life potential, permanent deformation potential, and Lottman tensile strength ratio are arrived at by relatively sophisticated test procedures for which few commercial or state materials laboratories are equipped. Consequently, other means for representing these data are needed. Fortunately, the fatigue life potential of an asphalt concrete has been found to be very dependent on three other material properties or factors that are included as data to be collected: dynamic modulus, asphalt content, and air voids. Other asphalt concrete material properties to be collected, such as viscosity of asphalt and type of coarse aggregate, may also offer additional "explanation" of the variation in fatigue life potential. As discussed previously, the permanent deformation potential is also correlated with the original stability of the mix used in design as well as with the other material properties discussed above.

It appears, then, that it is possible to include essentially all of the information to be gained from the sophisticated test program through other data that should be available from construction records. If these data are not available from construction records, most of them can be obtained through extraction tests on cores removed.

Polish of surface aggregates is an important factor in the rate of loss of skid resistance, but it is a difficult property to determine in the laboratory and may be adequately represented by type of coarse aggregate and skid numbers from the monitoring data. The modulus of elasticity of the concrete in rigid layers is important but does not vary over a large range, and its effects may be adequately "explained" by a combination of deflection measurements and modulus of rupture. The thermal coefficient of concrete is important to volume change in concrete, but it is very difficult to obtain and is generally negatively correlated with modulus of rupture.

It should be noted that percentage of modified AASHTO compaction is specified for uniformity for

the base and subbase layers. Since standard Proctor or AASHTO compaction was frequently used for density control, it will be necessary to convert from standard to modified AASHTO compaction. For one limestone material in Texas, maximum densities were determined for a range of compactive energies. The maximum density for modified AASHTO compaction was 8 percent higher than that for standard AASHTO compaction. It is believed that this is rather typical and that little error would result if the percentage of standard AASHTO compaction were simply divided by 1.08 to convert to percentage of modified AASHTO compaction.

Temperature susceptibility of the asphalt was also proposed for elimination. It is of primary importance to the occurrence of low-temperature cracking and flushing but is not considered to be a strong parameter and will also be partly represented by other properties.

#### DATA COLLECTION

##### Uniformity of Data

One of the biggest problems with the use of historical data (available in limited quantities in state DOTs) is the incompatibility of the data. As discussed previously, there are many methods in use among the individual states for measuring cracking, roughness, deflections, and other important pavement indicators. Without some means to successfully correlate these diverse data, there is really little hope of using them to develop reliable relations. Consequently, it is critically important that the procedures and testing requirements for this data-collection effort be uniform. It will not do to consent to a state DOT using the Bureau of Public Roads Roughometer if an accelerometer-based roughness system is selected for general use. Benkelman beam deflections are not adequate since they do not generally provide basin information. Pavements that have unusual characteristics (such as rigid pavements with unusual load-transfer devices) should be avoided since they are not representative. If the proposed monitoring efforts are not to degenerate into ineffective exercises in trying to match "apples and oranges", the inconvenience and extra expense for this standardization in the measurements must be accepted and implemented from the beginning.

All condition surveys to obtain information on distresses must use the same rating forms and must generally report severity of the distress as well as the area affected. A number of state DOTs, such as Washington and Texas, use measurements of both area and severity, but many of the states do not. The selection of forms for conducting the condition surveys will require some study in order to ensure that the data are in a form directly useful for multiple regression analyses and may be practically obtained in the field. To the extent possible, these forms should be similar to those in common use.

There is much variation in the perception of pavement engineers and technologists regarding the identification of certain distresses and assignment of severity levels. To reduce the variability in these perceptions, several manuals have been developed over the years. The latest and perhaps the most comprehensive is the manual developed by Smith, Herrin, and Darter (5), which is recommended as a standard.

##### Centralized Collection of Some Data

The usual approach to data collection at the national level is to levy requirements on the state DOTs and to provide some or all of the funding for

collection. This is the approach planned for an FHWA pilot study and may be the approach taken for any broad, long-term monitoring studies in the future. Similar divisions of monitoring responsibilities may be made through delegation to districts within states, but centralized data collection by teams mobilized for that purpose (in-house or by contract) should be considered. The primary advantage of a centralized data-gathering effort would be uniformity in those data. It may be very difficult to get state personnel who are already committed to specific condition-measurement techniques and procedures to use some other method in a uniform manner. The trade-offs between improved data for study, management of one or more teams instead of dealing with a number of states or districts, relative costs, and the politics involved should be assessed during planning for individual data-collection efforts.

There are some data that are best collected centrally in any event. These include environmental data, which are generally available in a centralized data base, and some traffic and axle-weight distributions that are available from W-4 tables. Other traffic and axle-load distribution data will undoubtedly be required, but test sections for which traffic and weigh-station data are available should be used where possible.

##### Centralized Data Management

The data collected may be used by both state agencies and FHWA. These data should be controlled and stored centrally at state and/or national levels by suitable "data managers", which are software packages designed to store and process data so that they are available when needed and can be easily manipulated for the intended purposes. A number of systems are available and in use by both federal and state governments.

##### Pavement Maintenance Costs

Some explanation is required for definition of accumulated pavement maintenance costs as differentiated from routine maintenance. The intent is to include those costs that can be related to axle loads or to the environment and that represent correction of distresses to the pavement itself.

#### SAMPLE SIZES

##### Long-Term Pavement Monitoring Studies

As with most sampling surveys, the goal for long-term monitoring studies will be the selection of the smallest sample sizes possible that will still provide sufficient information of suitable quality to support the multiple regression analyses.

Mahoney and Lytton (6) conducted studies to establish a suitable sample size for a network analysis for the State of Texas. Their purpose was to provide valid data for use by the state highway administrator in allocating highway rehabilitation and maintenance funds. Although the purposes of other long-term studies may be different, distress and performance measures are essentially the same and much insight can be gained from the Mahoney and Lytton studies.

The type of sampling conducted was a stratified two-stage random sample over the entire state. Two-mile highway segments were used, and approximately 1 percent of the statewide total centerline mileage was sampled. Construction, traffic, climate, roughness, visually determined condition, deflection, rut depth, and skid resistance are typical of the kinds

of information sampled for each of the highway segments involved. The stratification involved division of the highway network into the 25 districts. The two-stage sampling was obtained by first randomly sampling counties within each district and then randomly sampling the two-mile highway segments within each county. An average of four counties per district was used. There were a total of 250 test sections or segments, of which 21 were Interstate, 109 were U.S. and state highways, and 120 were farm-to-market roads. The percentage of centerline Interstate highway sampling was 1.8, that for U.S. and state highways was 1.0, and that for farm-to-market roads was 0.6. A mass inventory was also conducted of all the highway mileage in one district in order to study the effects of different sizes of samples.

The random sampling technique applied to selecting the test sections within a county did not always produce all three of the types of highways nor even a distribution. Consequently, a larger sampling of Interstate highways was selected for subsequent studies.

The simulation studies conducted on the mass inventory for the one district indicated that the optimum sample lies between 1.5 and 6.6 percent of the centerline mileage, depending on the ranges of utility weights applied to different attributes. As an example, the optimum sampling rate would be 1.5 percent if the costs of conducting the survey were weighted three times as heavily as sampling variability. If both attributes were weighted the same, an optimum sampling rate of 2.3 percent would result.

It appears that combinations of multiple stratification and staged random sampling are appropriate for long-term monitoring studies. The first stratification for a national study would logically be selection of all environmental regions, and then a random sample of the states within each region could be made. The next level of stratification might then be highway functional classes, and a third level would be urban and rural highways. The fourth level of stratification should then be type of highway (rigid or flexible). The selection of test sections could then be random and based on some selected length. The test sections selected would then include combinations of geographic regions, states, functional classes, rural or urban locations, and types of pavements. For a single state, a random sampling of districts might be appropriate as the first step in sample selection.

The overall sample size should ideally start at a relatively high level of the apparent required range--say, 5 percent of the centerline mileage of pavements within a state and of a specific functional class, rural or urban location, and pavement type. Simulations of partial data could then be conducted to determine what the effect on the data would have been if a smaller sample had been obtained the first year. After the second set of monitoring data has been received and the results are analyzed, it may be appropriate to cut the sample size down further to, say, 2 or 3 percent. For economic reasons, it may be necessary to start with a smaller sample and reduce it to, say, 1 percent, although some loss in accuracy may result.

The frequency of sampling for some data (such as deflection tests and cores) must also be established but will depend on test-section lengths and other considerations. Test-section samples of 10-20 percent have been used successfully for condition surveys.

Pilot Studies or Other Studies of Limited Scope

FHWA is currently implementing a pilot study that was initially to include the monitoring of a minimum of some 50 test sections located in five states. The five states to participate (seven were actually selected) were to be selected on the basis of interest, availability of useful existing data, proposed approach to the monitoring task, and environmental conditions. With only 50 sections and the further limitation that they be divided among five states, it is apparent that the sampling techniques discussed above for the broad study could not generally be applied. In addition, the limited size of the factorial requires that some levels of discrimination be determined.

We recommended for this limited study that only rural highways be considered and that there be no direct consideration of functional class or location in a state. The actual distribution of the average of 10 test sections in a state could be in terms of type of pavement and combined traffic and axle loads. Assuming that only typical flexible and typical rigid pavements are included (total of two types), an average of 5 test sections per state, or a total of 25, may be monitored for each pavement type. The selection of actual numbers of test sections for each pavement type within a state could also reasonably reflect the percentage of total mileage within that state for each type.

The test sections for each pavement type should include a distribution of traffic levels and axle loads--i.e., highways with low, moderate, and high traffic levels and a more or less typical distribution of axle-load magnitudes. As traffic and axle-load data are very important, test sections should be selected where relatively accurate and comprehensive data exist.

When a limited factorial is planned, the test sections selected should be "screened" to the extent possible to ensure that they are representative and will not reflect nontypical problems, such as stripping of asphalt concrete or abnormal joint problems in rigid pavement caused by deficient design or construction practice rather than loads or environment. In addition, only one type of rigid pavement--jointed reinforced or plain jointed--should be included.

An alternative approach to designing this limited study might have been selection of one state, a "cluster sample" in one part of the state, and a combination of stratified and random sampling within the cluster. Although this would have offered some advantages in testing sample design techniques and allowed consideration of urban test sections and functional classes, it would not have allowed any environmental discrimination or broad state participation.

A rough approximation of precision for this small sampling may be evaluated by using Equations 1-3 in the paper by Lytton and others in this Record. Assuming 25 test sections for a pavement type, the detectable percentage change in mileage of test sections that are experiencing unacceptable distress levels would be as follows:

| Level of Confidence (%) | Detectable Change (%) |
|-------------------------|-----------------------|
| 70                      | 14.7                  |
| 80                      | 18.2                  |
| 90                      | 23.3                  |
| 95                      | 27.7                  |

## SUMMARY

This paper describes the requirements of long-term pavement monitoring studies to collect data for use in the development of multiple regression relations among pavement types, traffic loadings, environmental factors, and other important parameters. The study approach for this paper is aimed specifically at defining data requirements that would support development of multiple regression relations, but it is hoped that a reasonable amount of the data might be common to data-collection activities for other purposes, such as identifying needs for maintenance or rehabilitation, project design, and budgeting funds for these activities.

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## Simplified Pavement Management at the Network Level

R. DARYL PEDIGO AND W. RONALD HUDSON

A simplified pavement management system at the network level is presented, and an example is provided to demonstrate how this framework can be applied to produce a priority ranking on a network basis. This framework has been specifically designed to be independent of the organization of any specific highway agency. The framework is organized around the flow of information on either management level, and three major subsystems are identified at each level. Essential features of pavement management systems are identified, and specific characteristics are described for the example models and outputs. Existing pavement management practices are reviewed to demonstrate several different levels at which pavement management activities are occurring in U.S. agencies. The findings of the study suggest that implementation of simple systems can probably best begin at the network level of pavement management. These simple steps can be coordinated with later development work to recognize analysis of alternatives and optimization at the network level. A research plan and problem statements are included to address continued development and implementation at both the network and project levels.

Pavement management is a concept that involves the coordination, scheduling, and accomplishment of all of the activities performed by a highway agency in the process of providing adequate pavements for the public. The systems approach to pavement management is a rational, highly structured process that attempts to achieve the best value possible for the public funds expended to provide pavements. This is accomplished by comparing investment alternatives; coordinating design, construction, maintenance, and evaluation activities; and making efficient use of existing methods and knowledge (1). Of course, management decisions are made each day in the course of normal operations of highway agencies throughout the nation. The purpose of a pavement management system (PMS) is to improve the efficiency of this

decision-making process, expand its scope, provide feedback regarding the consequences of decisions and the results of activities, and ensure the consistency of decisions made at different levels within the same organization (2).

Many agencies and individuals have conducted research into the various component models and procedures involved in pavement management. A significant portion of this work has been summarized in two recent books (1,3), which suggest that there are several major underlying considerations in pavement management:

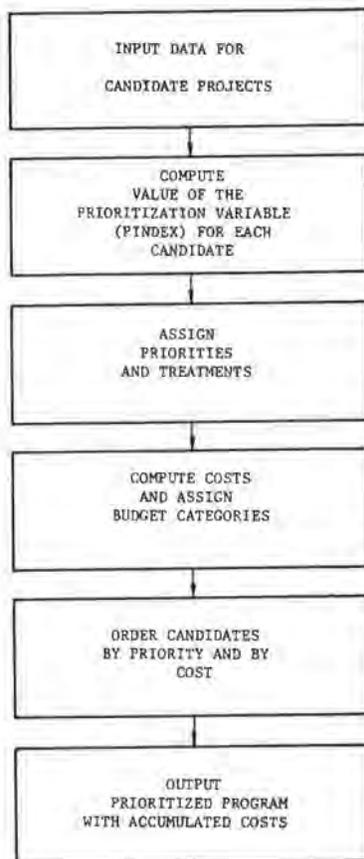
1. Management decisions occur at several levels, ranging from investment decisions covering the network to detailed design decisions at the individual project level.

2. Periodic, in-service evaluation of existing pavements is basic to the programming of rehabilitation and maintenance, the updating of earlier design estimates, and the improvement of models.

3. A PMS must be capable of being adapted to the varying needs and resources of different agencies in order to be implemented. It must also be capable of serving the various management levels noted in item 1 above.

These considerations led to the development of a general framework for PMSs during the first phase of research under this project (2). A major finding of this study is that most PMS development and experience to date have occurred at the project level and within the areas of design or maintenance. This

Figure 1. Basic steps in rehabilitation programming.



concentration of effort has produced many significant results, and it has also created a void in network-level development. Consequently, the greatest current need is for a comparable focusing of effort at the network level. This need derives largely from the concurrent need to preserve investments in existing pavements and to obtain the maximum value for limited available dollars. A major portion of these funds is required for rehabilitation, which makes this a prime area for the concentration of initial development. This is in consonance with the findings of the Tumwater Workshop on Pavement Management (4).

A PMS is a tool for use in decisionmaking; as such, it is highly specific and particularly structured to the attitudes and procedures of the implementing agency. Consequently, many of the details of a PMS must be fit or molded by the implementing agency. Nevertheless, significant portions of the development and implementation work involved in setting up and operating a PMS are potentially applicable to a wide variety of uses.

#### SIMPLIFIED NETWORK-LEVEL PMS

A simplified PMS for network-level rehabilitation programming (PMS-N), based on the framework and characteristics discussed above, is presented here. The system to be described represents a "bare-minimum" PMS-N. However, this paper provides recommendations and examples for upgrading this simplified system, as envisioned in Figure 1.

The desired result for a PMS-N is a specified program of work to be performed annually, including rehabilitation and maintenance. Ideally, this would include a list of projects to be rehabilitated during each construction season over a period of

5-20 years. The simplified version considers programs one year at a time and provides a prioritized listing of projects to be rehabilitated during the last year of the program.

The basic steps in a simplified programming scheme are shown in Figure 1. These represent the minimum procedures required for programming in a PMS-N and comprise a subset of the 15 steps in the priority programming process identified in a recent synthesis report (5).

#### Input Data for Candidate Projects

The information required for the technical, nontechnical, and economic analyses to be carried out in subsequent steps must be provided for those projects that are candidates for rehabilitation during the programming period. The bare-minimum data required for the simplified PMS-N are as follows:

1. Project identifiers--(a) Project limits (description) and (b) milepost or control section numbers;
2. Project characteristics--(a) Length, number of lanes, and lane width, (b) pavement type and last rehabilitation, (c) functional classification, and (d) shoulder type and width;
3. Engineering data--(a) All variables required for the calculation of the prioritization variable (PINDEX) (as a minimum, the serviceability index), (b) safety variables, if not included in item a (as a minimum, skid number of accident rates), and (c) traffic variables [as a minimum, average daily traffic (ADT)];
4. Nontechnical data--Identifier to flag "committed" projects; and
5. Economic data--(a) "Average" costs may be used as a minimum, so that no data will be required for individual projects, and (b) eligible budget category or categories.

Candidate projects must be selected before prioritization can be carried out. For a small highway network, it may be convenient to consider all pavement sections as candidates, whereas for a larger network a screening process may be necessary to reduce the data-collection and analysis efforts. One possible screening method involves the routine monitoring of a simple variable, such as serviceability, for all roadway sections. Then, based on the value of this variable, the "worst" 25-50 percent of the existing pavements may be chosen for further analysis. Alternatively, each district could be charged with selecting candidates, or the agency may use its existing procedure for project selection. In any event, it is desirable that the candidate selection process be compatible with the prioritization analysis. For example, if roughness is to play a large part in determining rehabilitation priorities, then the selection process should be designed to ensure that all very rough pavements are considered for inclusion as candidates.

Once candidates have been selected, the project identifiers and project characteristics listed above must be recorded in the PMS data base. This data base may be a separate computer file or a set of data records exclusively for the PMS function, or it may be simply a master list that indicates where all of the necessary individual data elements may be found. In either case, the information must be readily accessible to the PMS staff, and such access is efficiently provided in a computerized data management system.

The same type of information must be provided for all projects. For example, either mileposts or control section numbers can be used to identify a

project, but it is unacceptable to use milepost identifiers for some projects and control section numbers for others. In addition, all data must be keyed to the same identifier. Thus, if construction information is currently reported by control section and engineering data are reported by milepost, it will be necessary to convert from milepost to control section (or vice versa) in setting up the PMS data base. This can be a troublesome undertaking, but some states (e.g., Washington) have already accomplished such a conversion.

The engineering data collected on each project must be updated on a regular schedule. Generally, data will be collected on each section each year; however, this requirement may be relaxed to allow data collection every other year or every third year on pavements that are known to be in good condition. Such decisions are made on the basis of budget, manpower, and equipment constraints, and it is preferable to carry out a limited monitoring program accurately and completely rather than to hastily and partially perform a more comprehensive survey. However, it should also be remembered that these data are to be used for overall judgments only and that those candidates that appear in the final work program will generally require further scrutiny before any rehabilitation activity is performed. It is therefore desirable to limit the scope of the data collected for the determination of the prioritization variable (PINDEX) and to gather more complete data only on those projects that make the cut. Thus, the obvious or most simple choice (PINDEX = the existing sufficiency rating) may not prove to be the best choice in every agency.

For example, suppose that a sufficiency rating is currently performed in programming projects for rehabilitation. A typical sufficiency rating procedure involves three factors (condition, safety, and service), and each factor requires the evaluation of approximately five variables (6). In addition, many states now include three other type factors (environmental, social, and economic), each of which also involves several variables. This means that 15-30 or more variables must be measured or assessed in order to arrive at a sufficiency rating. When faced with the prospect of gathering such information systemwide, an agency might well decide that it can only afford to carry out such a rating on one-half or one-third of the highway system annually. For prioritization purposes, the agency may feel that it is more important to have reliable, current information of a less extensive nature on the entire network each year. If this is the case, PINDEX should be constructed from only a few of the most crucial sufficiency variables. In fact, a single variable may be chosen as the PINDEX if the agency feels that this variable provides enough information to allow a meaningful prioritization. We feel that serviceability is the best candidate for a single-variable PINDEX but that it would also be preferable to incorporate other variables into PINDEX if resources are available to do so.

Safety variables may, of course, be directly incorporated in the calculation of PINDEX. However, during the course of this research, we discussed this possibility with representatives of several highway agencies. The vast majority of those consulted indicated a desire or preference for developing a separate program of safety-related projects or in any case to avoid assigning specific weights to safety variables in relation to pavement condition variables. Only a few persons indicated that it would be desirable to incorporate safety variables in the determination of PINDEX. Hence, we have developed a procedure that treats safety variables independently without impact on the value of PINDEX.

The prioritization procedure may be used with or without the safety module so that safety projects may be totally excluded from consideration in the simplified PMS-N if the agency so desires.

#### Analysis of Data and Computation of PINDEX

The prioritization procedure for the simplified PMS-N is summarized in Figure 2, which is an expanded version of Figure 1. The first element of this procedure is the development of a complete list of candidate projects along with the data on each project that are required for prioritization and economic analysis. Any candidates that have already been approved in other programs are deleted from further consideration. Those projects that have been held over from previous years, "promised" to local governments or other agencies, or otherwise previously committed for completion are assigned highest priority and are not technically evaluated.

Next, the remaining projects are evaluated. The value of PINDEX is calculated for each project from the input data according to the method selected by the agency. Any number of variables may be used by the agency in constructing the prioritization variable PINDEX, but it is recommended that no more than three variables be used in the initial implementation of this procedure. The definition of PINDEX may be extended fairly easily to incorporate additional variables, if so desired, after some experience is gained in the operation and results of this procedure.

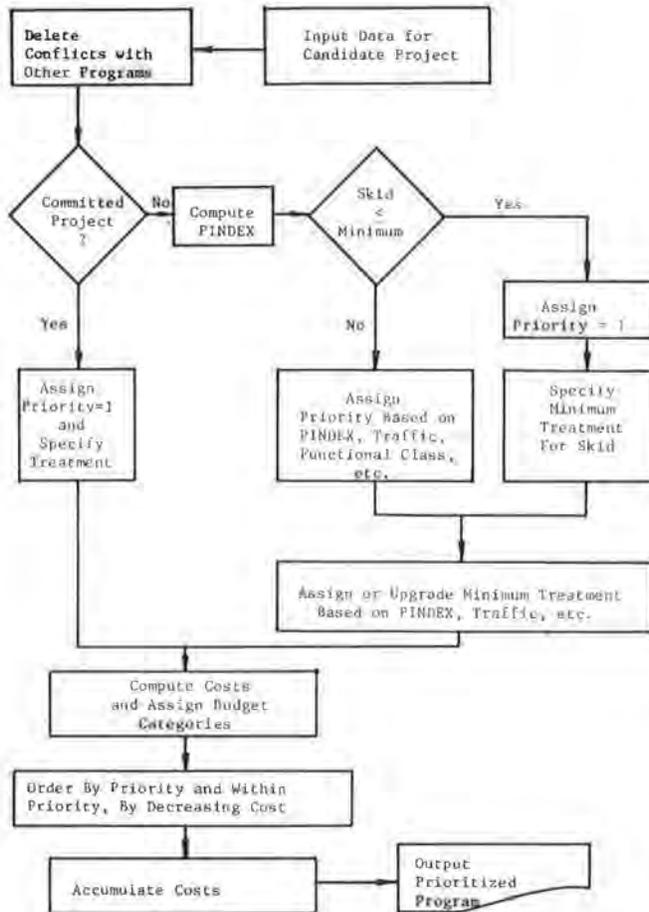
Once the value of PINDEX has been computed, the projects may be prioritized. However, as mentioned previously, an option has been provided at this point for the treatment of safety projects outside the framework of the PINDEX calculation. For this purpose, skid number ( $SN_{40}$ ), as measured by almost every state by using a standard American Society of Testing and Materials skid trailer, has been selected as the safety variable. This choice was made for example purposes because this variable is almost universally recognized, is easily measured, and appears in the form of an easily understandable numerical index. Another safety-related variable, such as accident rate or wet-weather-safety index, could easily be substituted in this procedure should the agency so desire.

The value of skid number recorded for each project is compared with a fixed minimum value determined by the agency. If the minimum standard for safety is not met, the project is assigned highest priority and a minimum treatment to correct this deficiency is assigned. The agency must choose an appropriate minimum treatment, and it is recommended that several alternative minimum treatments be specified, depending as a minimum on the functional class of roadway involved and the level of traffic to be carried.

If the minimum standard for safety is met or exceeded, the project is assigned a priority based on the value of PINDEX. In the case of safety projects, all deficient projects received highest priority; however, for resurfacing or structural rehabilitation it will be desirable to assign highest priority to major roadways that have high traffic volumes. Hence, priority is to be determined on the basis of functional class of roadway and ADT as well as PINDEX.

At this point, a minimum treatment is assigned to each project on the basis of PINDEX, ADT, and functional class of roadway. This minimum treatment represents the least costly action that can be carried out to return a deficient section to acceptable condition or maintain an adequate section in acceptable condition. As before, in assigning such

Figure 2. Flowchart of prioritization procedure.



minimum treatments the agency should consider not only the condition of the pavement but also the functional class and level of traffic. However, the priority of the project should not have a direct impact on the nature of the treatment (it will, of course, ultimately determine whether or not the treatment is carried out). The exception to this rule is that the lowest-priority projects will virtually always receive a treatment such as "no rehabilitation--continue routine maintenance".

A list of example minimum treatment options and some example criteria for their application is provided later in this paper.

Output

Since a main purpose of the simplified PMS-N is to produce a prioritized program, the major output report must be a listing of this program of work. It should be remembered, however, that the information collected and the analysis performed may prove useful for other purposes as well. Hence, several types of optional output may be prepared. For example, a listing of the total estimated quantities of materials needed to carry out the program, or a listing of the average condition of the existing highway network by functional class, might be desirable. If the PMS-N is computerized, such output reports will be relatively easy to develop, produce, and modify. In fact, this flexibility in generating output is one of the major benefits derived from a computerized PMS.

The exact nature of these optional reports will

be very agency dependent. The primary output report will, of course, also depend on the needs of the individual agency, but generally very similar types of information on the prioritized program will be needed by most agencies. It is, therefore, this primary output report that is treated here.

The minimum information content of the primary output report on the prioritized program of work is as follows:

1. Project identifiers--(a) Project limits (description) and (b) milepost or control selection numbers;
2. Technical information--(a) Priority assigned to the project, (b) summary of the deficiencies for which treatment is recommended (as a minimum, a numeric or alpha-numeric code indicating the general nature of the deficiency or deficiencies); and (c) recommended minimum treatment as a bare minimum (this may be a simple code, such as M = continue maintenance or R = rehabilitate); and
3. Economic information--(a) Cost of the proposed treatment, (b) allocation of this cost to eligible budget categories (as a minimum, a simple list of eligible budget categories), and (c) cumulative costs of this project and all higher-priority projects.

In general, it is recommended that the projects be listed in order of decreasing priority and that projects of equal priority be listed in order of decreasing cost. This may, of course, be modified by the agency so that projects may be listed by district, budget category, functional class, or in any other useful arrangement. If a priority value is listed for each project, no information will be lost by reorganizing the output listing.

APPLICATIONS, INTERPRETATION, AND APPRAISAL

The findings of this project are now brought into a more practical focus. As an illustration of the applicability of the techniques discussed in the preceding section, a detailed sample problem is worked out. This sample problem is for illustrative purposes only and is neither totally representative of typical conditions nor directly applicable to any existing highway agency.

Following the sample problem, a phased implementation plan for a network-level PMS is presented. This plan is intended to assist highway agencies in implementing the techniques described in this paper. Some specific guidelines for application of these principles are also discussed, and the current practices of several states are reviewed.

Sample Problem

The specific values quoted in this example problem were chosen on the basis of experience and discussions with representatives of various highway agencies. The problem itself is worked out in sufficient detail to provide step-by-step guidance for agencies that wish to develop their own systems along these lines. It must be emphasized that the specific choices of variables and values to be used in the simplified PMS-N are extremely agency dependent. Consequently, the specific choices given here are not recommendations; they are examples only. Each agency must assess its own needs and make its own choices regarding the relevant variables to be used, standards to be adopted, terminal values, etc.

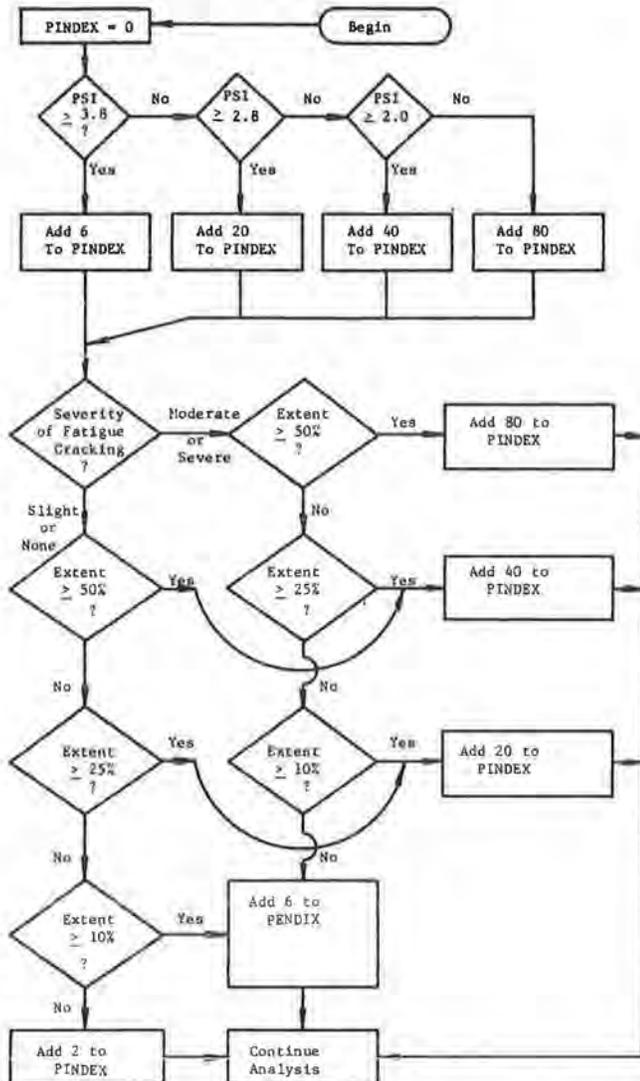
The input data for the eight candidate projects chosen for the sample problem are given in Table 1. These data are based on actual values taken from field studies, modified to produce the desired range

Table 1. Project input for sample problem.

| Project Identification | Milepost |       | Lanes |            | Pavement Type           | Functional Class   | Fatigue Criteria |          |            |        |
|------------------------|----------|-------|-------|------------|-------------------------|--------------------|------------------|----------|------------|--------|
|                        | Begin    | End   | No.   | Width (ft) |                         |                    | Avg PSI          | Severity | Extent (%) | ADT    |
| 51-FI-0356700-E        | 100.4    | 105.0 | 2     | 12         | AC-overlay <sup>a</sup> | Principal arterial | 2.4              | None     |            | 6 000  |
| 54-MA-0356712-S        | 16.3     | 18.7  | 1     | 12         | AC                      | Local              | 2.6              | Slight   | 30         | 500    |
| 05-ST-0311130-S        | 6.5      | 8.0   | 2     | 12         | AC                      | Minor collector    | 2.1              | Moderate | 60         | 1 400  |
| 26-CO-0356700-N        | 7.5      | 9.3   | 1     | 12         | AC                      | Minor arterial     | 1.9              | Slight   | 10         | 4 200  |
| 52-KI-0330000-W        | 43.8     | 46.5  | 2     | 12         | AC-overlay <sup>d</sup> | Principal arterial | 3.1              | Slight   | 5          | 12 500 |
| 58-OA-0330001-N        | 27.3     | 30.8  | 1     | 12         | AC                      | Major collector    | 2.9              | Severe   | 5          | 3 900  |
| 75-AP-0330006-N        | 11.2     | 15.3  | 1     | 12         | AC                      | Minor arterial     | 2.7              | Slight   | 15         | 4 000  |
| 05-ED-0311130-S        | 9.4      | 12.7  | 1     | 12         | AC                      | Local              | 2.0              | Severe   | 60         | 600    |

Note: PSI = present serviceability index and AC = asphalt concrete.  
<sup>a</sup>Portland cement concrete.

Figure 3. Calculation of PINDEX for sample problem.



of solutions. Since the significant variables, terminal values, and treatment options will vary with pavement type, only pavements with asphaltic concrete (AC) surfacing were chosen. Portland cement concrete pavements can of course be treated in a similar fashion.

Arbitrary project identification codes were assigned to each project for illustrative purposes.

In practice, each agency will have its own coding system for identifying projects. Beginning and ending milepost values are coded according to an imagined state route milepost system. These projects range in length from approximately 1 to 5 miles. Both two and four-lane roadways are included; however, projects are defined in this sample problem according to the direction of traffic flow so that the number of lanes quoted represents the number of lanes in the direction of traffic flow.

Since it will generally be necessary to consider programs that involve pavements of different functional classes, the sample problem includes arterial, collector, and local roadways. ADT levels range from 500 to 12 500. The average values of serviceability and distress recorded in Table 1 represent roadways in less-than-desirable condition--i.e., roadways that are candidates for rehabilitation.

The economic input required for the sample problem involves only average unit costs for potential rehabilitation actions, since this approach does not require project-specific cost calculations. The required values are specified below (SAMI = stress-absorbing membrane interlayer):

| Rehabilitation Action                   | Avg Cost (\$/yd <sup>2</sup> /unit of thickness) |
|---|--|
| AC overlay                              | 2.25   |
| AC leveling course                      | 2.32   |
| Place new AC                            | 2.25   |
| Replace cracked areas with 6-in AC      | 19.20  |
| Cold plane                              | 3.25   |
| Heater plane                            | 1.47   |
| Heater scarify and compact              | 0.60   |
| Remove existing AC                      | 0.50   |
| Full-depth AC                           | 2.25   |
| Fabric in traffic lanes                 | 1.21   |
| SAMI in traffic lanes                   | 2.00   |
| Chip seal                               | 0.55   |
| No rehabilitation, continue maintenance | 0.00   |

Again, although these costs are believed to be reasonable for each listed activity, these values are included for illustrative purposes only and should not be used by any agency without independent verification.

For simplicity, no safety variables are considered in this example, and no projects are preselected as "committed". In addition, no attempt has been made to assign projects to different budget categories.

For each of these projects, it is necessary to calculate a value of PINDEX. The logic for accomplishing this calculation is illustrated in Figure 3. Two variables have been chosen for use in calcu-

lating PINDEX: PSI and fatigue cracking. These variables were chosen for illustrative purposes and are not being "recommended". We do feel, however, that PSI offers an inexpensive, reasonable, overall assessment of the adequacy of a pavement to serve traffic and, in conjunction with structural and/or condition survey variables, can be used to prioritize and derive generalized rehabilitation strategies for programming purposes. Fatigue cracking was chosen as a significant condition indicator that could be used along with PSI in this fashion. Each agency must select a set of variables that is appropriate to its purpose and experience. For example, many agencies feel that deflection information would be very useful for such purposes, and this choice certainly could be used within the simplified PMS-N. In choosing an appropriate set of variables, it should be kept in mind that the methodology recommended here is to be applied for programming purposes only. It will be desirable to supplement this information with additional data in order to finalize the rehabilitation design for any particular project (and update the program accordingly) before work is actually carried out.

The calculation of PINDEX in this sample problem involves the categorization of the condition of the pavement by values of PSI and the severity and extent of fatigue cracking. The categories chosen for this sample problem are given in the two tables below:

#### Serviceability

| Category  | PSI     |
|-----------|---------|
| Very good | 3.8-5.0 |
| Good      | 2.8-3.7 |
| Fair      | 2.0-2.8 |
| Poor      | 2.0     |

#### Fatigue-Cracking

| Category  | Severity        | Extent (%) |
|-----------|-----------------|------------|
| Excellent | Slight          | 10         |
| Very good | Slight-moderate | 10-25      |
|           | Severe          | 10         |
| Good      | Slight          | 25-49      |
|           | Moderate-severe | 10-25      |
| Fair      | Slight          | 50         |
|           | Moderate-severe | 25-49      |
| Poor      | Moderate-severe | >50        |

The specific values chosen to delineate these categories were developed on the basis of the experience of project staff. However, the number of categories and their specific delimiters are examples rather than recommendations.

The values added to PINDEX in Figure 3, which depend on the specific PSI or fatigue category into which the project falls, were also chosen based on their experience. We believe that the values are reasonable, but, again, they are illustrative values only. These values were chosen so that pavements that fall into the "poor" category of either PSI or fatigue will achieve a relatively high priority for rehabilitation. Pavements that fall into the "fair" category for either PSI or fatigue will receive considerably less emphasis.

The next step in the simplified PMS-N involves the development of treatment options and the assignment of priorities. For this sample problem, all treatment options (with the exception of the option "no rehabilitation, continue maintenance") will provide a new surface for the roadway, thus automatically improving any serviceability problem. Hence, the specific range of treatment options to be considered for a project will be selected on the basis of the severity and extent of fatigue cracking. The logic for this selection process is shown

in Figure 4. Notice that PSI is considered only if the project falls into the "excellent" category for fatigue cracking and that, in this case, only those projects that fail to meet a minimum PSI standard will be rehabilitated. For example purposes, minimum PSI standards of 2.5 for Interstate and arterial roadways and 2.0 for all other roadways were selected. These values were chosen for illustrative purposes only and are not to be considered recommendations.

The specific rehabilitation actions that make up each option list are discussed subsequently in the sample problem.

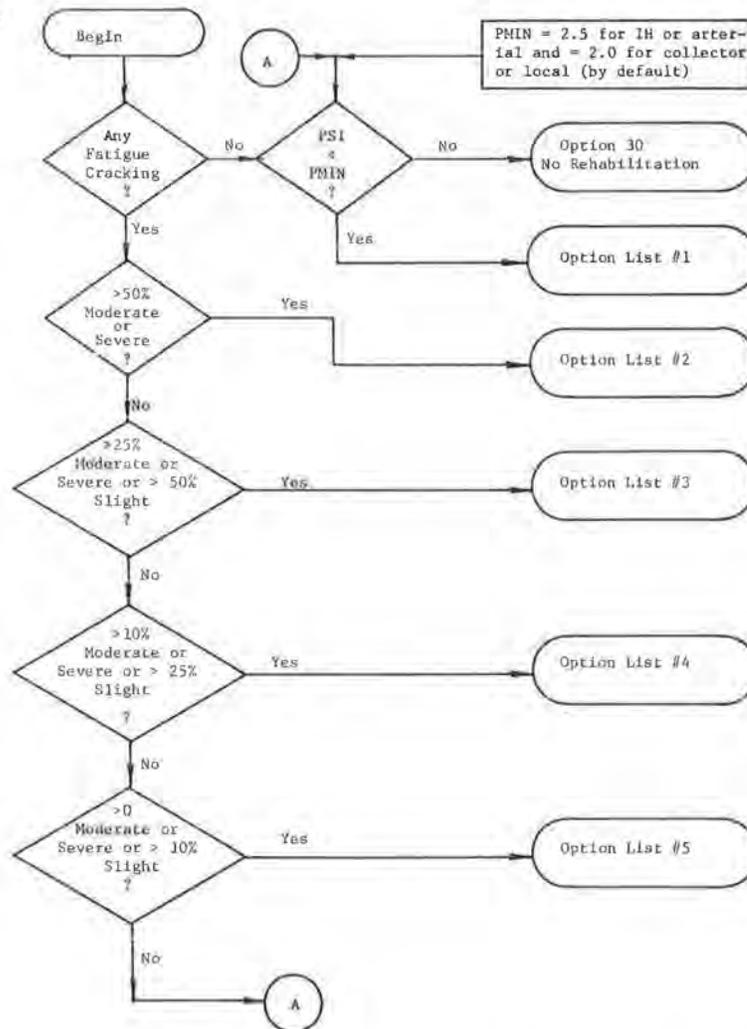
The prioritization of projects of mixed functional classes and varying traffic levels could hardly be carried out on the basis of the PINDEX obtained from Figure 3, since the values in Figure 3 make no distinction between local roadways and Interstate highways or between roadways with low traffic and similar roadways with high traffic. In order to take such variations into account, the value of PINDEX may be adjusted to reflect the relative priority of projects with roughly equal serviceability and fatigue cracking in various functional classes and with various traffic levels. Table 2 gives a list of factors that may be multiplied by PINDEX in order to assign greater priority to the higher functional class and also to assign greater priority to roadways with high traffic levels within a given functional class. As with all numerical values in this sample problem, the values of this multiplicative factor were chosen to provide reasonable answers for illustrative purposes only. Similarly, the specific numbers of vehicles given in Table 2 are not intended to be representative but were chosen only for use with the sample problem.

The logic of the prioritization process is shown in Figure 5. The value of PINDEX, as calculated through the procedure of Figure 3, is used as input to the procedure of Figure 5. This value of PINDEX, on a 0-100 scale, is multiplied by the appropriate factor for functional class and ADT to obtain an adjusted PINDEX. It is this adjusted value of PINDEX that is used in assigning relative priorities to the projects. For this example problem, projects may be classified as priority 1, priority 2, or priority 3. It is important to note that projects will be ranked by the value of PINDEX within each priority class, so that certain priority-1 projects will be of higher priority than other priority-1 projects. The priority value is used merely as a rough indicator to separate projects into those that are very urgent and those that are in less immediate need.

The prioritization and treatment-selection processes described above were applied to each of the eight candidate projects given in Table 1. The results are presented in Table 3 and the two tables below. Table 3 is a sample output report that contains a prioritized listing of the candidate projects based on the value of PINDEX calculated from the procedure illustrated in Figure 5. For each project, a set of treatment options has been selected according to the logic presented in Figure 4. These options are described in the tables below. The first table gives sample problem rehabilitation options, and the second table gives option lists for the sample problem:

| Option | Description                           |
|--------|---------------------------------------|
| 1      | Cold plane, 1.0 in; new AC, 1.5 in    |
| 2      | Heater plane, 1.5 in; new AC, 1.5 in  |
| 3      | AC level, 1.0 in; AC overlay, 1.0 in  |
| 4      | Remove existing AC; new AC            |
| 5      | AC overlay, 1.5 in                    |
| 6      | Heater plane, 0.75 in; new AC, 1.0 in |

Figure 4. Selection of rehabilitation option.



| Option | Description                                     |
|--------|---|
| 7      | Cold plane, 1.0 in; new AC, 1.0 in              |
| 8      | Cold plane, 1.0 in; SAMI; new AC, 1.0 in        |
| 9      | AC level, 1.0 in; fabric; new AC, 1.0 in        |
| 10     | Remove existing AC + 1.0 in base; new AC        |
| 11     | Fabric; new AC, 1.5 in                          |
| 12     | Heater plane, 0.75 in; new AC, 1.5 in           |
| 13     | Cold plane, 1.0 in; SAMI; new AC, 1.5 in        |
| 20     | Apply chip seal                                 |
| 22     | AC overlay, 1.0 in                              |
| 30     | No current rehabilitation, continue maintenance |

| Option List No. | ADT | Option No. |
|-----------------|-----|------------|
| 1               | >X  | 1,2,3,4    |
|                 | <X  | 5,6,7      |
| 2               | >X  | 8,9,10     |
|                 | <X  | 11,13      |
| 3               | >X  | 8,9,10     |
|                 | <X  | 11,13      |
| 4               | >X  | 1,2,3,4    |
|                 | <X  | 5,6,7,12   |
| 5               | >X  | 22         |
|                 | <X  | 20         |

Table 2. Example prioritization factors based on functional class and ADT.

| Functional Class   | ADT    |                 | Factor |
|--------------------|--------|-----------------|--------|
|                    | Level  | No. of Vehicles |        |
| Interstate         | High   |                 | 1.00   |
|                    | Medium |                 | 0.95   |
|                    | Low    |                 | 0.88   |
| Principal arterial | High   | > 15 000        | 0.93   |
|                    | Medium | 5-15 000        | 0.87   |
|                    | Low    | <5000           | 0.80   |
| Minor arterial     | High   | >12 000         | 0.83   |
|                    | Medium | 4-12 000        | 0.75   |
|                    | Low    | <4000           | 0.68   |
| Major collector    | High   | >8000           | 0.73   |
|                    | Medium | 2-8000          | 0.65   |
|                    | Low    | <2000           | 0.60   |
| Minor collector    | High   | >5000           | 0.60   |
|                    | Medium | 1-5000          | 0.53   |
|                    | Low    | <1000           | 0.45   |
| Local              | High   | >3000           | 0.55   |
|                    | Medium | 500-3000        | 0.45   |
|                    | Low    | <500            | 0.35   |

Cost figures are also presented for each project in Table 3. For each project, the minimum-cost treatment option was selected from the list of potential treatments, and the cost associated with that minimum-cost option was calculated by using the

values identified in the text table on page 34. This minimum cost is listed for each project in Table 3 along with a cumulative cost that assumes that the minimum-cost treatment will be applied to each project in order of decreasing priority.

Notice that there is only a single priority-1 project among those in Table 3. This occurred in the sample calculation because the arterial roadways were found to be in relatively good condition and the prioritization factors listed in Table 2 automatically reduce the emphasis given to collectors and locals. The selection of priority 1, 2, or 3, illustrated in Figure 5, is purely arbitrary and may be omitted from the PMS-N without changing the order of ranking based on PINDEX.

It is also interesting to note that project 52-KI-0330000-W is listed in Table 3 as requiring no

rehabilitation, even though it has a higher priority than project 58-OA-0330001-N, which does require rehabilitation. The former project retains a higher priority because it is a principle arterial with a fairly high traffic volume, whereas the latter is a major collector with a moderate traffic volume. The inclusion of such "no rehabilitation needed" projects will have no effect on cost factors associated with the program of work but will provide a flag for the decisionmaker when the proposed program of work is reviewed. The presence of a large number of such projects in the prioritized output list could indicate that the prioritization factors in Table 2 and/or the treatment-selection process in Figure 4 should be modified. In fact, the specific variables and values chosen by an agency in setting up a prioritization procedure should be subject to constant review and revision in order to upgrade the value of the recommended programs of work at each iteration of the procedure.

Finally, a comparative ranking of the candidate projects in the sample problem on the basis of serviceability index alone is provided in Table 4. Notice that the order of the projects is by no means the same as in Table 3; however, the changes are generally shifts in order by only one or two places rather than between the top of the list and the bottom. This is to be expected, since the value of PINDEX is based on serviceability and on fatigue cracking and those pavements that have very low serviceability will also generally exhibit appreciable fatigue cracking. It should be noted, however, that those pavements in higher functional classes and with higher traffic levels (Table 1) have moved up the list in Table 3 relative to their placement in Table 4. This illustrates the value of the weighted prioritization process in Figure 5 and Table 2.

**Implementation of PMS**

The simplified PMS-N described earlier and illustrated in the sample problem may be adapted and implemented by pavement management teams from a variety of highway agencies. Implementation of a PMS is of necessity a unique and individual undertaking related to individual organizational characteristics, funding levels, and needs. The purpose of this project has been to present a generalized, simplified PMS that can serve as a guide for use and development in several state highway departments. In this section, some general background guidelines on implementation are presented, as well as a series of individual factors that can be used to benefit specific implementation plans.

The reader who is expecting a step-by-step cookbook on pavement management implementation will no doubt be disappointed in this paper. Experience shows that it is no more possible to develop a

Figure 5. Prioritization process.

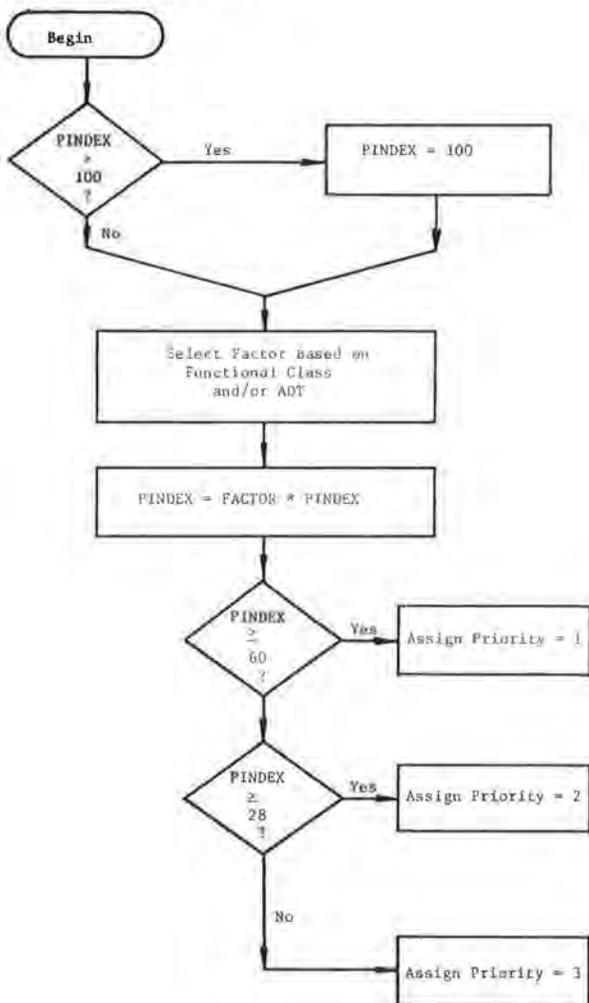


Table 3. Sample problem results.

| Ranking | Project Identification | Milepost |       | Priority | PINDEX | Fatigue Cracking |          |            |             | Minimum Cost (\$000s)     | Cumulative Minimum Cost (\$000s) |                   |
|---------|------------------------|----------|-------|----------|--------|------------------|----------|------------|-------------|---------------------------|----------------------------------|-------------------|
|         |                        | Begin    | End   |          |        | Avg PSI          | Severity | Extent (%) | Option List |                           |                                  | Treatment Options |
| 1       | 26-CO-0356700-N        | 7.5      | 9.3   | 1        | 65     | 1.9              | Slight   | 10         | 1           | 5 <sup>a</sup> , 6, 7     | 42.8                             | 42.8              |
| 2       | 05-ST-0311130-S        | 6.5      | 8.0   | 2        | 53     | 2.1              | Moderate | 60         | 2           | 11 <sup>b</sup> , 13      | 84.2                             | 127.0             |
| 3       | 05-ED-0311130-S        | 9.4      | 12.7  | 2        | 45     | 2.0              | Severe   | 60         | 2           | 8, 9 <sup>b</sup> , 10    | 134.3                            | 261.3             |
| 4       | 51-FI-0356700-E        | 100.4    | 105.0 | 2        | 37     | 2.4              | -        | -          | 1           | 5 <sup>a</sup> , 6, 7     | 218.9                            | 480.2             |
| 5       | 75-AP-0330006-N        | 11.2     | 15.3  | 2        | 35     | 2.7              | Slight   | 15         | 5           | 22 <sup>b</sup>           | 64.9                             | 545.1             |
| 6       | 54-MA-0356712-S        | 16.3     | 18.7  | 3        | 27     | 2.6              | Slight   | 30         | 4           | 5 <sup>a</sup> , 6, 7, 12 | 57.1                             | 602.2             |
| 7       | 52-KI-0330000-W        | 43.8     | 46.5  | 3        | 19     | 3.1              | Slight   | 5          | -           | 30 <sup>a</sup>           | 0                                | 602.2             |
| 8       | 58-OA-0330001-S        | 27.3     | 30.8  | 3        | 17     | 2.9              | Severe   | 5          | 5           | 22 <sup>a</sup>           | 55.4                             | 657.6             |

<sup>a</sup>Minimum-cost treatment.

Table 4. Sample problem candidate projects ranked by serviceability.

| Ranking | Project Identification | Milepost |       | Avg PSI | PINDEX | Ranking in Table 3 |
|---------|------------------------|----------|-------|---------|--------|--------------------|
|         |                        | Begin    | End   |         |        |                    |
| 1       | 26-CO-0356700-N        | 7.5      | 9.3   | 1.9     | 65     | 1                  |
| 2       | 05-ED-0311130-S        | 9.4      | 12.7  | 2.0     | 45     | 3                  |
| 3       | 05-ST-0311130-S        | 6.5      | 8.0   | 2.1     | 53     | 2                  |
| 4       | 51-FI-0356700-E        | 100.4    | 105.0 | 2.4     | 37     | 4                  |
| 5       | 54-MA-0356712-S        | 16.3     | 18.7  | 2.6     | 27     | 6                  |
| 6       | 75-AP-0330006-N        | 11.2     | 15.3  | 2.7     | 35     | 5                  |
| 7       | 58-OA-0330001-N        | 27.3     | 30.8  | 2.9     | 17     | 8                  |
| 8       | 52-KI-0330000-W        | 43.8     | 46.5  | 3.1     | 19     | 7                  |

single complete guideline for implementing pavement management than it is to write a simple, complete marriage manual that is directly applicable to all cases and all couples.

The following listing presents the major items in a summary implementation plan for major consideration:

1. Decision to start;
2. Preparation of goals, objectives, and preliminary budget;
3. Commitment from top management (usually network level first);
4. Preliminary work plan (form technical group);
5. Establishment of a steering committee (administration);
6. Development of a detailed work plan;
7. Evaluation of hardware and software needs;
8. Development of the preliminary system (network and level);
9. Testing and verification of the preliminary system;
10. Demonstration of the second-stage system;
11. Finding a home for the PMS group in the organization;
12. Acceptance of the PMS for implementation;
13. Routine operation of the system; and
14. Improvement, upgrading, and maintenance of the PMS.

Note that this summary includes 11 items to be accomplished prior to routine operation of the system; these range from a decision to start to acceptance of the PMS for implementation. It may appear that some of the items are self-evident; however, making a specific overt decision at each of these points is extremely important. Each of the points is discussed in more detail elsewhere (7).

#### SUMMARY AND SUGGESTED RESEARCH

The experience and research presented in this paper point to a continued need for PMS development and implementation. This may sound all too familiar to the reader, but in this case much remains to be done. As highway budget levels shrink in the face of inflationary cost increases and growing traffic levels, it will be increasingly important to apply systematic pavement management in a comprehensive, coordinated fashion.

Most PMS development and experience to date have occurred at the project level and within the areas of design or maintenance. Development has focused quite naturally and understandably on these areas, and this concentration of effort has produced significant results.

Currently, the major concerns in pavement management lie at the network level. Consequently, the greatest current need is for a focusing of research effort at the network level. This paper has ad-

ressed network-level PMS development, but substantial additional effort is required to "catch up" with project-level technology. This effort would include extensive development of network-level subsystems and prediction models as well as trial implementation of network-level systems and combined project- and network-level systems.

A PMS useful in decisionmaking is a highly specific tool that is particularly structured to the attitudes and procedures of the implementing agency. Consequently, many of the details of a PMS must be fitted or molded directly to the implementing agency. Nevertheless, significant portions of the development and implementation work involved in setting up and operating a PMS are potentially applicable to a wide variety of users. This type of work is most suitable for National Cooperative Highway Research Program funding.

Thus, two general types of effort are needed for comprehensive PMS development: (a) the detailed work that must be done by individual states and (b) the more general development that can be effectively carried out on a common national scale. A plan for future research of general applicability, including suggested research activities at both the project and network levels and estimated costs, is translated into a specific set of problem statements in the Appendix of the report by Pedigo, Roberts, and Hudson (8).

#### ACKNOWLEDGMENT

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# Combined Priority Programming of Maintenance and Rehabilitation for Pavement Networks

RALPH HAAS AND ALAN CHEETHAM

Total expenditures for maintenance and rehabilitation of a pavement network should represent an optimum combination of the two types of activities. Under the usual situation of budget constraints, this requires the establishing of priorities. An integrated method for programming maintenance and rehabilitation for paved road networks for any chosen program period is described. It begins with a common inventory of condition, serviceability, structural adequacy, traffic, unit costs, and other information. The maintenance programming subsystem evaluates alternative treatments for different types, densities, and severities of distresses and produces a demand-based budget by using a maximization of cost-effectiveness. The rehabilitation programming subsystem similarly evaluates alternatives, and a priority list of year-by-year projects over the program period, based on benefit maximization, is produced. The total of maintenance and rehabilitation costs for any given year does not exceed the total budget limit. A case application is provided to illustrate the method. It uses the arterial street network of a small city, subdivided into 100 sections. The outputs include section-by-section, year-by-year, recommended programs of maintenance and rehabilitation work. An additional feature of the method is a capability for evaluating the long-term effect of various budget options on average network serviceability. Two rehabilitation budget levels, representing the expected funding and a zero budget, have been tested for the case application. As expected, average network serviceability was estimated to decrease significantly over the 10-year programming period for the zero budget case. Finally, it is recommended that year-by-year updates be carried out on the inventory and the maintenance and rehabilitation programs.

Should we be spending more money on maintenance and less on rehabilitation, or vice versa? What is the optimal combination of expenditure for maintenance and (capital) rehabilitation to get the best possible value for the total available funds?

The overall objective of a public agency should be to obtain such a total best value. However, it is common practice to separately determine maintenance and rehabilitation needs and prepare separate budgets that are separately administered. Coordination between the two is on a judgment basis. Quantitative answers to whether the individual programs and budgets for each represent the best balance cannot easily be answered with present methodology.

A better approach is one that starts with a coordinated or combined inventory and inspection to establish the present status of the network. Maintenance needs and rehabilitation needs can then be identified by comparing this present status with policy variables such as minimum acceptable levels of serviceability and maximum levels of surface distress, and initial, "demand-based" budgets can be established. However, because the required funding is not usually available, various maintenance and rehabilitation alternatives, including deferral of the work, should be considered for each need and their cost and benefit implications should be analyzed. Then a final priority set of maintenance and rehabilitation alternatives and their timing are chosen by varying the individual budgets, within a total budget limit, to determine which combination gives the highest average serviceability for the network as a whole.

The purpose of this paper is to describe a working procedure for obtaining coordinated priority programs of maintenance and rehabilitation for a paved road network. An example application to the arterial network of a small city is provided to illustrate the procedure.

## FRAMEWORK FOR COORDINATING BUDGETING AND PROGRAMMING

Maintenance and rehabilitation expenditures should be programmed simultaneously to avoid incompatibilities in an operational sense and to approach a truly optimum allocation of available funds. Figure 1 shows an ideal framework for accomplishing this in a coordinated manner.

Three levels of budgeting and programming have been identified in this framework. The first level represents a "first cut" at the programming of pavement investments. The starting point is to conduct an inventory and to establish the present status of the system and the needs by considering policy variables such as acceptable levels of service. From these needs, it is possible to establish a "demand budget" that can then be compared with the total available funds.

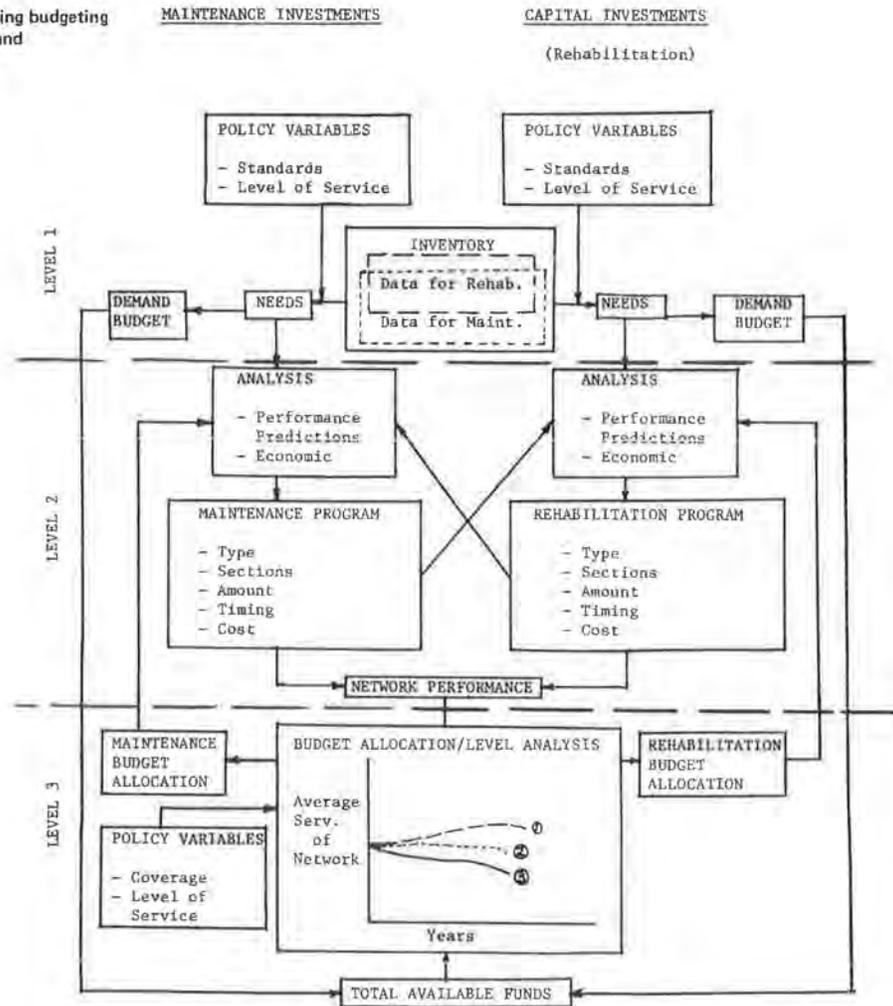
The types of inventory information required are simply categorized as to maintenance or rehabilitation use in Figure 1 and are listed in more detail in Figure 2. Some of this information is specific to the type of programming. For example, structural adequacy is not usually considered in programming maintenance, since maintenance does not normally deal with strengthening operations per se. Although there is considerable overlap in the types of information acquired, the intensity of the data or the specific parameters of concern may be different. For example, programming rehabilitation may require an indication of the overall condition of a pavement surface in the form of, say, a condition rating or index; on the other hand, programming maintenance requires specific details of the type, amount, and severity of distress, since this has a bearing on the type of maintenance activities that may apply.

The second level of programming involves the establishment of programs for maintenance and rehabilitation based on the needs identified in the first level plus analyses of the performance and economics of various strategies for fulfilling these needs. These strategies include not only the rehabilitation and maintenance alternatives but also the timing of these alternatives.

The determination of the budget allocations for maintenance and rehabilitation programs as well as the overall pavement budget levels for each year in the programming period are considered in the third programming level. By varying the total funds available in each year and also by altering the allocation ratio between maintenance and rehabilitation budgets, the analyses of level 2 can be iterated under different budget constraints to produce different profiles of average overall network serviceability with time. This allows policy planners to study the effect of budget levels and allocations on the performance of the pavement network and, by considering another set of policy variables as decision criteria, arrive at the most suitable budget allocations.

The relation of most current maintenance management systems to the framework of Figure 1 should be noted. Although these systems have generally achieved a high state of development, they only really take over after the programming has been done—i.e., they represent a "production control"

Figure 1. Framework for coordinating budgeting and programming for maintenance and rehabilitation.



type of process. The current methodology for programming of maintenance investments consists essentially of establishing maintenance needs and developing the associated demand budget as shown in level 1. The establishment of needs is based on attaching average unit volumes, weights, rates of application, times, (depending on type of maintenance activity) for different types of surfaces and road classes and then working out the total volumes or weights of materials and equipment and person hours required. These average volumes, weights, rates, etc., are themselves based on past maintenance management records.

#### WORKING METHOD

The following sections present a working method for the framework shown in Figure 1.

#### Pavement Inventory and Maintenance Alternatives

Good inventory information (Figure 1), both acquired (such as traffic and unit costs) and field measured (such as roughness), is the foundation for effective programming of maintenance and rehabilitation. Of these, the surface condition survey (Figure 2) is perhaps the key item for maintenance. With information from the survey on the types, extent, and severity of surface distresses, the most cost-effective maintenance alternatives can be identified.

One of the most up-to-date approaches for ac-

complishing this has been developed by the Ontario Ministry of Transportation and Communications (1-3). This approach has the following major elements: (a) a condition survey to identify the types, severity, and density of distresses; (b) the alternative maintenance treatments available and their expected lives for each combination of type, severity, and density of distress; and (c) an economic analysis to determine the most cost-effective treatments. Such an analysis makes it possible to prepare a direct demand budget.

Figure 3 (2) shows how alligator cracking (one of the 12 types of example distresses in the Ontario method) is described and how its severity is established and its density is determined. Photographs (not included in this paper) are provided in the method to assist field personnel in getting the right answers and being consistent.

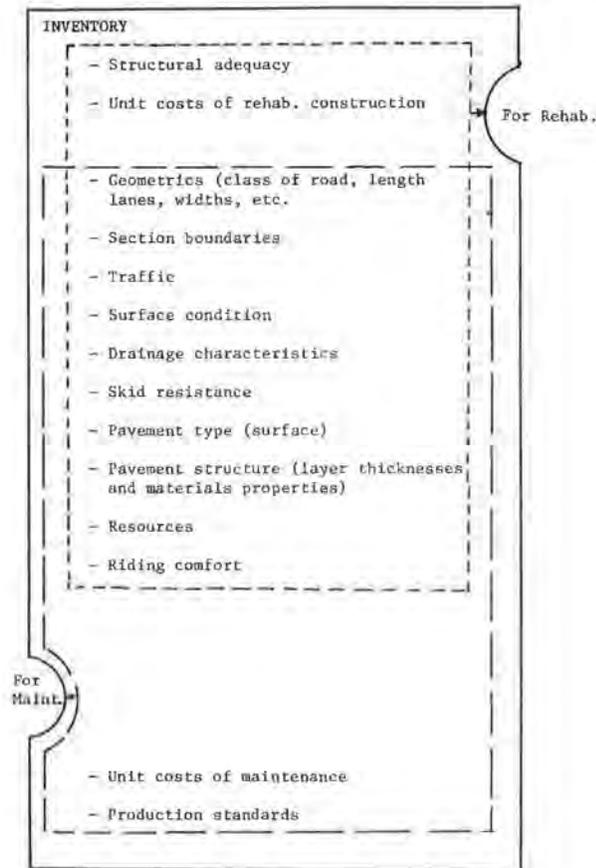
Table 1 gives the alternative treatments available and their expected effective lives. These lives, of course, apply to Ontario conditions and would have to be calibrated for other regions. Performance standards, available elsewhere (3), have also been developed for the treatment alternatives.

With cost information on the various items in the performance standards, plus the amount of maintenance to be performed, unit cost calculations can first be performed for each treatment alternative:

$$\text{Unit cost} = (\text{manpower} + \text{equipment} + \text{materials}) / \text{accomplishment per day}$$

(1)

Figure 2. Types of inventory information required for maintenance and rehabilitation use.



Then equivalent annual cost calculations can be made:

$$\text{Equivalent annual cost} = \frac{\text{unit cost/expected life of alternative}}{\text{in years}} \quad (2)$$

The most cost-effective alternative is the one with the least equivalent annual cost.

A quantitative end-product illustration of the foregoing procedures is provided later in this paper.

Pavement Inventory and Rehabilitation Alternatives

The inventory information for rehabilitation programming includes the acquired data items in Figure 2 (such as traffic and unit costs), the condition survey previously noted for maintenance programming, and several additional key items from field measurements. These include structural adequacy, riding comfort and skid resistance, and structural composition (from coring) if construction records are inadequate.

Although information on the individual data items should be retained, a composite "pavement quality index" (PQI), on a scale of 0-10, has been found quite useful for rehabilitation programming (4). This uses the Canadian riding comfort index (RCI), a structural adequacy rating (SAR) from deflection survey measurements, and a condition index (CI) from the condition survey measurements. RCI, SAR, and CI are all based on a scale of 0-10.

Rehabilitation alternatives may include overlays of varying thickness, full or partial reconstruction (including recycling), and surface treatment. The set of alternatives available will vary with the

Figure 3. Example of distress-type description, severity, and density.

Alligator Cracking

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

network considered and the jurisdiction involved.

Maintenance-Rehabilitation Programming System

The general structure of the maintenance-rehabilitation programming system is shown in Figure 4, including the two main subsystems. The entire process has been computerized, and a detailed description, including the various subsystems, is given by Cheetham (5).

The first step in operating the system is to collect the previously noted inventory data. Together with the available maintenance and rehabilitation alternatives and their associated unit costs, analyses are then performed to select the most cost-effective maintenance alternatives. In addition, through analyses of performance and a benefit maximization model, an optimized priority program of rehabilitation is determined. This priority programming method for rehabilitation has been extensively applied and is described by Karan and others (6-9). Typical outputs of the system are provided in the case illustration given later in this paper. As an alternative, Cheetham (5) has developed a ranking method, based on cost-effectiveness analysis, for selecting a network priority program of rehabilitation. Although it represents an approximation, it is very efficient with respect to computer time and gives essentially the same results as the optimization method (6-9). But it should be periodically calibrated with an optimization run.

The maintenance programming subsystem analyzes the pavement network one section at a time. Figure 5 outlines the subsystem, which has separate sub-routines for the different types of pavements.

The network inputs include the available maintenance alternatives for each pavement type and their associated unit costs, the number of sections in the network, and the year of analysis. The sectional inputs that are read in the main line include the section number, pavement type, PQI, average annual daily traffic (AADT), traffic growth rate, number of lanes, lane width, section length, surface thickness, and drainage condition.

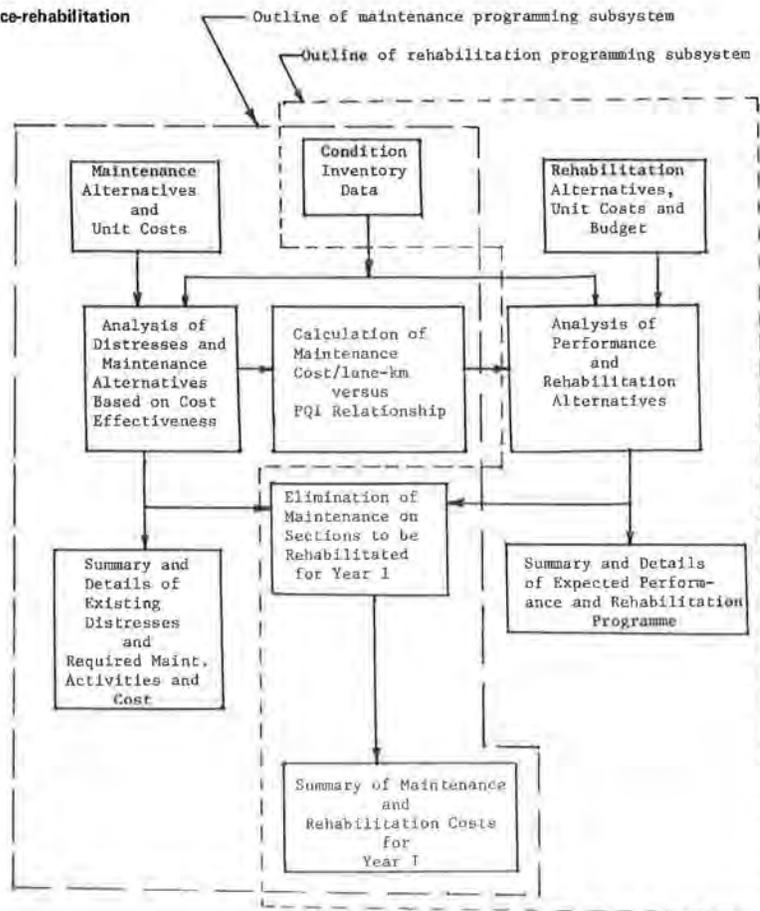
Each section can be analyzed as a whole or, for ease of condition inventory data collection, broken into subsections of constant length called "stations". Once all of the sections in the network have been analyzed, total costs are calculated for

**Table 1. Example of treatment alternatives for particular distress type: alligator cracking.**

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

<sup>a</sup>Contract only.

**Figure 4. General structure of maintenance-rehabilitation programming system.**



each of the maintenance activities and a summary of the network costs is produced. A maintenance cost versus serviceability relation is then calculated by using regression analysis.

An outline of the maintenance subroutine for flexible pavements is shown in Figure 6. The inputs

from the main line include the available maintenance alternatives and their associated unit costs, AADT, number of stations and station length, number of lanes in each station, number of lanes in the section, lane width, and surface thickness. The distress inputs that are read into the subroutines

Figure 5. Basic structure of maintenance programming subsystem.

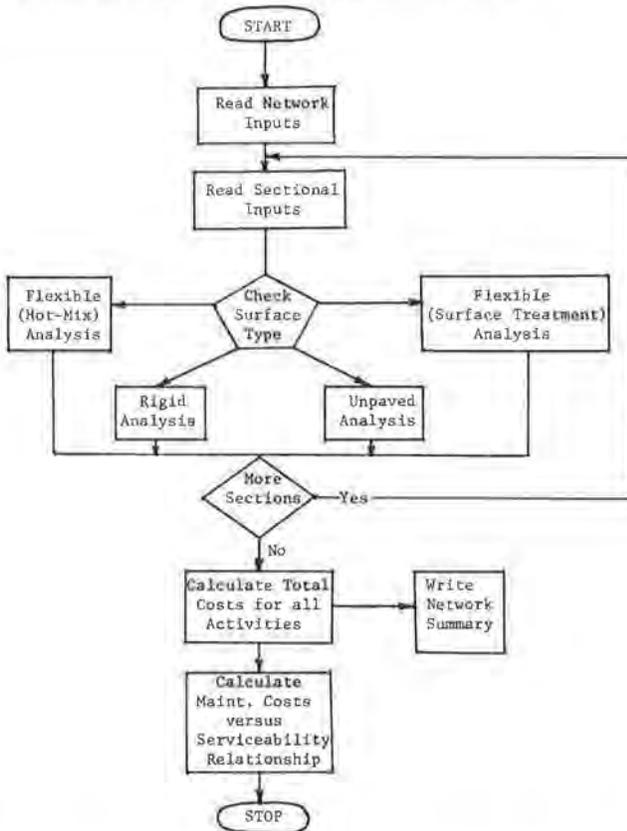
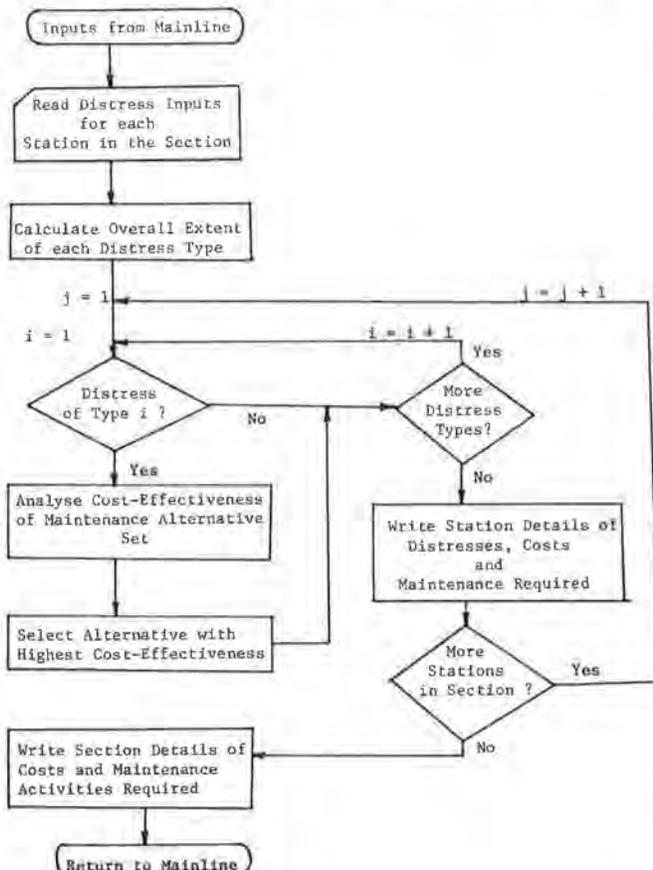


Figure 6. Outline of subroutines for maintenance analysis of flexible pavements.



include two vectors for each station: one that indicates the severity of each distress type and one that gives the extent of each type of distress in terms of an areal percentage.

The stations are then analyzed one at a time, and the most cost-effective alternative is selected for each distress in each station. Details of distresses, required maintenance, and costs are written for each station for the section.

The program considers 12 distinct types of distress for both flexible pavement subroutines (hot mix or asphalt concrete surface and surface treatment). These 12 distresses are subsequently listed in the case example. The areal units used for extent of distress are either percentage area or percentage length, depending on the distress type. Severity codes are also subsequently listed in the case example.

Subroutines for rigid pavements and for unpaved roads are included in the maintenance programming subsystem but are not covered in this paper.

The number and type of maintenance treatment alternatives to be considered vary with pavement type and distress type. These are described in detail elsewhere (3,5); an example of the alternatives available for alligator cracking has been given in Table 1.

The rehabilitation programming subsystem uses a rationally based ranking factor for prioritization but retains the performance prediction method described by Karan and others (6-9). The ranking factor was developed from a "calibration" procedure that used the optimization results of various network analyses carried out for Canadian and U.S. jurisdictions according to the method of Karan and others (6-9). This method includes benefit maximization through a linear programming formulation. The ranking factor is a function of minimum acceptable serviceability level, initial serviceability level, AADT, and length of section. It maximizes average network serviceability over the program period. Figure 7 shows the general structure of the rehabilitation programming subsystem.

The rehabilitation subsystem uses a Markov chain model of order 1 (8) to model the performance. The model describes the pavement as existing in a current "state", and subsequently the pavement undergoes transitions to lower states during successive time periods. The state is defined by the level of serviceability, and the time increment used is one year. Since the transition from one state to another is a stochastic process, the model requires transition probability matrices to function.

The model also requires the definition of pavement classes in order to include factors that affect performance, such as pavement thickness, subgrade strength, and traffic volume. The program defines 18 pavement classes, each of which requires a different transition probability matrix. The 18 classes are defined by three levels of pavement thickness (equivalent granular thickness for the structure), three levels of traffic (AADT), and two levels of subgrade type (strong and weak). The transition probability matrices have been developed under the assumption that "routine" maintenance is applied to the pavement.

Transition probability matrices are also required for each of the rehabilitation alternatives.

The rehabilitation programming subsystem has the capability of analyzing six different rehabilitation alternatives. These include three thicknesses of overlay, reconstruction, upgrading, and surface treatment. In addition, the program has the capability of analyzing sections with paved or unpaved shoulders and the addition of paved shoulders to sections that do not have them. The program allows

Figure 7. General structure of rehabilitation programming subsystem.

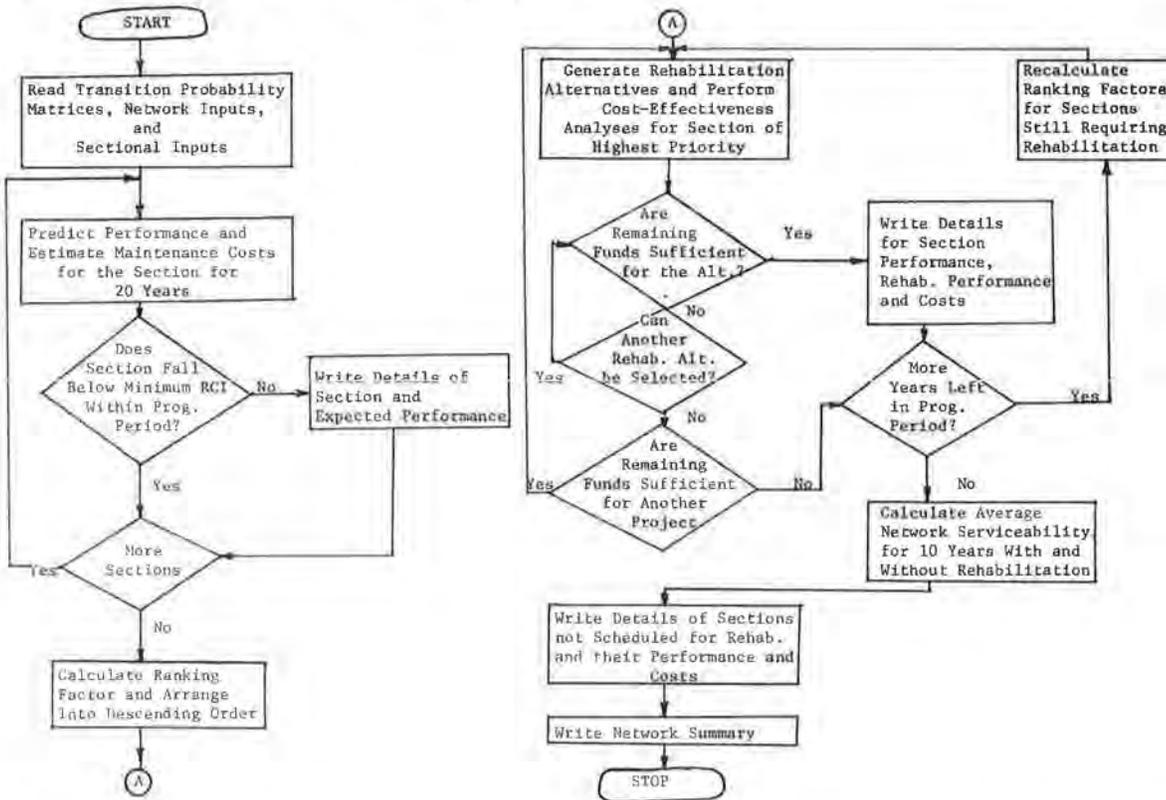
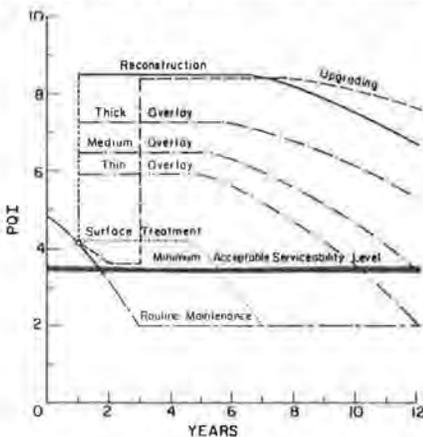


Figure 8. Typical performance curves for rehabilitation alternatives.



three reconstruction designs for different levels of traffic.

The upgrading alternative is a relatively new innovation and is based on the idea of "betterments" used in some agencies. This is a form of staged rehabilitation that, because of various constraints, can only be done on certain sections. Some agencies use a form of upgrading on certain rural sections that have a low level of serviceability but cannot be rehabilitated by using one of the usual alternatives because of limited funds. The upgrading or betterment strategy involves work over approximately three years, the first two of which are relatively low cost. This strategy involves shoulder and drainage work in the first year, the application of a few inches of granular material and a surface treatment in the second, and an overlay in the third

year. Since so much new material is added to the structure, this alternative can only be considered on sections that do not have height limitations due to curbs or utility services.

The sectional inputs to the program indicate which alternatives are to be considered for that section. Each desired strategy is then analyzed, and its subsequent performance and associated costs are predicted. Figure 8 shows typical performance curves for the six basic alternatives.

After the performance analysis of each rehabilitation alternative, the cost-effectiveness is determined. This is done by calculating the present worth of all of the costs involved with each alternative; i.e., the maintenance costs plus capital costs of rehabilitation. Then the area between the two performance curves (performance for the rehabilitation and performance for only routine maintenance) is calculated as a measure of the associated benefits. The cost-effectiveness of a given alternative can then be calculated by dividing the area between the curves by the present worth of the costs. The alternative with the highest cost-effectiveness is then selected for implementation since it makes the best use of the available funds.

In the case where insufficient funds remain in the budget to complete the project, other alternatives are compared with the remaining funds. Since budget estimates are usually approximate, a project will still be programmed for implementation if the remaining funds cover 95 percent of the cost. If sufficient funds remain to complete a rehabilitation project on a different section but not on the section being considered as top priority at the time, then other sections are considered and the process is repeated.

The programming subsystem can operate in one of two modes, depending on the number of budgets used by the particular agency. If two separate budgets

are used (one for maintenance and one for rehabilitation), then the maintenance costs, although used in the cost analyses, are not subtracted from the total budget. In the case of a single budget, the maintenance costs are subtracted from the budget.

Two modes of operation are also available for the use of any remaining funds in the budget after projects are scheduled for rehabilitation in a given year. Since fractional projects are not considered in the system (as in linear programming), there is usually some amount of money remaining for a given year's budget. One mode of operation allows the remaining funds to be added to the budget for the following year, and the other mode simply leaves the remaining funds as extra. This choice of mode again depends on the agency involved and exists because of the approximate nature of future budgets.

CASE ILLUSTRATION OF THE SYSTEM

The City of Cambridge in Ontario provides a good case illustration for the maintenance-rehabilitation programming system. The arterial street network was subdivided into 100 sections (see Figure 9 for typical listings), and a condition survey was conducted according to Ontario procedures (1). Other field measurements included a deflection survey, using a Dynaflect, at an average of 6 tests/km, a roughness survey using the ARAN unit (10), and cores, at a minimum of one per section, for structural composition and subgrade characteristics. In addition, data were acquired on traffic volumes, unit costs for the various maintenance and rehabilitation al-

ternatives, etc. Figure 10 shows an example of the input data, and the two tables below provide unit cost data used in the analysis:

| Activity          | Unit Cost (\$/unit) |
|-------------------|---------------------|
| Spray patch       | 1.15/m <sup>2</sup> |
| Sand or chip seal | 1450.0/lane-km      |
| Hot-mix patch     |                     |
| Manual            | 1.75/m <sup>2</sup> |
| Machine           | 1.20/m <sup>2</sup> |
| Cold-mix patch    |                     |
| Manual            | 2.35/m <sup>2</sup> |
| Machine           | 1.65/m <sup>2</sup> |
| Deep patch        | 6.80/m <sup>2</sup> |
| Crack seal        | 0.60/linear m       |
| Surface treatment | 1574.50/lane-km     |

| Alternative       | Design   | Unit Cost (\$/lane-km) |
|-------------------|--|------------------------|
| Overlay           |  |                        |
| 1                 | 19-mm dense AC   | 7 686.00               |
| 2                 | 38-mm HL3 AC   | 8 198.40               |
| 3                 | 51-mm HL4 AC   | 8 820.60               |
| Reconstruction    | 38-mm HL3 AC, 51-mm HL6 AC, 152-mm granular A, 305-mm granular B | 56 766.60              |
| Surface Treatment | Single application, 6-mm stone, emulsion binder                  | 1 574.50               |

The system requires only entry of the inventory data and selection of various options (i.e., budget

Figure 9. Section identification.

| SEC. NO. | NAME            | FROM                          | TO                            | LENGTH M |
|----------|-----------------|-------------------------------|-------------------------------|----------|
| 5        | AVENUE ROAD     | 1 MILE EAST OF FRANKLIN BLVD. | TOWN LINE ROAD                | 1060     |
| 6        | AVENUE ROAD     | FRANKLIN BLVD.                | 1 MILE EAST OF FRANKLIN BLVD. | 1620     |
| 54       | GLENMORRIS ST.  | FIFTH AVE.                    | CULHAM DR.                    | 200      |
| 55       | GLENMORRIS ST.  | CULHAM DR.                    | CITY LIMITS                   | 1020     |
| 59       | FIRST AVE.      | GLENMORRIS ST.                | BORDEN ST.                    | 500      |
| 403      | NORFOLK AVE.    | BRUCKLYN RD.                  | JARVIS ST.                    | 400      |
| 404      | NORFOLK AVE.    | BRUCKLYN RD.                  | SAMUELSON ST.                 | 780      |
| 405      | ELGIN ST. NORTH | SAMUELSON RD.                 | AVENUE RD.                    | 1020     |

Figure 10. Section input data for analysis.

| SECTION NO. | PVT. THK. | SUBG. TYPE | AACT | TGR   | LANES | LENGTH (M) | PQI | MINPQI |
|-------------|-----------|------------|------|-------|-------|------------|-----|--------|
| 5           | 28        | 1          | 600  | 1.05  | 2     | 1060       | 4.3 | 4.0    |
| 6           | 30        | 1          | 770  | 1.05  | 2     | 1620       | 5.9 | 4.0    |
| 54          | 38        | 1          | 1700 | 1.01  | 2     | 200        | 6.0 | 4.0    |
| 55          | 28        | 1          | 1700 | 1.01  | 2     | 1020       | 5.8 | 4.0    |
| 59          | 20        | 1          | 610  | 1.005 | 2     | 500        | 3.7 | 4.0    |
| 403         | 20        | 1          | 2660 | 1.01  | 2     | 400        | 6.0 | 4.0    |
| 404         | 25        | 2          | 2090 | 1.01  | 2     | 780        | 4.3 | 4.0    |
| 405         | 20        | 1          | 6880 | 1.02  | 2     | 1020       | 7.0 | 4.0    |

NOTES:

- 1 : Pavement thickness: total equivalent granular thickness of structure (cm.)
- 2 : Subgrade Type: 1 = strong; 2 = weak
- 3 : Traffic Growth Rate: i.e. 1.05 means 5 percent
- 4 : Pavement Quality Index: Scale of 0 to 10 (Ref. 4)
- 5 : Minimum Acceptable PQI

limits); operation of the entire system is automated and includes the following output reports:

1. Section-by-section summary of maintenance and rehabilitation costs--Figure 11 shows the final results for the case application, for 1980, where the most cost-effective maintenance program would cost \$22 444 and the optimum rehabilitation program would cost \$100 969. This latter number represents the maximum budget available for rehabilitation.
2. Overall network maintenance summary--Figure 12 shows the results for the case application, for 1980, for the situation of no rehabilitation expenditures. It may be noted for this situation that the cost of the most cost-effective maintenance program would rise to \$27 667 compared with the \$22 444 in Figure 11. The larger sum is of course due to maintenance being required in 1980 on all sections.
3. Detailed record of section distresses and maintenance requirements--Figure 13 shows an example for section 55 in the network of the case application. The record of distresses comes directly from the condition survey, and the maintenance shown is the most cost-effective. The recording of distresses and costs by station, within the section, allows persistent problem areas to be identified and simplifies the collection of distress data in the condition survey.
4. Detailed record of section rehabilitation--

Figure 14 shows an example for section 404. The optimum rehabilitation alternative is overlay 3 (51 mm), in 1981, for the budget level used. It should be noted that rehabilitation may be performed when the section reaches its minimum acceptable PQI level (i.e., in the year of need) or it may be accelerated or delayed. This timing is determined through use of the optimization model, which also determines the particular alternative. Both depend on the budget level specified for the optimization. Figure 14 also lists the expected PQI for 10 years for the section for two situations: (a) rehabilitation carried out according to schedule and with the alternative specified and (b) no rehabilitation (i.e., application of regular maintenance only). The latter case, as expected, would result in the section deteriorating to a PQI level of 2.5 at the end of 10 years. These expected performances are based on the performance prediction models incorporated in the system.

A summary of the five-year recommended priority program of rehabilitation, by section and rehabilitation type, is provided in Table 2.

Figure 15 compares the recommended sections of Table 2 with the needs (i.e., when the sections reach or fall below the minimum acceptable PQI level). For example, section 404 has been "accelerated" for rehabilitation (i.e., rehabilitation does not become a need until 1982 but the section is

Figure 11. Maintenance-rehabilitation cost summary for 1980.

| SECTION     | TYPE | PQI | MAINT. COST | REHAB. COST |
|-------------|------|-----|-------------|-------------|
| 5           | 2    | 4.3 | 122.        |             |
| 6           | 2    | 5.9 | 508.        |             |
| 7           | 1    | 1   | 1           |             |
| 8           | 1    | 1   | 1           |             |
| 54          | 1    | 8.0 | 546.        |             |
| 55          | 1    | 5.8 | 1644.       |             |
| 59          | 1    | 3.7 | 723.        |             |
| 1           | 1    | 1   | 1           |             |
| 1           | 1    | 1   | 1           |             |
| 403         | 1    | 6.0 | 70.         |             |
| 404         | 1    | 4.5 | 1303.       |             |
| 405         | 1    | 7.0 | 158.        |             |
| 406         | 1    | 7.0 | 0.          |             |
| 407         | 1    | 2.2 |             | 9879.       |
| 408         | 1    | 5.0 | 15.         |             |
| 409         | 1    | 4.4 |             | 10938.      |
| TOTAL COSTS |      |     | 22444.      | 100969.     |

Figure 12. Network maintenance cost summary for 1980 for no rehabilitation.

THE SECTION TYPES ARE

1 : FLEXIBLE (HOT-MIX SURF.)  
 2 : FLEXIBLE (SURFACE-TREAT. SURF.)  
 3 : RIGID (PORTLAND CEMENT CONCRETE)  
 4 : UNPAVED (GRAVEL)

|   |               |             |
|---|---------------|-------------|
| NO. OF SECTIONS =   | 100           | ( 53.6 KM ) |
| NO. OF SECTIONS BY SECTION TYPE :   |               |             |
| FLEXIBLE, HOT-MIX SURFACE :   | 93            | 46.3 KM     |
| FLEXIBLE, SURFACE-TREATMENT SURF. :   | 7             | 7.3 KM      |
| TOTAL EXPECTED MAINTENANCE COST =   | 27667.59      |             |
| NOTE : THIS TOTAL INCLUDES ONLY ROUTINE MAINTENANCE PERFORMED ON THE PAVEMENT SURFACE. IT DOES NOT INCLUDE WINTER, BRIDGE, SIGNS OR OTHER MAINTENANCE ACTIVITIES. |               |             |
| MAINTENANCE COST SUMMARY BY ACTIVITY FOR THE NETWORK :  |               |             |
| ACTIVITY  | EXPECTED COST |             |
| SPRAY PATCH   | 4282.70       |             |
| SAND- ON CHIP-SEAL  | 788.80        |             |
| HOT-MIX PATCH (MANUAL)  | 18959.87      |             |
| HOT-MIX PATCH (MACHINE)   | 2197.77       |             |
| COLD-MIX PATCH (MANUAL)   | 428.85        |             |
| CRACK SEAL  | 1009.80       |             |

Figure 13. Example of detailed record of section distresses (from condition survey) and maintenance required for 1980.

NOTE: THE STATION-DISTRESS MATRIX FOR EACH SECTION SHOWS THE SEVERITY LEVEL AND EXTENT OF EACH DISTRESS TYPE FOR THE GIVEN STATION.

THE DISTRESS TYPES ARE AS FOLLOWS.

FLEXIBLE (HOT-MIX OR SURF-TREAT) : 1 = RAVELLING / STREAKING  
 2 = FLUSHING  
 3 = SLIPPERY SURFACE  
 4 = PCT HOLES  
 5 = RIPPLING / SHOYING  
 6 = BUTTING  
 7 = DISTORTION  
 8 = EXCESSIVE CROWN  
 9 = LONG. & TRANS. CRACKING  
 10 = MAP CRACKING  
 11 = PROGRESSIVE EDGE BREAKING  
 12 = ALLIGATOR CRACKING

THE SEVERITY LEVELS ARE : 0 = NO DISTRESS  
 1 = SLIGHT  
 2 = MODERATE  
 3 = SEVERE

AS DEFINED IN THE M.T.C. GUIDELINES.

SECTION 55

PAVEMENT TYPE = 1                      PRESENT RCI = 5.8  
 NO. OF LANES = 2                      LENGTH = 1020. (METRES)  
 AADT = 1700.                          TRAFFIC GROWTH RATE = 1.00 (%)  
 DRAINAGE CONDITION = 0  
 STATION LENGTH = 102 (METRES)

| STATION | DISTRESSES |   |   |   |   |   |   |   |   |    |    |    | COST |   |   |   |   |           |       |
|---------|------------|---|---|---|---|---|---|---|---|----|----|----|------|---|---|---|---|-----------|-------|
|         | 1          | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 |      |   |   |   |   |           |       |
| 1       | 0          | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0  | 1  | 1  | 0    | 0 | 0 | 0 | 0 | 0.0       |       |
| 2       | 0          | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0  | 0  | 0  | 0    | 0 | 1 | 1 | 0 | 2-15 2-15 | 266.2 |
| 3       | 0          | 0 | 0 | 0 | 0 | 1 | 1 | 0 | 0 | 0  | 0  | 0  | 0    | 0 | 1 | 2 | 0 | 2-20 3-10 | 254.9 |
| 4       | 0          | 0 | 0 | 0 | 0 | 1 | 2 | 0 | 0 | 0  | 0  | 0  | 0    | 0 | 0 | 0 | 0 | 2-15 3-15 | 280.3 |
| 5       | 0          | 0 | 0 | 0 | 0 | 1 | 1 | 0 | 0 | 0  | 0  | 0  | 0    | 0 | 0 | 0 | 0 | 2-5 2-10  | 149.3 |
| 6       | 0          | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0  | 0  | 0  | 0    | 0 | 0 | 0 | 0 | 2-4 0-0   | 28.2  |
| 7       | 0          | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0  | 0  | 0  | 0    | 0 | 1 | 0 | 0 | 0-0 0-0   | 0.0   |
| 8       | 0          | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0  | 0  | 0  | 0    | 0 | 0 | 0 | 0 | 3-20 0-0  | 146.9 |
| 9       | 0          | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0  | 0  | 0  | 0    | 0 | 0 | 0 | 0 | 3-25 0-0  | 183.6 |
| 10      | 0          | 0 | 0 | 0 | 0 | 1 | 1 | 0 | 0 | 0  | 0  | 0  | 0    | 0 | 0 | 0 | 0 | 3-30 3-10 | 334.5 |

| REQUIRED MAINTENANCE                              | EXPECTED COST |
|---|---------------|
| SPRAY PATCH                                       | 450.43        |
| HOT-MIX PATCH ( MANUAL )                          | 462.60        |
| HOT-MIX PATCH ( MACHINE )                         | 550.80        |
| <b>TOTAL MAINT COST FOR THE SECTION = 1463.83</b> |               |

Figure 14. Example of detailed section rehabilitation requirements for 1981.

SECTION NO. = 404

AAAT = 2090                      TRAFFIC GROWTH RATE = 1.070  
 LENGTH = 0.78 (KM)              NO. OF LANES = 2  
 EQUIV. GRAN. PAVEMENT THICKNESS = 25 CM.      SUBGRADE TYPE = 2  
 MIN. ACCEPTABLE SERVICEABILITY LEVEL = 4.0

RECOMMENDED REHABILITATION ALTERNATIVE : 3 (TRIPLE LIFT OVERLAY) IN 1981

EXPECTED 10-YEAR PERFORMANCE (PQI) WITHOUT REHABILITATION :  
 4.3    4.2    4.0    3.7    3.5    3.3    3.1    2.9    2.7    2.5

EXPECTED 10-YEAR PERFORMANCE (PQI) WITH REHABILITATION :  
 4.3    1.2    1.2    1.2    1.2    1.2    1.3    1.1    1.0    0.8

scheduled for rehabilitation in 1981). By comparison, other sections, such as section 59, have had their rehabilitation delayed. These accelerations or deferrals, and the actual rehabilitation type specified in Table 2, are a function of the budget level used, among other factors such as traffic volume, and are determined by the optimization model in the system. Higher budget levels would result in fewer deferrals, to some point where all the needs could be satisfied in the year in which they occur. Most, if not all, highway agencies, however, are faced with limited budgets.

Figure 16 shows the results of a budget analysis for the case illustration. Two budget levels have been chosen: \$100 000/year for rehabilitation and \$0/year for rehabilitation. The first level represents that used in the analysis and would result in the program given in Table 2. With such a budget, the network would remain at a nearly constant PQI of 7 over the first 5 years and this would drop only marginally over the last 5 years. However, with no funds available at all for rehabilitation, the average PQI of the network will drop to near 5 at the end of 10 years. A histogram of the distribution of

Table 2. Recommended sections and rehabilitation alternatives for Cambridge, Ontario, network.

| 1980    |             | 1981    |             | 1982    |             | 1983    |             | 1984    |             |
|---------|-------------|---------|-------------|---------|-------------|---------|-------------|---------|-------------|
| Section | Alternative |
| 153     | 3           | 59      | 3           | 6       | 3           | 5       | 3           | 110     | 3           |
| 254     | 1           | 60      | 3           | 153     | 3           | 16      | 3           | 214     | 3           |
| 255     | 3           | 64      | 3           | 254     | 3           | 55      | 5           | 252     | 3           |
| 407     | 3           | 78      | 3           | 405     | 1           | 259     | 3           | 255     | 3           |
| 409     | 3           | 103     | 3           | 406     | 1           | 262     | 5           | 257     | 5           |
| 653     | 3           | 106     | 1           | 578     | 3           | 304     | 3           | 267     | 3           |
| 803     | 3           | 252     | 3           | 803     | 3           | 407     | 3           | 402     | 3           |
|         |             | 404     | 3           |         |             | 655     | 3           | 403     | 5           |
|         |             | 408     | 3           |         |             | 801     | 5           | 552     | 3           |
|         |             | 601     | 3           |         |             |         |             | 553     | 3           |
|         |             | 602     | 3           |         |             |         |             | 652     | 5           |
|         |             | 806     | 3           |         |             |         |             | 654     | 3           |
|         |             | 901     | 3           |         |             |         |             | 806     | 3           |
|         |             | 951     | 3           |         |             |         |             |         |             |

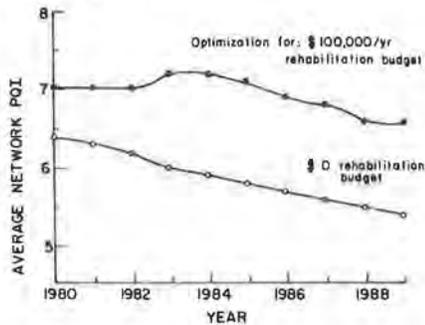
Note: Alternative 1 = 19-mm dense AC overlay, alternative 3 = 51-mm III.4 AC overlay, alternative 5 = surface treatment (single application, 6-mm stone, emulsion binder).

Figure 15. Comparison of recommended sections and needs.

| 1980  |        | 1981  |        | 1982  |        | 1983  |        | 1984  |        |
|-------|--------|-------|--------|-------|--------|-------|--------|-------|--------|
| Needs | Rehab. |
| 59    |        | 59    |        | 578   |        |       |        | 5     | 110    |
| 60    |        | 60    |        | 803   |        |       |        | 16    | 214    |
| 103   |        | 103   |        | 405   |        |       |        | 55    | 252    |
| 106   |        | 106   |        | 153   |        |       |        | 259   | 255    |
| 153   |        | 64    |        |       |        | 64    |        | 262   | 257    |
| 252   |        | 252   |        | 6     |        |       |        | 304   | 267    |
| 255   |        | 255   |        | 78    |        |       |        |       |        |
| 407   |        | 407   |        | 404   |        |       |        | 407   | 402    |
| 602   |        | 602   |        | 254   |        |       |        | 655   | 403    |
| 803   |        | 803   |        | 408   |        |       |        | 801   | 552    |
| 806   |        | 806   |        |       |        |       |        |       | 553    |
| 951   |        | 951   |        |       |        |       |        |       | 652    |
|       |        | 254   |        |       |        |       |        | 254   | 654    |
|       |        | 409   |        |       |        |       |        | 409   | 806    |
|       |        | 653   |        |       |        |       |        | 653   |        |
|       |        |       |        | 601   |        |       |        |       |        |
|       |        |       |        | 901   |        |       |        |       |        |

NOTES:  
 Needs: sections which fall below minimum acceptable PCI level.  
 Rehab: sections recommended for rehabilitation.  
 →: rehabilitation action delayed  
 ←: rehabilitation action advanced  
 ↔: rehabilitation action same year.

Figure 16. Analysis of rehabilitation budget level for 53.6-km network in Cambridge, Ontario.



PQIs at the end of 10 years would show 38 sections, or about 35 percent of the mileage, at or below the minimum PQI level of 4.0. The use of computer graphics to illustrate quickly the effects of various budget options (such as the two shown in Figure 16) and the associated accumulation of deficient mileage (i.e., at or below the minimum PQI level) is

a part of the rehabilitation subsystem and is more adequately illustrated elsewhere (4).

PERIODIC UPDATING

Although the system described in this paper can produce a 5- or 10-year program of work, periodic updating, preferably on an annual basis, is recommended. This is especially relevant for the maintenance program, whereas the rehabilitation update could be biannual.

Such updating involves two major aspects: update of the inventory and update runs of the maintenance and rehabilitation programs. The basic reasons for updating the inventory include the variations possible in predicting performance. The magnitude of uncertainty in predicting performance of course increases further along in the program period. Even next year's predictions of serviceability can be in considerable error if some unusual weather has occurred and/or if traffic volumes are significantly different from those estimated. Similarly, updates of the maintenance and rehabilitation programs are necessary (a) if the inventory reveals significant differences from earlier predictions, (b) if there have been unexpected changes in unit costs, and (c)

if the actual prices for last year's projects resulted in substantially more or less work being done than originally programmed.

#### SUMMARY AND CONCLUSIONS

This paper is based on the premise that maintenance and rehabilitation programming for pavement networks should be integrated in order to achieve the best possible total value for total funds available. A working method for accomplishing this objective has been presented and illustrated with a case study.

The working method starts with a common inventory of field measurements (e.g., condition survey, roughness, and structural adequacy) and acquired data (e.g., traffic and unit costs). Separate subsystems for maintenance programming and rehabilitation programming are included. These subsystems evaluate various maintenance treatment alternatives, for different distress types, densities, and severities, and rehabilitation alternatives for the various sections for the various years of the program period. The outputs are optimized programs of maintenance and rehabilitation whose total cost does not exceed the budget limit.

The case example, which uses the arterial street network of a small city, provides a quantitative illustration of the method. It also shows how the method can be used to test the effects of different budget levels on the future average serviceability of the network.

Finally, it is recommended that periodic updates of the maintenance and rehabilitation programs be carried out. This includes updating of the inventory.

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## Arizona Pavement Management System: Phase 2-- Verification of Performance Prediction Models and Development of Data Base

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A pavement management system has been defined as "the systematic development of information and procedures in optimizing the design and maintenance of pavements". Research conducted to verify and adjust performance prediction models (equations) developed during the course of research on a pavement management system in Arizona is described. The verification process involved testing models against real data and determining the correlation. Appropriate adjustments were made to enhance the final predictions. Results of this work indicate that the prediction models can reasonably predict the future ride and cracking condition for newly constructed, in-service, and overlaid asphaltic concrete pavements. The ability to predict future ride and cracking gives Arizona a powerful planning and programming tool.

A pavement management system (PMS) has been defined as "the systematic development of information and

procedures necessary in optimizing the design and maintenance of pavements" (1). Implementation of a PMS within the Arizona Department of Transportation (ADOT) has involved three phases:

1. Phase 1--Develop a program to optimize the design of new construction and major maintenance completed by Woodward-Clyde Consultants in 1976 (1);
2. Phase 2--(a) Verify prediction models with actual data and create a computerized data base, and (b) develop a functional PMS within ADOT (accomplished by ADOT staff by March 1981); and
3. Phase 3--Develop a network optimization system (developed by Woodward-Clyde Consultants and tested by ADOT staff).

Phase 2 and 3 projects represent a joint effort between ADOT and Woodward-Clyde Consultants. Information, highway condition data, and general overall direction of both projects were managed by a series of meetings between principal investigators. In addition, ADOT created a management steering committee composed of the following positions: chief deputy engineer (chairman), assistant state engineer traffic, priority program manager, maintenance engineer, materials engineer, and information systems project manager. This committee addressed important operational problems and recommended to the state engineer appropriate actions to be taken.

The purpose of this part of the phase 2 project was to verify and adjust existing models and develop a suitable data base for the use of the phase 3 program as well as design, maintenance, and management.

#### DESCRIPTION OF NEW MODELS

In phase 1, Woodward-Clyde Consultants developed pavement performance prediction models by using the Bayesian method (1). Models were created by interviewing knowledgeable highway engineers about their expectations of future pavement performance in terms of several variables. From these values mathematical models (equations) were developed.

The above represented ADOT and Woodward-Clyde Consultant's best approximation of future ride and skid number. During two and one-half years of using these equations, it became obvious that a percentage-cracking prediction model was needed as well as an improved ride model based on real data. The skid-number prediction model, although technically correct, was always predicting no future problem due to aggregate abrasion; nevertheless, serious low skid numbers did occur, evidently for other reasons. Generally, these reasons were related to uncontrollable factors at the construction site or maintenance activity. With these historical experiences in mind, it was decided in this project to develop prediction models for both roughness and percentage cracking. Skid numbers would not be predicted but rather monitored closely to determine those miles of highway in need of fix up. It is hoped that historical construction and maintenance data accumulated as part of this project will in the future be able to identify and correct the reasons for low skid numbers.

#### FACTORIAL DESIGN

Since the results of the phase 2 work would be incorporated into phase 3, discussions were held to set guidelines for the new prediction models. These guidelines included the following:

1. Models should be able to predict next year's ride and percentage cracking very accurately, since they would represent the condition at time of overlay. Any prediction errors are corrected in the following year by performing the annual monitoring.
2. Models should be able to predict reasonably well for a four to five-year time frame. This would fit into the five-year plan, which ADOT must compile and present to the ADOT commission and the Governor for approval each year.
3. Models should contain no more than five independent variables, preferably fewer. In this way, the size of the network problem could be kept within reason.
4. Models should predict in one-year increments.

With these guidelines, a fractional factorial

experiment was designed by Woodward-Clyde Consultants. Originally, only projects built since 1969 were going to be incorporated into the project. The year 1969 was chosen because in 1969 a new set of specifications was published and also the design of asphaltic concrete (AC) changed. Since it was not possible to fill more than half of the cells, the sample was changed to increase the time frame from 1963 to the present. The year 1963 was selected because it represented that time when the American Association of State Highway and Transportation Officials (AASHTO) Interim Guidelines (2) were put into practice. The selection process was widened to include any mile of highway built since 1963, and a mile could represent more than one cell as its condition changed with time. Unfortunately, the initial cell design was unsatisfactory in solving the problem. A substitute factorial scheme was devised. In this new scheme, region and time were divided into three levels, as given below:

| Factor                     | Level      | Value     |
|----------------------------|------------|-----------|
| Region (AASHTO)            | Desert     | 0-1.6     |
|                            | Transition | 1.7-3.0   |
|                            | Mountains  | 3.1-5.0   |
| Age of AC pavement (years) |            | 0-5.0     |
|                            |            | 5.1-10.0  |
|                            |            | 10.1-15.0 |

This produced nine combinations. For each combination, 15 different miles were randomly selected, which gave a total of 135 miles of new construction and 135 miles of overlays. Thus, each sample represented about 2.3 percent of the miles in the system. This was judged to be an adequate sample size. In addition, those miles where all data were present were also included. That is, if roughness, cracking, and deflection data were present for years 1973, 1975, and 1979, all of these years of data were included under the same milepost. The data included the following information:

1. Route number,
2. Direction,
3. Milepost,
4. Cell number,
5. Record year (the year condition tests performed),
6. Regional factor (AASHTO regional factor, derived from elevation, rainfall, and climate zone: (a) 0.1 for each 1000 ft of elevation, (b) 0.1 for each inch of average annual rainfall, and (c) 0.1 for climate zones),
7. Thickness of original AC surfacing in inches,
8. Thickness of AC overlay in inches,
9. Number of 80-kN (18-kip) equivalent single-axle loads (ESALs) in the year of record,
10. Percentage cracking in the year of record,
11. Percentage cracking one year after the year of record,
12. Mays Meter inches of roughness in the year of record,
13. Mays Meter inches of roughness one year after the year of record,
14. Dynaflect deflections for all five geophones [all deflections were temperature corrected according to the Asphalt Institute method (3)], and
15. Age of pavement according to the year of record (if the year of record was 1976 and pavement age was 8 years, then the pavement was built in 1968).

All of the data are contained in Appendix A of a report by Way and Eisenberg (4), which could not be printed in this paper due to space restrictions.

A number of regression runs were made to deter-

mine correlation with either roughness or percentage cracking directly from the other variables. New variables were created, including spreadability index, surface curvature index, and base curvature index. Direct correlation of all variables to either the magnitude of roughness or percentage cracking gave very poor results. An approach similar to the 1976 equation was attempted that included the use of the change in roughness ( $\Delta R$ ) per year. This approach developed reliable equations that represent the new predictive equations based on real data.

**NEW MODELS**

The models developed represent prediction of future roughness and percentage-cracking conditions based on past data. These models are intended to be used in conjunction with annual pavement condition surveys. The models predict future ride and cracking conditions; that is, given what happened, they predict what will happen. The following predictive models for new and in-service construction were developed and represent ADOT future predictive models.

Percentage Cracking

The predictive model for percentage cracking is as follows:

$$\Delta\%C_n = 0.55(\Delta\%C_p) + 0.031(\Delta\%C_p * \%C) + 0.01(R_g)^2 + 0.05(R_g * \%C) \quad (1)$$

$$- 0.0059(\%C)^2 + 0.186 \quad R^2 = 0.70$$

$$SE = 0.64$$

$$F = 84$$

where

- $\Delta\%C_n$  = change in amount of cracking during next year,
- $\Delta\%C_p$  = change in amount of cracking during previous year,
- $\%C$  = present amount of cracking, and
- $R_g$  = regional factor.

As an example, given that 1976 percentage cracking = 10 and 1977 percentage cracking = 15, change in percentage cracking = 5, and regional factor = 2.0, find the 1978, 1979, and 1980 percentage cracking:

| Year | Percentage Cracking | Change in Percentage Cracking |
|------|---------------------|-------------------------------|
| 1976 | 10                  |                               |
| 1977 | 15                  | 5                             |
| 1978 | 20                  | 5                             |
| 1979 | 27                  | 7                             |
| 1980 | 34                  | 7                             |

Roughness

The predictive model for roughness is as follows:

$$R_n = 0.138(R) + 2.65(R_g)^2 - 0.047(R_g * R) - 0.125 \quad R^2 = 0.54 \quad (2)$$

$$SE = 10.4$$

$$F = 38$$

where  $R_n$  is the change in roughness during the next year and  $R$  is present roughness (Mays Meter roughness: 0-165 = smooth or good ride, 165-255 = fair, and >256 = rough).

As an example, given that 1976 roughness = 100 and the regional factor = 2.0, find the 1977, 1978, and 1979 roughness:

| Year | Roughness | Change in Roughness |
|------|-----------|---------------------|
| 1976 | 100       |                     |
| 1977 | 115       | 15                  |
| 1978 | 130       | 15                  |
| 1979 | 147       | 17                  |

Naturally, each year new roughness and cracking values would be measured in the field; thus, the starting value or seed value would change to reflect the real-world value.

Percentage Cracking with Overlay

The model for percentage cracking with overlays is as follows:

$$\Delta\%C_n = 0.51 + 0.069(\%C) + 0.52(\Delta\%C_p) - 9.9934(D_L)^2 - 0.003(\%C)^2 \quad (3)$$

$$+ 0.068(\Delta\%C_p)^2 \quad R^2 = 0.68$$

$$SE = 0.71$$

where all symbols mean the same as before except that one new term has been added.  $D_L$  is the index to the first year of cracking, a factor that represents the relative amount by which each overlay and overlay plus treatment delays the first crack.

The following levels were used in deriving the index values:

| Factor        | Level      | Value          |
|---------------|------------|----------------|
| Traffic (ADT) | Low        | 2000           |
|               | Medium     | 2001 to 10 000 |
|               | High       | >10 001        |
| Region        | Desert     | 0.0-1.7        |
|               | Transition | 1.8-2.7        |
|               | Mountains  | >2.8           |

Table 1 gives the index values for all treatments, as derived from the performance data base.

It should be noted that immediately after an overlay both  $\%C$  and  $\Delta\%C_p$  are set equal to zero to predict the change in cracking in one year. The term  $D_L$  accounts for the benefit derived by using various treatments to prevent reflective cracking and is similar to the use of the term CRH in the 1976 Woodward-Clyde model. The percentage cracking of the existing pavement and the traffic level are considered by the designer and incorporated into the ADOT network optimization program in such a manner that only feasible designs are considered.

As an example, given a 1976 existing highway with regional factor = 2.0, traffic = 4000 ADT, present cracking = 20 percent, and change in cracking last year = 3, and if a 64-mm (2.5-in) AC overlay would have an index to first crack of 6.5, find the percentage cracking in the years 1977-1984:

| Time                | Year | Percentage Cracking | Change in Percentage Cracking |
|---------------------|------|---------------------|-------------------------------|
| Before overlay      | 1976 | 20                  | 3                             |
| Overlay             | 1976 | 0                   | 0                             |
| Years after overlay |      |                     |                               |
| 1                   | 1977 | 0                   | 0                             |
| 2                   | 1978 | 1                   | 1                             |
| 3                   | 1979 | 2                   | 1                             |
| 4                   | 1980 | 3                   | 1                             |
| 5                   | 1981 | 4                   | 1                             |
| 6                   | 1982 | 5                   | 1                             |
| 7                   | 1983 | 6                   | 1                             |
| 8                   | 1984 | 8                   | 2                             |
| 9                   | 1985 | 9                   | 1                             |

Roughness

For an overlay, the roughness change was found to be

Table 1. Index to first year of cracking.

| Treatment     | Low ADT |            |           | Medium ADT |            |           | High ADT |            |           |
|---------------|---------|------------|-----------|------------|------------|-----------|----------|------------|-----------|
|               | Desert  | Transition | Mountains | Desert     | Transition | Mountains | Desert   | Transition | Mountains |
| SC            | 1.67    | 1.17       | 1.00      | 1.17       | 1.00       | 1.00      | 1.00     | 1.00       | 1.00      |
| ACFC          | 3.00    | 2.50       | 2.00      | 2.83       | 2.50       | 2.00      | 2.83     | 2.50       | 2.00      |
| ACFC + AR     | 7.50    | 6.50       | 5.50      | 6.50       | 4.50       | 3.50      | 5.50     | 4.50       | 4.50      |
| ACFC + HS     | 5.50    | 4.50       | 3.50      | 4.50       | 3.50       | 3.00      | 3.50     | 3.00       | 2.50      |
| 38-mm AC      | 7.50    | 6.50       | 5.50      | 6.50       | 4.50       | 3.50      | 5.50     | 4.50       | 4.50      |
| 38-mm AC + AR | 11.50   | 10.50      | 9.50      | 10.50      | 8.50       | 7.50      | 9.50     | 7.50       | 7.00      |
| 38-mm AC + HS | 7.50    | 6.50       | 5.50      | 6.50       | 4.83       | 4.00      | 5.50     | 5.00       | 5.00      |
| 64-mm AC      | 9.50    | 8.50       | 7.50      | 8.50       | 6.50       | 5.50      | 6.00     | 6.00       | 5.50      |
| 64-mm AC + AR | 12.50   | 11.50      | 10.50     | 11.50      | 9.50       | 8.50      | 11.50    | 9.00       | 7.17      |
| 64-mm AC + HS | 10.83   | 9.83       | 8.83      | 9.83       | 7.83       | 6.83      | 7.17     | 6.50       | 6.17      |
| 89-mm AC      | 11.67   | 10.50      | 9.50      | 10.50      | 9.50       | 6.83      | 8.50     | 8.00       | 7.50      |
| 89-mm AC + AR | 13.50   | 12.83      | 11.83     | 12.83      | 11.83      | 10.83     | 12.50    | 10.83      | 9.83      |
| 89-mm AC + HS | 11.83   | 10.83      | 9.83      | 10.83      | 9.83       | 8.83      | 9.50     | 8.83       | 8.00      |
| 114-mm AC     | 12.50   | 11.50      | 10.50     | 11.50      | 10.50      | 9.50      | 9.50     | 9.00       | 8.50      |
| 140-mm AC     | 13.83   | 12.83      | 11.83     | 12.83      | 11.50      | 10.50     | 11.83    | 10.50      | 9.50      |
| Recycle       | 16.50   | 15.50      | 14.50     | 15.50      | 14.50      | 13.50     | 14.50    | 13.50      | 12.50     |

Notes: 1 mm = 0.039 in.  
 SC = seal coat, ACFC = AC friction course, HS = heater scarification, AR = asphalt rubber, and recycle = combination of AC plus new AC overlay (total AC thickness of nominal 102 mm).

related to the roughness before overlay:

$$R_N = 65.29 - 0.78(R_B) - 0.3055(TH) \quad R^2 = 0.9379 \quad (4)$$

where

$R_N$  = change in roughness one year after an overlay (typically a negative number, which is added to  $R_B$  to find the roughness one year after overlay),

$R_B$  = roughness before overlay, and

$TH$  = thickness of overlay in millimeters or  $-7.76(TH)$  for inches of thickness.

If calculated roughness after overlay is less than 50, roughness is set to 50.

After an overlay, the in-service equation is used to perform future calculations. As an example, given a 1976 pavement with roughness = 200, regional factor = 2.0, and overlay thickness of 64 mm of AC, find roughness for the years 1977-1985:

| Model                | Time           | Year | Roughness | Change in Roughness |
|----------------------|----------------|------|-----------|---------------------|
| Roughness            | Before overlay | 1976 | 200       |                     |
|                      | Overlay        | 1976 |           |                     |
| In-service roughness | After overlay  |      |           |                     |
|                      | 1              | 1977 | 90        | 110                 |
|                      | 2              | 1978 | 104       | 14                  |
|                      | 3              | 1979 | 120       | 16                  |
|                      | 4              | 1980 | 135       | 15                  |
|                      | 5              | 1981 | 152       | 17                  |
|                      | 6              | 1982 | 169       | 17                  |
|                      | 7              | 1983 | 197       | 18                  |
|                      | 8              | 1984 | 205       | 18                  |
| 9                    | 1985           | 225  | 20        |                     |

For both roughness and percentage cracking, the actual amount one year after construction will be monitored. To test the accuracy of future predictions, a verification process was undertaken.

MODEL VERIFICATION

Twenty-nine miles of new construction or in-service pavements as well as 24 miles of overlays were randomly selected from the ADOT file. A verification test was conducted by comparing expected future predicted roughness and percentage cracking with

actual measurements. In addition, the predicted 1976 roughness derived from Woodward-Clyde's original equation was also calculated.

To test the equations, it was necessary to conduct two separate calculations:

1. Case 1--Given a mile of highway built in 1970, assume a new ride of 50 and 0 percent cracking (\* = assumed):

| Year | Ride   |            | Percentage Cracking |            |
|------|--------|------------|---------------------|------------|
|      | Actual | Calculated | Actual              | Calculated |
| 1970 | 42     | 50*        | 0                   | 0*         |
| 1971 | 57     | 55         | 0                   | 1          |
| 1972 | 63     | 60         | 1                   | 2          |
| 1973 | 70     | 65         | 1                   | 3          |

2. Case 2--Given some existing ride or percentage cracking condition, calculate ride or percentage cracking in a future year. As an example, given a mile of highway, find the actual measured ride and the percentage cracking for a given year. Use this measured value to calculate ride or percentage cracking in a future year:

| Year | Actual Roughness | Calculated Ride |            |            |
|------|------------------|-----------------|------------|------------|
|      |                  | Given 1972      | Given 1973 | Given 1974 |
| 1972 | 69               |                 |            |            |
| 1973 | 75               | 77              |            |            |
| 1974 | 86               | 90              | 87         |            |
| 1975 | 103              | 110             | 205        | 100        |

| Year | Actual Percentage Cracking | Calculated Percentage Cracking |            |            |
|------|----------------------------|--------------------------------|------------|------------|
|      |                            | Given 1973                     | Given 1974 | Given 1975 |
| 1973 | 5                          |                                |            |            |
| 1974 | 7                          | 8                              |            |            |
| 1975 | 9                          | 12                             | 10         |            |
| 1976 | 15                         | 16                             | 14         | 13         |

To interpret the results of the above analysis, regressions between the actual and calculated ride and percentage cracking were performed. This is quite straightforward for case 1; for case 2, however, actual and calculated values were grouped by year. Thus, all one-year predictions were grouped together. Likewise, all two-year, three-year, and so forth.

For the sake of brevity, case 1 is not discussed

**Table 2. Correlation between predicted future ride in years 1-7 based on current measured ride.**

| Future Year | N   | R <sup>2</sup> | SE   | A    | B    | Coefficient of Variation (%) |
|-------------|-----|----------------|------|------|------|------------------------------|
| 1           | 195 | 0.8922         | 25.4 | 9.2  | 0.90 | 12                           |
| 2           | 169 | 0.8622         | 28.7 | 12.6 | 0.84 | 14                           |
| 3           | 139 | 0.8327         | 31.4 | 12.9 | 0.80 | 16                           |
| 4           | 111 | 0.8144         | 33.4 | 15.8 | 0.75 | 17                           |
| 5           | 82  | 0.8047         | 34.8 | 16.0 | 0.73 | 18                           |
| 6           | 53  | 0.8066         | 34.6 | 19.7 | 0.70 | 17                           |
| 7           | 25  | 0.8085         | 36.6 | 5.9  | 0.74 | 18                           |

Note: A = intercept and B = slope.

**Table 3. Correlation between predicted cracking in years 1-6 based on current measured percentage cracking.**

| Future Year | N   | R <sup>2</sup> | SE   | A    | B    | Coefficient of Variation (%) |
|-------------|-----|----------------|------|------|------|------------------------------|
| 1           | 163 | 0.9186         | 4.0  | 1.8  | 0.89 | 12                           |
| 2           | 136 | 0.8266         | 6.0  | 4.5  | 0.72 | 18                           |
| 3           | 107 | 0.6435         | 9.0  | 8.0  | 0.55 | 28                           |
| 4           | 79  | 0.6158         | 9.7  | 10.2 | 0.53 | 30                           |
| 5           | 49  | 0.6068         | 10.0 | 12.8 | 0.45 | 31                           |
| 6           | 20  | 0.7091         | 8.5  | 13.2 | 0.42 | 26                           |

**Table 4. Correlation between predicted future ride in years 1-7 based on current measured ride: overlays.**

| Future Year | N   | R <sup>2</sup> | SE   | A    | B    | Coefficient of Variation (%) |
|-------------|-----|----------------|------|------|------|------------------------------|
| 1           | 161 | 0.6555         | 20.9 | 16.5 | 0.75 | 22                           |
| 2           | 138 | 0.6107         | 22.6 | 16.7 | 0.71 | 24                           |
| 3           | 115 | 0.6607         | 21.7 | 8.7  | 0.74 | 22                           |
| 4           | 92  | 0.5777         | 25.1 | 11.7 | 0.66 | 26                           |
| 5           | 69  | 0.5944         | 25.8 | 10.9 | 0.66 | 26                           |
| 6           | 44  | 0.5952         | 23.5 | 25.6 | 0.54 | 21                           |
| 7           | 23  | 0.6760         | 22.4 | 11.6 | 0.56 | 21                           |

in detail in this paper. In the study, however, the case 1 type of prediction for roughness and cracking was found to be quite good considering the uncertainties in site-specific prediction.

Since case 2 represents the model currently in use in the ADOT network optimization program, this paper concentrates on this case.

Case 2: Prediction Given Existing Condition in the Field

For all miles of highway, a predicted expected future roughness or cracking was determined for each future year based on an existing condition.

**Roughness**

Since roughness measurements have been taken since 1972, only those actually measured values were used in this part of the interpretation. Table 2 summarizes the results of this work. The values given in this table clearly show that the PMS equation is very good in predicting the future roughness condition given the present existing pavement condition. The coefficient of variation is less than 20 percent from one year to seven years, which is also very good, considering the uncertainty of the future. It should be noted that the slope (B) decreases with

time. This is similar to the trend for case 1. In order to equate the predicted values more closely to the actual in terms of magnitude, it is suggested that an adjustment factor be used that is equal to the slope up to four years and is set equal to 0.70 for five or more years.

In general, the PMS equation is capable of predicting future roughness extremely well given the existing condition of the highway. Predictions of cracking with small standard errors (less than 20 percent coefficient of variation) are at best very difficult to make due to large increases in cracking that can and do occur in one year. With this in mind, the present PMS equation is considered to be a very good prediction model, as Table 3 indicates.

R<sup>2</sup> values, although lower than the roughness values, are still quite good. The standard error and coefficient of variation are greater than 20 percent, an indication of how dramatic increases in cracking can occur in the field. The slope value decreases with time and should be used to adjust the predicted cracking values back down to magnitudes closer to those observed in the field. For those years beyond five, an adjustment factor of 0.40 is suggested.

In summary, the new PMS equations for both roughness and cracking for both cases 1 and 2 can do a very good job of predicting future pavement distress conditions. This is possible because the models are of a recursive form. The logic behind a recursive model is that a future condition is dependent on a past condition. Thus, more roughness or cracking accelerates the rate of progression to still more and more roughness and cracking until the pavement has lost its desirable serviceability and structural characteristics. To demonstrate still further how the recursive model emulates the real world, additional investigations were performed.

**Roughness for Overlays**

Table 4 summarizes the results of the case 2 calculations of roughness for overlays. Although the R<sup>2</sup> is lower than for new construction, the other values would indicate a good correlation.

Since the slope changes only slightly with time, it is suggested that the average slope (0.66) be used as an adjustment factor for all seven years.

**Cracking for Overlays**

Table 5 summarizes the various case 2 correlation statistics for cracking with overlays. Although the correlation values fall off by year four, the error terms are not excessively large and the slope value is still good. Predictions for four or more years should be adjusted by using a 0.75 value to give more reasonable answers.

In summary, both the roughness and cracking PMS equations for routine overlays appear to do a good job of predicting the future expected conditions. As an additional reinforcement of the recursive equation mode, two additional overlay equations were examined.

Special Treatments with Overlays

Over the years, ADOT has used either heater scarification or asphalt rubber to improve the roughness and cracking performance of overlays. Generally such treatments have been used when unusual amounts of cracking (greater than 10 percent) have been present in the existing road. In addition, they have been used when no other conventional material or process short of reconstruction appeared capable of providing satisfactory performance. Therefore,

Table 5. Correlation between predicted future percentage cracking in years 1-5 based on current measured percentage cracking: overlays.

| Future Year | N   | R <sup>2</sup> | SE   | A   | B    | Coefficient of Variation (%) |
|-------------|-----|----------------|------|-----|------|------------------------------|
| 1           | 124 | 0.7520         | 1.82 | 0.3 | 0.98 | 15                           |
| 2           | 103 | 0.6810         | 2.14 | 0.4 | 0.96 | 17                           |
| 3           | 79  | 0.5316         | 2.74 | 0.8 | 0.91 | 22                           |
| 4           | 57  | 0.3587         | 3.49 | 1.8 | 0.74 | 28                           |
| 5           | 34  | 0.3514         | 4.04 | 1.9 | 0.76 | 32                           |

Table 6. Ride and cracking statistics for asphalt rubber.

| Case | Average                  | Ride   | Cracking |
|------|--------------------------|--------|----------|
| 1    | R <sup>2</sup>           | 0.5777 | 1.0000   |
|      | SE                       | 12.6   | 0.0      |
|      | A                        | 44.3   | 0.0      |
|      | B                        | 0.70   | 1.00     |
|      | Coefficient of variation | 17     | 0.0      |
| 2    | R <sup>2</sup>           | 0.3238 | 1.0000   |
|      | SE                       | 31.3   | 0.0      |
|      | A                        | 39.0   | 0.0      |
|      | B                        | 0.53   | 1.00     |
|      | Coefficient of variation | 33     | 0.0      |

Table 7. Ride and cracking statistics for heater scarification.

| Case | Average                  | Ride   | Cracking |
|------|--------------------------|--------|----------|
| 1    | R <sup>2</sup>           | 0.6239 | 0.8993   |
|      | SE                       | 13.6   | 0.4      |
|      | A                        | -7.2   | 0.1      |
|      | B                        | 1.23   | 0.95     |
|      | Coefficient of variation | 17     | 1.8      |
| 2    | R <sup>2</sup>           | 0.4489 | 0.9257   |
|      | SE                       | 22.3   | 1.2      |
|      | A                        | 35.6   | -0.7     |
|      | B                        | 0.57   | 1.1      |
|      | Coefficient of variation | 23     | 1.6      |

when either conventional-overlay or special-treatment performance is observed, it should be recalled that generally both heater scarification and asphalt rubber were used where the degree of difficulty in improving performance was indeed much higher than that for a routine conventional overlay. It should also be mentioned that extensive use of special treatments as part of routine overlay design strategies is relatively new, which means the data base on field performance is limited. Numerous special research reports have been issued that document performance (5-8). Indeed, Gonsalves (8) reports on the performance of all asphalt rubber projects.

The results of this analysis are grouped by treatment and case.

#### Asphalt Rubber

The ride and cracking statistics for asphalt rubber for cases 1 and 2 are given in Table 6. The ride values are not too good, primarily due to the limited nature of the data. Only five years of data have been collected up until now. The range of ride values is very limited. The standard error and coefficient of variation values are reasonable and indicate that the model is performing as intended. Values of B are smaller than one, which indicates a longer-than-expected life; however, current expected lives already are predicted to be 20 years. Given that the current performance trend represents only

five years of actual data, it is felt that adjustments at this time would be unwise. The cracking prediction for the five-year period is remarkably good. The cracking equation predicted no cracking, and up until now there has been none.

#### Heater Scarification

Statistics for cases 1 and 2 for heater scarification are given in Table 7. As in the cracking case, the ride values are not too good; however, a maximum of only nine years of ride history is known. In addition, the fact that virtually all of the ride values are still in the good range restricts the size of the numbers considerably. The PMS equation seems capable of giving good ride correlation in the future. Cracking statistics are very good for both cases, which indicates that the PMS cracking equation has good prediction capabilities.

In summary, the special-treatments portion of the PMS overlay equations appears to be a reasonably good approximation of the future performance of these materials. As additional ride and cracking data are collected in future years, the equations can be updated and certainly improved.

#### CONCLUSIONS

It has been demonstrated that the PMS models (equations) can reasonably predict both future ride and cracking for AC pavements (new, existing, and overlays). Many suggested minor adjustments should be made to produce an improved set of models. It should be recalled that this is a start; no doubt future verification calculations will make additional adjustments that will improve the models' ability to predict the future.

#### RECOMMENDATIONS

The new PMS prediction models, with adjustments, should become part of the PMS network optimization program. A similar verification process should be repeated about once every four years to test the equations and evaluate new designs or construction techniques, such as recycling, sulfur asphalt, overlays with special treatment, and grinding and overlaying of concrete. Additional special investigations that would determine why some miles of highway have not performed as expected are also encouraged.

ADOT has available to it a valuable prediction tool not available in any other state at this time. This valuable tool should be implemented and used as much as possible within the context of pavement management, design, and research in Arizona.

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## Overview of PAVER Pavement Management System

M.Y. SHAHIN AND S.D. KOHN

A brief overview of the PAVER pavement management system and the capabilities it offers its users is presented. PAVER is designed for use by military installations, cities, and counties. The system capabilities discussed are data storage and retrieval, pavement network definition, pavement condition rating, project prioritization, inspection scheduling, determination of present and future network condition, determination of maintenance and repair needs, performance of economic analysis, and budget planning.

PAVER is a pavement management system designed for use by military installations, cities, and counties. The system was developed and tested over the past 10 years and is currently being implemented by several agencies, including Fort Eustis, the Great Lakes Naval Training Center, and the City of Mesa, Arizona. This system was developed by the U.S. Army Construction Engineering Research Laboratory under the auspices of the Office of the Chief of Engineers, U.S. Army Corps of Engineers. It has been extensively tested prior to its implementation. The objective of this paper is to provide an overview of PAVER with emphasis on what is available to system users. Details of the system's development and results of an economic analysis of its implementation have been documented in a paper by Shahin and Kohn (1) and a paper by Kohn and Shahin in this Record.

PAVER provides the engineer with a practical decisionmaking procedure for identifying cost-effective maintenance and repairs on roads and streets. The System 2000 is the data base manager. This system and other "interface" programs provide the user with report generation capability for critical information. This information allows objective input to the decisionmaking process.

PAVER provides its users with many important capabilities. These include data storage and retrieval, pavement network definition, pavement condition rating, project prioritization, inspection scheduling, determination of present and future network condition, determination of maintenance and repair (M&R) needs, performance of economic analysis, and budget planning. This paper describes these capabilities and presents example reports for each area.

### DATA STORAGE AND RETRIEVAL

The PAVER data base is a custom-designed data structure defined on a commercially available computer data base manager called System 2000 (System 2000 is a registered trademark of the Intel Corporation).

The data structure consists of 12 data groups that are linked together to form a tree structure (see Figure 1). Storing the data in this structure enables the user to retrieve information based on

its connection to other data in the data base. Space is available in each data group to store specific items related to that data group. The Pavement Structure data group shown in Figure 2 is an example.

The data can be stored and retrieved through special "interface" programs (FORTRAN or COBOL) or through the access language of the data base manager. Since these programs are interactive, the user has immediate access to the data base. The programs are designed to supply the information in useful format.

### DEFINITION OF PAVEMENT NETWORK

An installation's (city's) pavement network consists of all surface areas that provide access ways for ground or air traffic (airfield pavements). This network must be divided and identified in order to use the data base. Networks are divided into branches, sections, and sample units, which can be briefly defined as follows:

1. A branch is any identifiable part of the network that is a single entity and has a distinct function, such as an individual street.
2. A section is a division of a branch that has consistent structural composition, construction history, and traffic volume.
3. A sample unit is the smallest unit of the network and is an area of the pavement section used during inspection.

The data base provides information on the pavement network through reports such as "lists" or "inventories". Figure 3 shows a typical output of the inventory report. This report provides general information about specific branches or sections, thus providing the user with overall inventory information.

### PAVEMENT CONDITION RATING

A key component of any pavement management system is a condition rating procedure. The PAVER system uses the pavement condition index (PCI), a composite index of the structural integrity and operating condition of the pavement. It is a numerical index from 0 to 100, where 100 represents excellent condition. The PCI is determined based on quantity, severity, and type of distress, as shown in Figure 4. The PCI was developed to agree closely with the collective judgment of experienced pavement engineers.

The PCI has been divided into seven condition categories, ranging from "excellent" to "failed", as

shown in Figure 5. These categories are useful for developing maintenance policies and guidelines.

The PAVER data base uses reports such as PCI, Inspect, and Sample to provide PCI information. Figure 6 shows a typical output of the Inspect report, which provides the user with PCI and distress information. The report can be used to prepare desk estimates of repairs and to determine history of pavement condition.

PROJECT PRIORITIZATION

Project prioritization is an immediate payoff of pavement network definition and pavement condition rating. The PCI report can be used for this purpose. It lists pavement sections in an increasing order of PCI. Figure 7 shows an example report out-

Figure 1. PAVER data structure.

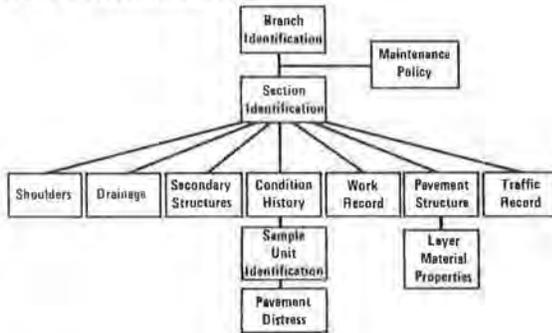


Figure 2. Pavement Structure data group.

- 2500\* PAVEMENT STRUCTURE (RG IN 1000)
- 2501\* DATE CONSTRUCTED (DATE IN 2500)
- 2502\* LAYER CATEGORY (NAME X(10) IN 2500)
- 2503\* LAYER MATERIAL CODE (INTEGER NUMBER 999 IN 2500)
- 2504\* LAYER MATERIAL (NAME X(20) IN 2500)
- 2505\* LAYER THICKNESS (DECIMAL NUMBER 99.9 IN 2500)
- 2506\* TYPE OF CURBING (NAME X(10) IN 2500)
- 2507\* LAYER COMMENTS (NON-KEY NAME X(39) IN 2500)
- 2509\* PAVEMENT STRUCTURE UPDATE (NON-KEY DATE IN 2500)
- 2509\* FACTOR 2509 (NON-KEY DECIMAL NUMBER 9(B),99 IN 2500)
- 2510\* FACTOR 2510 (NON-KEY DECIMAL NUMBER 9(B),99 IN 2500)
- 2511\* FACTOR 2511 (NON-KEY DECIMAL NUMBER 9(B),99 IN 2500)
- 2512\* FACTOR 2512 (NON-KEY DECIMAL NUMBER 9(B),99 IN 2500)
- 2513\* FACTOR 2513 (NON-KEY DECIMAL NUMBER 9(B),99 IN 2500)
- 2514\* PSTR-CONCAT (NAME X(19) IN 2500)
- 3100\* LAYER MATERIAL PROPERTIES (RG IN 2500)
- 3101\* TEST DATE (DATE IN 3100)
- 3102\* TEST TYPE (NAME X(31) IN 3100)
- 3103\* TEST VALUE (DECIMAL NUMBER 9(S),9999 IN 3100)
- 3104\* TEST UNIT (NON-KEY NAME X(13) IN 3100)
- 3105\* FACTOR 3105 (NON-KEY DECIMAL NUMBER 9(B),99 IN 3100)
- 3106\* FACTOR 3106 (NON-KEY DECIMAL NUMBER 9(B),99 IN 3100)
- 3107\* FACTOR 3107 (NON-KEY DECIMAL NUMBER 9(B),99 IN 3100)
- 3108\* FACTOR 3108 (NON-KEY DECIMAL NUMBER 9(B),99 IN 3100)
- 3109\* FACTOR 3109 (NON-KEY DECIMAL NUMBER 9(B),99 IN 3100)
- 3110\* LMAT-CONCAT (NAME X(26) IN 3100)

Figure 3. Example output of Inventory report.

REPORT DATE- 02/18/82

| INVENTORY<br>NON-FAMILY HOUSING PAVEMENTS |              |   |                  |              |
|---|--------------|---|------------------|--------------|
|   | SURF<br>TYPE | BRANCH<br>USE                                       | PAVEMENT<br>RANK | AREA<br>(SQ) |
| IWASN WASHINGTON NORTH                    |              |   |                  |              |
| SECTION 01                                | AC           | ROADWAY   | PRIMARY          | 4007         |
| FROM- ROUTE 105                           |              |   |                  |              |
| TO- CL MADISON AVE                        |              |   |                  |              |
| SECTION 02                                | AC           | ROADWAY   | PRIMARY          | 6651         |
| FROM- CL MADISON AVE                      |              |   |                  |              |
| TO- N'LY SIDE HINES CIR                   |              |   |                  |              |
| SECTION 03                                | AC           | ROADWAY   | PRIMARY          | 4000         |
| FROM- S'LY SIDE HINES CIR                 |              |   |                  |              |
| TO- CENTER OF SOMERVELL                   |              |   |                  |              |
| SECTION 04                                | AC           | ROADWAY   | PRIMARY          | 6340         |
| FROM- CENTER OF SOMERVELL                 |              |   |                  |              |
| TO- N'LY EDGE TAYLOR                      |              |   |                  |              |
| SECTION 05                                | PCC          | ROADWAY   | SECONDARY        | 4453         |
| FROM- S'LY EDGE TAYLOR                    |              |   |                  |              |
| TO- N'LY EDGE WILSON                      |              |   |                  |              |
|   |              | TOTAL BRANCH AREA                                   |                  | 25451        |
|   |              | TOTAL AREA OF SELECTED NON-FAMILY HOUSING PAVEMENTS |                  | 25,451       |

put. The information in the report can be sorted based on pavement surface type, pavement rank (functional class), traffic type and volume, PCI range, or a combination of factors. Therefore, the report can be used to prioritize projects based on the user's policy.

INSPECTION SCHEDULING

The Inspection Schedule report has been developed to maintain current condition data with efficient inspection level. This report produces a plot and list of the pavement sections to be surveyed for the next six years for any type of branch use (roadway, parking, etc.) and surface type (asphalt, portland cement, concrete, etc.).

The schedule is based on two criteria. One is the minimum PCI that a given pavement type is allowed to reach, and the second is the rate of deterioration (loss of PCI points per year). The user inputs the minimum PCI values and the years allowed between inspections for various deterioration rates. The PCI for the selected sections is then predicted by a straight-line extrapolation based on

Figure 4. Steps for determining PCI of a pavement section.

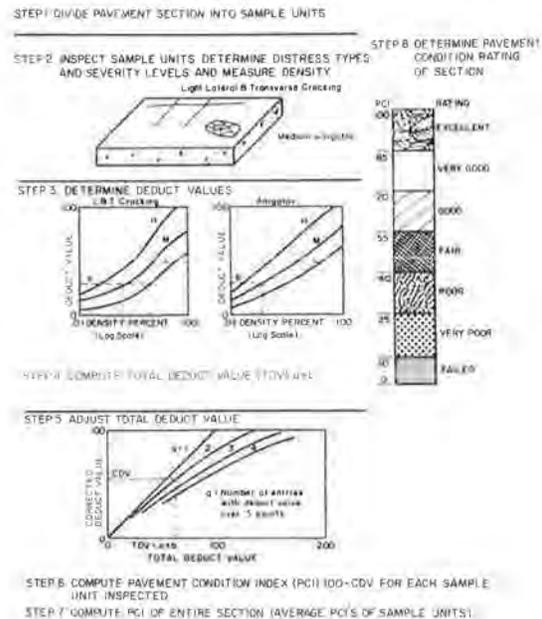


Figure 5. Correlation of M&R zones with PCI and condition rating for air-field pavements.

| M & R ZONE              | PCI | RATING    |
|-------------------------|-----|-----------|
| ROUTINE                 | 100 | EXCELLENT |
|                         | 85  | VERY GOOD |
| ROUTINE, MAJOR, OVERALL | 70  | GOOD      |
|                         | 55  | FAIR      |
| MAJOR, OVERALL          | 40  | POOR      |
| OVERALL                 | 25  | VERY POOR |
|                         | 10  |           |
|                         | 0   | FAILED    |

Figure 6. Example output of Inspect report.

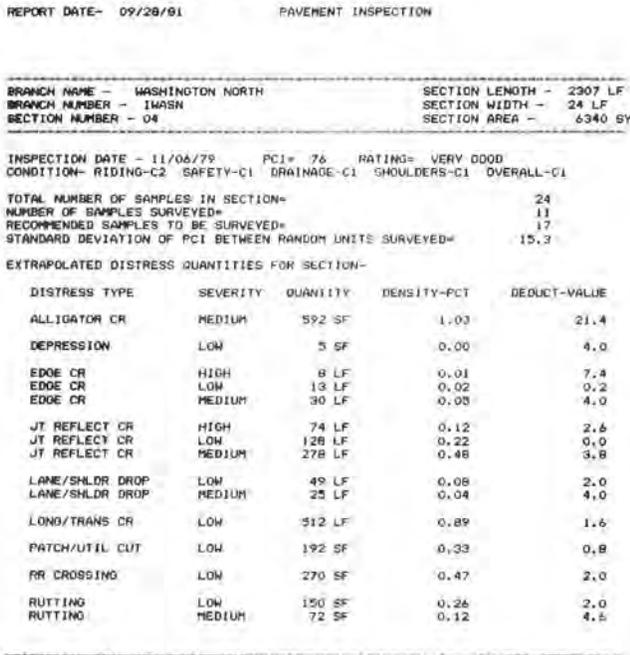


Figure 7. Example output of PCI report.

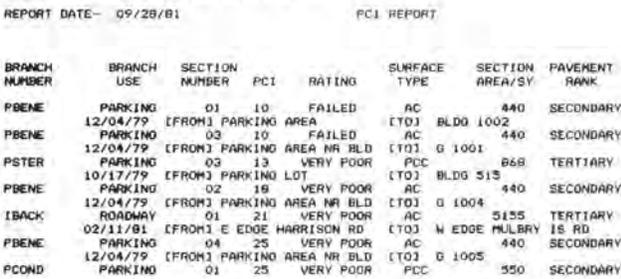
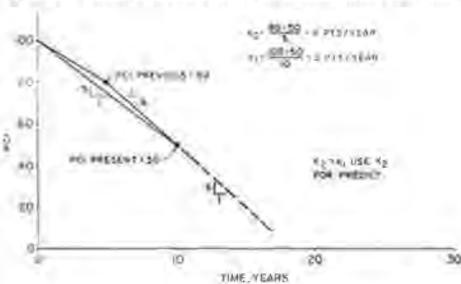


Figure 8. Example case of PCI prediction when PCI was previously determined.



the maximum slope from either the last inspection or construction-overlay date (see Figure 8). Sections reaching the minimum PCI within six years of reaching the time limit based on the rate of deterioration are selected for inspection in the appropriate year.

Figure 9 shows a typical Inspection Schedule output with plot and list of cases. The example shown is for primary roadways with asphalt concrete surfaces. By using this report, the engineer can keep the pavement network data base up to date with minimal effort.

Figure 9. Example output of Inspection Schedule report.

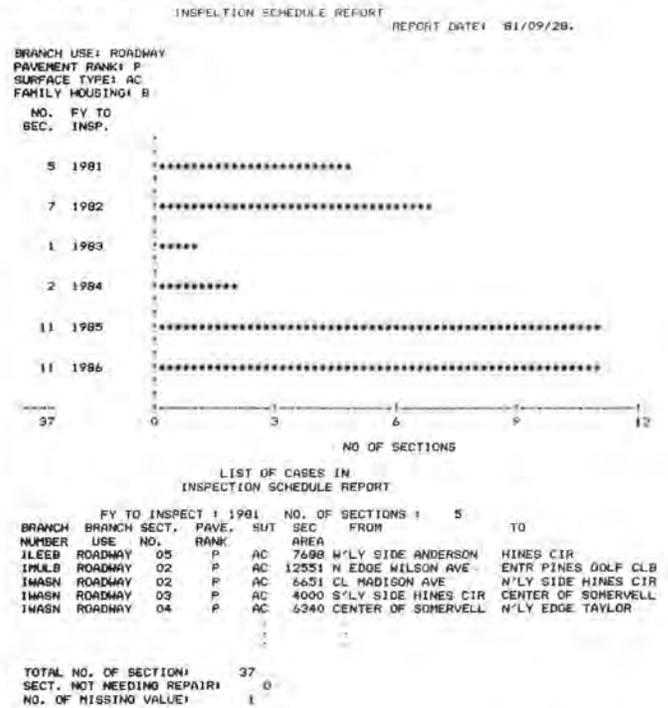
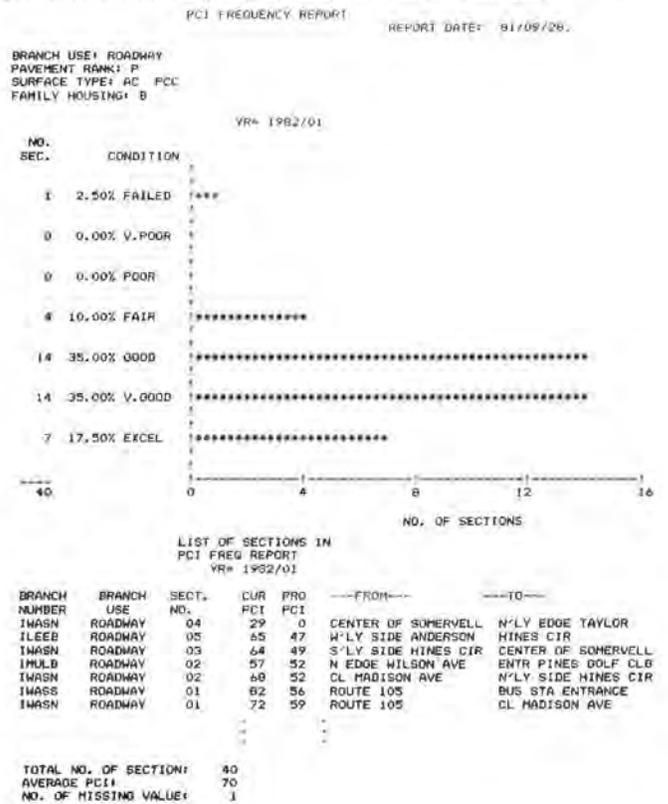


Figure 10. Example output of PCI Frequency report for January 1982.



DETERMINATION OF PRESENT AND FUTURE NETWORK CONDITION

An overall PCI Frequency report has been developed to help plan future M&R and to inform management of

Figure 11. Example output of PCI Frequency report for January 1983.

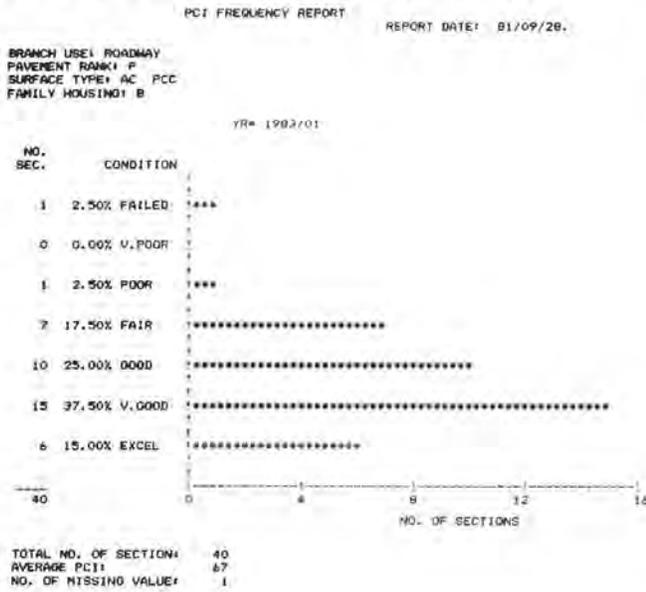
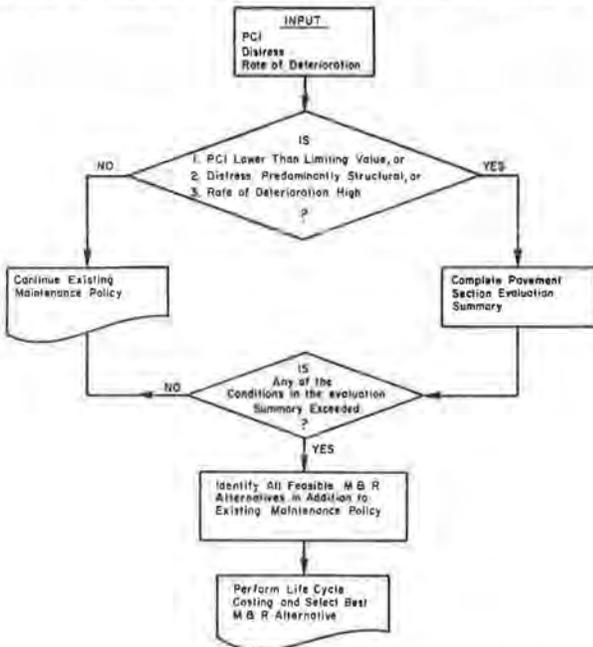


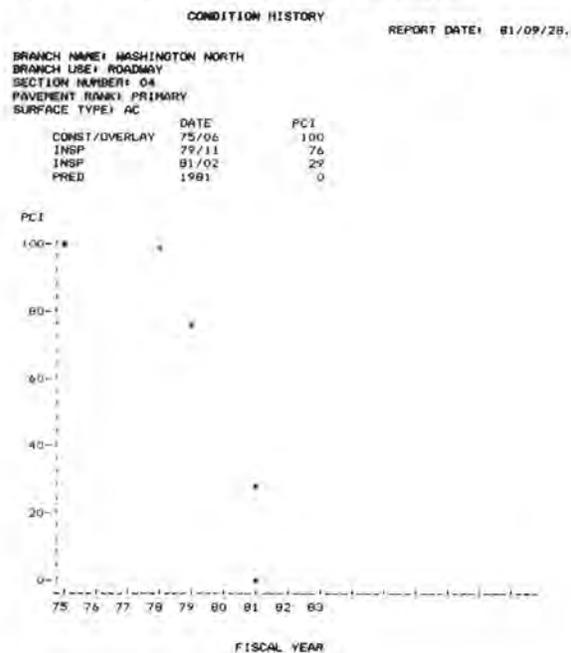
Figure 12. Flow diagram of decision process for determining M&R needs.



the network condition. The report shows an estimated frequency of condition (based on the PCI scale) for the year requested. The pavement sections included in the report can be selected based on branch use, pavement rank, and surface type.

The frequency is estimated as in the Inspection Schedule report by using a straight-line extrapolation of the PCI. Figures 10 and 11 show typical outputs of this report. These two figures show the estimated frequency of occurrence for the same set of pavement sections for two different years. The extrapolation presumes that no major repairs (such as slab replacement or overlay) have occurred between the last inspection and prediction dates. Thus, the impact of performing no major repairs can be seen.

Figure 13. Example output of Condition History report.



DETERMINATION OF M&R NEEDS

A decision process has been devised for determining the M&R needs of a pavement section. Figure 12 shows a flow diagram of this process. A first-level decision can be made, based on the PCI value, type of distress, and deterioration rate. PAVER provides reports such as PCI and Condition History to help the user make the first decision. The PCI report is an ordered listing of sections ranked by PCI (Figure 7). The Condition History report can be used to determine the rate of deterioration; the report plots the PCI over time for a given section. The plot shows the PCI at each inspection date and linearly extrapolates a point five years beyond the last inspection date. Figure 13 shows an example of this report. The type of distress can be determined from the inspect report, shown in Figure 6.

If a pavement section does not require further analysis, routine maintenance practices can be continued. Routine maintenance includes practices such as spall repair and crack filling. By using maintenance guidelines for specific distress types, such as those given in Table 1, the user can input a repair policy. This policy is used in a program called MRG (Maintenance and Repair Guidelines) to estimate the type and cost of routine repair to specific sections. The MRG report can also be used to compute the cost of overlay after distress repair. Figure 14 shows an output of the MRG report.

If a section requires further analysis, an evaluation summary is completed for the section. The evaluation is based on structural capacity, roughness, skid problems, and other relevant factors, as shown in the top half of Figure 15. Complete guidelines for performing the evaluation are presented elsewhere (1). Feasible M&R alternatives are identified based on the results of the evaluation, as shown in the bottom half of Figure 15. This figure is an output of the Evaluation Summary report that was developed based on input from many experiences (maintenance engineers). The output from the report is general. Therefore, the engineer needs to select specific alternatives and perform the design based on the user agency policy. This may include using nondestructive testing.

PERFORMANCE OF ECONOMIC ANALYSIS

Several repair (or construction) alternatives may be considered feasible for any given pavement section. To help select the appropriate alternative, an Eco-

nomics Analysis program has been developed and added to the system. The program allows the user to input initial costs, periodic maintenance costs, and separate future maintenance costs. Figure 16 shows a typical input, and Figure 17 shows a corresponding

Table 1. Maintenance guidelines for asphalt pavement distresses.

| Distress Type                       | M&R Method |            |                     |                  |            |                 |                          |                             |                    |                           |
|-------------------------------------|------------|------------|---------------------|------------------|------------|-----------------|--------------------------|-----------------------------|--------------------|---------------------------|
|                                     | Do Nothing | Crack Seal | Partial-Depth Patch | Full-Depth Patch | Skin Patch | Pothole Filling | Apply Heat and Roll Sand | Apply Surface Seal Emulsion | Apply Rejuvenation | Apply Aggregate Seal Coat |
| Alligator cracking                  |            |            | M,H                 |                  |            |                 |                          | L                           | L                  |                           |
| Bleeding                            | L          |            |                     |                  |            |                 | L,M,H                    |                             |                    |                           |
| Block cracking                      | L          | L,M,H      |                     |                  |            |                 |                          | L                           | L,M                |                           |
| Bumps and sags                      | L          |            | M,H                 | M,H              | M,H        |                 |                          |                             |                    |                           |
| Corrugation                         | L          |            | M,H                 | M,H              |            |                 |                          |                             |                    |                           |
| Depression                          | L          |            | M,H                 | M,H              | M,H        |                 |                          |                             |                    |                           |
| Edge cracking <sup>a</sup>          | L          | L,M        | M,H                 | M,H              |            |                 |                          |                             |                    |                           |
| Joint reflective cracking           | L          | L,M,H      | H                   |                  |            |                 |                          |                             |                    |                           |
| Lane-shoulder drop-off <sup>b</sup> | L          |            |                     |                  |            |                 |                          |                             |                    |                           |
| Longitudinal transverse cracking    | L          | L,M,H      | H                   |                  |            |                 |                          | L                           | L                  | L,M                       |
| Patching and utility cut            | L          | M          | H <sup>c</sup>      | H <sup>c</sup>   |            |                 |                          |                             |                    |                           |
| Polished aggregate                  | A          |            |                     |                  |            |                 |                          |                             |                    | A                         |
| Potholes                            |            |            | L                   | L,M,H            |            | L,M,H           |                          |                             |                    |                           |
| Railroad crossing                   | L          |            |                     |                  | L,M,H      |                 |                          |                             |                    |                           |
| Rutting                             | L          |            | L,M,H               | M,H              | L,M,H      |                 |                          |                             |                    |                           |
| Shoring                             | L          |            | M,H                 |                  |            |                 |                          |                             |                    |                           |
| Slippage cracking                   | L          | L          | M,H                 |                  |            |                 |                          |                             |                    |                           |
| Swell                               | L          |            |                     | M,H              |            |                 |                          |                             |                    |                           |
| Weathering and raveling             | L          |            | H                   |                  |            |                 |                          | L,M                         | L                  | M,H                       |

Note: L = low severity; M = medium severity; H = high severity; and A = has only one level of severity.

<sup>a</sup> If predominant, apply shoulder seal, e.g., aggregate seal coat.

<sup>b</sup> If predominant, level off shoulder and apply aggregate seal coat.

<sup>c</sup> Replace patch.

Figure 14. Example output of MRG report. REPORT DATE - 01/10/05.

```

MAINTENANCE AND REPAIR GUIDELINES
BRANCH NAME - WASHINGTON NORTH          SECTION LENGTH - 2307 LF
BRANCH NBR - 1WASH                      SECTION WIDTH - 24 LF
SECTION NBR - 04                        SECTION AREA - 6340 SF

INSPECTION DATE - 02/11/01              SECTION PCI - 29

DISTRESS   DIS  DIST-QTY  WORK          MATL  LABOR  LABOR  MAT'L  EQUIP  TOTAL
TYPE       SEV  WORK-QTY  TYPE          CODE  HOURS  COST$  COST$  COST$  COST$
-----
ALLIGATOR CR  M  1482 SF      SHALLOW PATCH  120  841.0  10092  336  1867  13136
LONG/TRANS CR  M  975 LF      CRACK FILLING  171  0.0    0      0     0     639
                                OVERLAY        120                                19029
TOTAL        841.0  10092  336  1867  32769
    
```

Figure 15. Example output of Evaluation Summary report.

```

CURRENT VALUES ARE AS FOLLOWS ==
1  PCI == 29
2  LOCAL VARIATION(Y/N) == N
3  SYSTEMATIC VARIATION(Y,N) == N
4  SHORT TERM RATE OF DETERIORATION(L,N,H) == L
5  LONG TERM RATE OF DETERIORATION(L,N,H) == H
6  MAJOR SOURCE OF DISTRESS(LOAD,CLIMATE) == L
7  LOAD CARRYING DEFICIENCY(Y,N) == Y
8  SURFACE ROUGHNESS(L,M,H) == L
9  SKID/HYDROPLANING PROBLEMS(L,M,H) == L
10 PREVIOUS MAINTENANCE(L,N,H) == N
SELECT(A-D) ==
I>C

DATE == 10 FEB 02          FEASIBLE M&R ALTERNATIVES
BASE == M Y SHAHIN        FEATID == WASH      PCI == 29
FEATNM == WASHINGTON BLVD M&R REPAIR ZONE == MAJOR-OVERALL

***** RECOMMENDED MAINTENANCE ALTERNATIVES *****
1 == RECONSTRUCTION
2 == OVERLAY STRUCTURAL AC
4 == OVERLAY PCC
11 == RECYCLE STRUCTURE

*** END ***
    
```

Figure 16. Typical input to Economic Analysis program.

| M&R ACTIVITY DESC      | YEAR | COST     | TIME-SPACING |
|------------------------|------|----------|--------------|
| 1 & IN GRAN.#10/TON    | 1982 | 32630.00 | 0            |
| 2 PRIME #.27/SY        | 1982 | 3802.00  | 0            |
| 3 4 IN AC.#30/TON      | 1982 | 38704.00 | 0            |
| 4 REP 2 X.1.5 INT COST | 1987 | 3754.00  | 5            |
| 5 PATCH & MAINT..05/SY | 1988 | 704.00   | 1            |
| 6 SURF SEAL.#.1/SY     | 1989 | 1408.00  | 0            |
| 7 SURF SEAL.#.1/SY     | 1996 | 1408.00  | 0            |

Figure 17. Economic Analysis output for input shown in Figure 16.

DATE:= 82/02/18. PROJECTED COST ANALYSIS (DETAIL)

SECTION ID:=TW & IN GRAN SECTION AREA(S.Y.):= 14080.0  
LIFE OF ALTERNATIVE:= 20 INTEREST RATE:= 10.0 INFLATION RATE:= 0.0

| M&R ACTIVITY                        | YEAR | COST(S)   | PRESENT VALUE(S) |
|-------------------------------------|------|-----------|------------------|
| & IN GRAN.#10/TON                   | 1982 | 32630.00  | 32630.00         |
| PRIME #.27/SY                       | 1982 | 3802.00   | 3802.00          |
| 4 IN AC.#30/TON                     | 1982 | 38704.00  | 38704.00         |
| TOTAL:=                             |      | 125136.00 | 125136.00        |
| REP 2X.1.5 INT COST                 | 1987 | 3754.00   | 2330.94          |
| PATCH & MAINT..05/SY                | 1988 | 704.00    | 397.39           |
| PATCH & MAINT..05/SY                | 1989 | 704.00    | 361.26           |
| SURF SEAL.#.1/SY                    | 1989 | 1408.00   | 722.53           |
| TOTAL:=                             |      | 2112.00   | 1083.79          |
| PATCH & MAINT..05/SY                | 1990 | 704.00    | 326.42           |
| PATCH & MAINT..05/SY                | 1991 | 704.00    | 298.56           |
| REP 2X.1.5 INT COST                 | 1992 | 3754.00   | 1447.33          |
| PATCH & MAINT..05/SY                | 1992 | 704.00    | 271.42           |
| TOTAL:=                             |      | 4458.00   | 1718.75          |
| PATCH & MAINT..05/SY                | 1993 | 704.00    | 246.75           |
| PATCH & MAINT..05/SY                | 1994 | 704.00    | 224.32           |
| PATCH & MAINT..05/SY                | 1995 | 704.00    | 203.92           |
| PATCH & MAINT..05/SY                | 1996 | 704.00    | 185.39           |
| SURF SEAL.#.1/SY                    | 1996 | 1408.00   | 370.77           |
| TOTAL:=                             |      | 2112.00   | 556.16           |
| REP 2X.1.5 INT COST                 | 1997 | 3754.00   | 898.68           |
| PATCH & MAINT..05/SY                | 1997 | 704.00    | 168.93           |
| TOTAL:=                             |      | 4458.00   | 1067.21          |
| PATCH & MAINT..05/SY                | 1998 | 704.00    | 153.21           |
| PATCH & MAINT..05/SY                | 1999 | 704.00    | 139.28           |
| PATCH & MAINT..05/SY                | 2000 | 704.00    | 126.62           |
| PATCH & MAINT..05/SY                | 2001 | 704.00    | 115.11           |
| INITIAL COST(S):=                   |      | 125136.00 |                  |
| PRESENT VALUE(S):=                  |      | 134126.43 |                  |
| EQUIVALENT UNIFORM ANNUAL COST(S):= |      | 15754.44  |                  |
| EUAC PER SQ. YD. (S):=              |      | 1.12      |                  |

END OF REPORT

output. As shown, the user is provided with the initial cost, present value, equivalent uniform annual cost, and equivalent uniform annual cost per square yard. The Economic Analysis program allows the user to vary interest rates, inflation rates, repair costs, and timing so that their effect on alternatives can be easily analyzed.

BUDGET PLANNING

A budget planning report was developed to provide an estimate of the rehabilitation dollars required over a 10-year period for a given level of condition. The report is based on the user's input of minimum PCI levels for various branch uses and pavement rank. The user also inputs unit repair costs based on pavement surface type and the PCI scale; i.e., the cost of repair can be varied, depending on the PCI value. Thus, the increased cost of differing rehabilitation can be anticipated. The program also takes into account the inflation rate. Figure 18 shows an example output of this report.

This program predicts, for each pavement section, the year in which the minimum PCI is reached and calculates the cost of repair. The prediction is the straight-line prediction procedure explained in the Inspection Schedule report.

SUMMARY

This paper has presented a brief overview of PAVER, a pavement management system for military installations, cities, and counties. PAVER assists engi-

Figure 18. Example output of Budget Planning report.

BUDGET PLANNING REPORT REPORT DATE: 81/09/28.

BRANCH USE: ROADWAY  
PAVEMENT RANK: P  
SURFACE TYPE: AC  
INFLATION RATE: 10.00  
FAMILY HOUSING: B

| COST   | FY TO REPAIR |
|--------|--------------|
| 364.63 | 1981         |
| 132.10 | 1982         |
| 12.07  | 1983         |
| 0.00   | 1984         |
| 60.21  | 1985         |
| 178.01 | 1986         |
| 11.22  | 1987         |
| 3.60   | 1988         |
| 91.03  | 1989         |
| 45.12  | 1990         |
| 0.00   | 1991         |
| 917.99 |              |

COST IN THOUSANDS

LIST OF CASES IN BUD PLANNING REPORT

| FY TO REPAIR | BRANCH USE | BRANCH NO. | SECT. | PAVE. RANK | SUT   | SEC AREA | COST TO REPAIR (#1000'S) |
|--------------|------------|------------|-------|------------|-------|----------|--------------------------|
| 1981         | ILEEB      | ROADWAY 05 | P     | AC         | 7688  | 35.74    |                          |
| 1981         | INULB      | ROADWAY 02 | P     | AC         | 12351 | 96.02    |                          |
| 1981         | IPERS      | ROADWAY 03 | P     | AC         | 1917  | 13.80    |                          |
| 1981         | IWASN      | ROADWAY 01 | P     | AC         | 4007  | 26.45    |                          |
| 1981         | IWASN      | ROADWAY 02 | P     | AC         | 6651  | 47.22    |                          |
| 1981         | IWASN      | ROADWAY 03 | P     | AC         | 4000  | 29.20    |                          |
| 1981         | IWASN      | ROADWAY 04 | P     | AC         | 6340  | 64.03    |                          |

TOTAL NO. OF SECTION: 20  
SECT. NOT NEEDING REPAIR: 9  
NO. OF MISSING VALUE: 1

MINIMUM PCI TABLE

| ROADWAY | P  |
|---------|----|
|         | 65 |

SUT UNIT COST TABLE

| SUT | PCI | 0-20  | 21-40 | 41-60 | 61-80 | 81-100 |
|-----|-----|-------|-------|-------|-------|--------|
| AC  |     | 12.00 | 10.00 | 8.00  | 7.00  | 5.00   |

neers and planners with pavement management by providing the data base and computational capabilities. These capabilities are data storage and retrieval, pavement network definition, pavement condition rating, project prioritization, inspection scheduling, determination of present and future network condition, determination of M&R needs, performance of economic analysis, and budget planning.

ACKNOWLEDGMENT

The views expressed in this paper are ours and do not necessarily reflect the views of the U.S. Department of the Army or the U.S. Department of Defense.

REFERENCE

1. M.Y. Shahin and S.D. Kohn. Pavement Maintenance Management for Roads and Parking Lots. U.S. Army Construction Engineering Research Laboratory, Champaign, IL, Tech. Rept. M-294, Dec. 1981.

# Economic Analysis of Field Implementation of PAVER Pavement Management System

S.D. KOHN AND M.Y. SHAHIN

The results of an economic analysis of the performance of the PAVER pavement management system at a military installation are presented. PAVER was developed by the U.S. Army Corps of Engineers over the past 10 years for use by military installations, cities, and counties. It provides the user with practical management tools, including data storage and retrieval, pavement network definition, pavement condition rating, project prioritization, inspection scheduling, determination of present and future network condition, determination of maintenance and repair needs, performance of economic analysis, and budget planning. The economic analysis was performed based on data collected during a prototype evaluation test (PET), which consisted of PAVER data gathering for the entire installation and monitoring of the use and cost of use of PAVER by the installation personnel. Although the PET took two years, the official cost-monitoring period was four months. The monitoring team consisted of 21 pavement engineers. Two economic analyses were performed: (a) the "PET data comparison", an analysis based strictly on the data collected during the four-month PET, and (b) the "estimated data comparison", an analysis based on estimated times and costs for expected annual use. The results of the PET data comparison showed that the annual cost of pavement management with PAVER is approximately half that with the current operating method. The results of the estimated data comparison showed that the annual cost of PAVER is approximately 30 percent that of the current method.

PAVER is an automated pavement management system that provides the user with practical management tools, including data storage and retrieval, pavement network definition, pavement condition rating, project prioritization, inspection scheduling, determination of present and future network condition, determination of maintenance and repair (M&R) needs, performance of economic analysis, and budget planning. PAVER uses the System 2000 (a trademark of Intel Corporation) as the data base manager. This system and other "interface" programs allow the user to generate preformatted reports of critical information. This information allows objective input to the decisionmaking process. A complete description of PAVER is provided in a paper by Shahin and Kohn in this Record and in another paper by the same authors (1).

This paper presents an economic analysis of PAVER based on a full-scale field prototype evaluation test (PET) at a U.S. military installation. The official PET monitoring was started on February 16, 1981, and ran through June 15, 1981. The pavements of the military installation under study are equivalent to 212 lane miles. The test was monitored by 21 pavement experts from Major Command Headquarters of the U.S. Army Corps of Engineers and several installations. Two analyses are presented, one based on the PET data only and the other based on the PET data and estimates.

## DESCRIPTION OF PET

The PET was started by letting a lump sum contract in September 1979 to collect all the necessary information to create a full data base on the study pavements. This contract included the following items:

1. Divide the pavement network into branches and sections and provide maps documenting the division.
2. Perform a pavement condition survey on all paved areas: roadways, parking areas, motorpools, helipads, runways, taxiways, and aprons.
3. Collect pavement structure information from

as-built drawings and core borings.

4. Collect all information regarding drainage, secondary structures, and shoulders.

5. Input data into data base and verify the input.

The total contract price of the data collection was \$91 437. Breakdowns of the amount of pavement surveyed and the contract cost are given in Table 1 and in the table below, respectively (1 lane mile = 7330 yd<sup>2</sup>):

| Item                      | Cost (\$) |
|---------------------------|-----------|
| Inspection                | 64 800    |
| Coring                    | 15 650    |
| Keypunch                  | 1 650     |
| Data verification         | 9 000     |
| Contract overhead         | 327       |
| Total                     | 91 427    |
| Additional computer input | 1 000     |
| Total                     | 92 427    |
| Cost per lane mile        |           |
| Total                     | 436       |
| Inspection                | 306       |

Based on the data given in Table 1, the cost of inspection was calculated to be \$306/lane mile. This reflects the inspection cost for a sampling rate of 51 percent.

It was learned from the PET that the initial sampling rate need not be this high for the initial implementation to provide adequate information on pavement condition. It is anticipated that a sampling rate of approximately 15 percent would be sufficient. By using this reduced sampling rate, the estimated contract cost for full-scale implementation given below was derived:

| Activity            | Contract Cost (\$) |
|---------------------|--------------------|
| Inspection          | 19 100             |
| Keypunch (or input) | 500                |
| Data verification   | 2 600              |
| Computer time       | 1 000              |
| Coring              | 15 650             |
| Terminal equipment  | 500                |
| Total               | 39 350             |

Table 1. Amount of pavement surveyed.

| Branch Use | No. of Branches | No. of Sections | Equivalent Lane Miles | Total Section Area (yd <sup>2</sup> ) |
|------------|-----------------|-----------------|-----------------------|---------------------------------------|
| Roadway    | 94              | 188             | 78                    | 569 862                               |
| Parking    | 75              | 224             | 88                    | 648 500                               |
| Motorpool  | 2               | 7               | 25                    | 181 569                               |
| Runway     | 1               | 1               | 4                     | 26 431                                |
| Taxiway    | 0               | 0               | 0                     | 0                                     |
| Apron      | 4               | 4               | 16                    | 121 875                               |
| Helipad    | 1               | 1               | 1                     | 7 147                                 |
| Total      | 177             | 425             | 212                   | 1 555 384                             |

Note: A branch is an easily identifiable entity of the network, such as "Washington Boulevard"; a section is a portion of a branch that is uniform in construction history, structure composition, traffic, etc. (a sample unit is an inspection unit of approximately 2500 ft<sup>2</sup> for asphalt sections and 20 slabs for jointed concrete pavements. The total number of sample units was 5198, and the total number of samples inspected was 2637, for a sampling rate of 50.7 percent.

These values were obtained by linearly interpolating the contract prices for the 51 percent rate.

During the PET, the form shown in Figure 1 was used to record the computer time and person hours associated with using PAVER and to provide an estimate of the time involved in performing each task manually. A portion of the data from the returned forms is given in Table 2. The hours recorded for M&R project development given at the bottom of the table were estimated by the Engineering Planning Division at the installation. The 120 h shown were used with PAVER information in planning a total of 36 projects when end-of-year money was available. The figure of 480 h is an estimate of the time required to do the same work without the aid of the PAVER system. The installation personnel indicated that without the PAVER system several projects would have had to be eliminated due to lack of time. Thus, the installation would not have been able to obligate the full amount of money available.

A review of the data indicated that the principal time savings occurred in developing long-range plans, budget information reports, M&R cost estimating, and economic analysis. The savings come from the extra computing power offered by PAVER that is not available under the current operating method. By projecting the totals given in Table 2 over a one-year period, the following totals are estimated:

| Category            | Time               |
|---------------------|--------------------|
| PAVER time          | 525 person-h/year  |
| PAVER computer time | 17 391 ccu's/year  |
| Current method time | 1748 person-h/year |

The ccu's given above were incurred both interactively and through the PAVER "batch process" procedure. Interactive runs cost about \$0.12/ccu; the ccu cost in the batch process can vary from \$0.015 to \$0.075/ccu, depending on the selected priority. To develop a weighted average cost for computer use,

the costs and percentages of use given below were used:

| Priority     | ccu Cost (\$) | Use (%) |
|--------------|---------------|---------|
| P01          | 0.015         | 20      |
| P02          | 0.025         |         |
| P04          | 0.05          |         |
| P06          | 0.06          | 30      |
| P10          | 0.075         | 10      |
| P15          | 0.12          | 40      |
| Weighted avg | 0.0765        |         |

These percentages of use were verified with the chief of the Buildings and Grounds Division. The resulting average cost based on this table is \$0.0765/ccu. The Buildings and Grounds Division chief indicated that, as the installation becomes more familiar with PAVER, it is likely to use more of the lower priority (i.e., P01) than the PET indicates. This will result in a reduced computer cost.

The data presented in this section are used in the economic analysis in the following sections.

ECONOMIC ANALYSIS

General

The economic analysis of the PAVER system and the current operating method is developed in the following two ways:

1. The alternatives (i.e., the PAVER system and the current method) are compared based on the PET data projected annually. An inherent assumption in this comparison is that the activities performed during the four months of the PET represent normal annual operations. This comparison is referred to from now on as the "PET data comparison".

2. The alternatives are compared based on estimated times and costs for expected annual use. The data used for this analysis are based on Table 2 and additional input from the chief of the Buildings and Grounds Division at the study installation. This comparison is referred to from now on as the "estimated data comparison".

The analysis method used was a present-worth analysis that used a life of eight years for the PAVER system.

Assumptions

The economic analysis presented here is based on the following assumptions:

Figure 1. Form used to record time and cost data during PET.

NAME: \_\_\_\_\_  
 ACTIVITY DESCRIPTION: \_\_\_\_\_

| RESOURCES          | PAVER | PREVIOUS METHODS | REMARKS |
|--------------------|-------|------------------|---------|
| COMPUTER COST      |       |                  |         |
| LABOR HOURS & RATE |       |                  |         |

Table 2. Summary of PAVER use and estimated current system time.

| Date    | Activity   | PAVER    |                       |                  |
|---------|--|----------|-----------------------|------------------|
|         |  | Time (h) | Computer Charge Units | Current Time (h) |
| 6/3/81  | Develop \$200 000 bids   | 4        | 135.561               |                  |
| 6/8/81  | List of work requirements  | 0.25     | 217.222               |                  |
|         |  |          | 23.110                |                  |
| 6/13/81 | Edit cost in work requirements                                       | 1.5      | 598.786               |                  |
|         | Generate work-requirements reports and add sections to work required | 1        | 258.121               |                  |
| 6/20/81 | Develop construction projects  | 0.25     | 187.407               |                  |
| 6/14/81 | Develop BMAR plan  |          |                       | 11               |
| 6/20/81 | Generate work requirements   | 0.5      | 116.445               |                  |
| 6/20/81 | Generate areas   | 0.5      | 29.177                |                  |
| 6/22/81 | Inspection   |          |                       | 2                |
| 6/24/81 | Inspection   |          |                       | 2                |
| 7/81    | Phase I and II of M&R project development                            | 120      |                       | 480              |

Note: BMAR = backlog maintenance and repair.

1. The installation was selected as an average installation so that the cost of the PET should be representative of the costs of implementing the system at other installations of similar size. However, the selected installation has used a manual management system over the past years.

2. The data processing equipment necessary to operate the automated system (ASCII terminal and acoustical modem) is purchased by the installation. The terminal cost is distributed over the systems supported by the terminal.

3. The data base is maintained for all installations by a single organization. The costs of management are split between installations for unit cost purposes.

4. No additional employees are needed at the installation level to operate the system.

5. PAVER offers the user more information and procedures than are currently available. These items are here considered benefits.

Constraints

The following constraints on the analysis should be noted:

1. The use of PAVER during the four-month PET is not necessarily proportional to a full year's use because different types of activities are required at certain times of the year. Therefore, the two analyses are performed as indicated under the heading "General" above.

2. Time estimates of activities during the PET were made while the PAVER system was in use. Thus, a true dichotomy of tasks was not possible.

Current Operating Method

The current method of operation at the installation is a manual card-file procedure. This method has been developed by the installation personnel and has been in operation for several years. The procedure basically consists of a card catalogue of pavement sections in which information on pavement structure and past major maintenance is recorded.

PET Data Comparison

The costs based on PET data for the current method consisted of 582.5 person-h (Table 2). These hours were split among three engineers, which resulted in an average rate of approximately \$15/h. If one uses the total hours given in Table 2, the current method cost is calculated to be \$8737 for four months, or approximately \$26 200/year.

Estimated Data Comparison

Activities performed during a normal year have been categorized into six groups. The time and costs for these categories are given in Table 3. The total

Table 3. Estimated data comparison of annual activities and costs for current operating method.

| Activity  | No. of Hours | Avg Hourly Rate (\$) | Cost (\$) |
|---|--------------|----------------------|-----------|
| Periodic pavement inspection                                    | 160          | 13.44                | 2 150     |
| Determination of M&R requirements and setting of M&R priorities | 240          | 15.74                | 3 778     |
| Validation of M&R projects                                      | 80           | 15.00                | 1 200     |
| Annual work plan  | 80           | 14.26                | 1 141     |
| Long-range planning   | 160          | 15.74                | 2 518     |
| M&R cost estimating   | 480          | 13.44                | 6 451     |
| Total   | 1200         |                      | 17 238    |

estimated annual cost is \$17 238/year. These costs are based on discussions with the Buildings and Grounds Division chief at the installation and the breakdown of the costs in Table 2.

Benefits

There are no tangible benefits associated with the current method of operation. However, there are certain intangible benefits associated with its continuation: (a) The current method is a local method that is user acceptable, and (b) no sophisticated equipment is required. These benefits, however, are particular to the test installation studied, since most other installations have no manual system.

Risks

If the current operating method continues, the following risks should be considered:

1. The number of projects not funded will most likely continue to rise, and the total dollar requirement for pavement maintenance will increase.

2. No common ground of communication will be established between the installation engineers and Major Command engineers.

3. No objective procedure for pavement rating will be established, which will reduce the chances for division of maintenance money based on the condition of the pavements.

4. Continual backup of work and inconsistent evaluation procedures will decrease pavement life.

Automated PAVER System

The PAVER system was fully implemented at the installation (i.e., all paved areas were inspected). The initial inspection and data input were performed under a lump-sum contract. The actual cost of this initiation along with operation costs from the PET will be considered.

PET Data Comparison

The "operating" costs from the PET for PAVER, as given in Table 2, are 175 person-h and 5796 ccu's for computer use. The cost of a person hour is again the average of \$15/h, which results in a four-month cost of \$2628, or approximately \$7886/year. The computer cost used was \$0.0765/ccu, as computed earlier. This yields a computer cost of \$443 for four months, or approximately \$1330/year. This is the actual computer time cost; there are also support costs associated with computer use. These can be itemized as follows:

| Item                           | Unit Cost               |
|--------------------------------|-------------------------|
| Connect time                   | \$8.50/h                |
| Tape storage                   | \$0.25/day              |
| Disc storage                   | \$22/1000 sectors/month |
| Communication line (telephone) | \$29/month              |
| Computer paper                 | \$21/box                |
| Equipment (terminal and modem) | \$1500                  |

The connect-time costs are based on Boeing Computer Service rates (the Corps of Engineers vendor at the time of the PET). The computer connect time for the PET was approximately 15 h. Based on the \$8.50/h rate, the connect time is calculated to be \$129 for four months, or \$387/year.

Tape storage was not used during the PET, so no tape charges are included for the PET analysis. No tapes were used in the PET as a matter of convenience.

Table 4. Summary of PAVER costs from PET data comparison.

| Item                           | Person Hours <sup>a</sup> | Avg Hourly Rate (\$) | Cost (\$)   |        |         |
|--------------------------------|---------------------------|----------------------|-------------|--------|---------|
|                                |                           |                      | Four Months | Annual | Initial |
| Labor                          | 175.25                    | 15.00                | 2629        | 7886   |         |
| Computer cou's <sup>b</sup>    |                           |                      | 443         | 1330   |         |
| Computer connect               | 15.13                     | 8.50                 | 129         | 386    |         |
| Disc storage                   |                           |                      |             | 1848   |         |
| Communication line (telephone) |                           |                      |             | 348    |         |
| Paper                          |                           |                      |             | 21     |         |
| Terminal equipment             |                           |                      |             |        | 500     |
| Initiation cost                |                           |                      |             |        | 92437   |
| Total                          |                           |                      |             | 11819  | 92927   |

<sup>a</sup>Four months.  
<sup>b</sup>At \$0.0765/ccu.

Table 5. Estimated data comparison of annual activities and costs for PAVER system.

| Activity  | Time (h) | Avg Hourly Rate (\$) | Cost (\$)         |
|---|----------|----------------------|-------------------|
| Periodic pavement inspection                                | 160      | 13.44                | 2150              |
| Determination of M&R requirements and setting of priorities | 96       | 15.67                | 1504              |
| Validation of M&R projects                                  | 40       | 15.00                | 600               |
| Annual work plan  | 40       | 14.26                | 570               |
| Long-range planning   | 24       | 15.74                | 378               |
| M&R cost estimating   | 120      | 13.44                | 1613              |
| One person year of FESA support, all bases                  |          |                      | 1600 <sup>a</sup> |
| Total   |          |                      | 8415              |
| Computer support  |          |                      | 2948              |
| Total   | 480      |                      | 11363             |

Note: FESA = Facilities Engineering Support Agency.

<sup>a</sup>Twenty five installations requiring one person year (GS-12) - \$26 951 x 1.5 (overhead) = \$40 000 per base = \$40 000/25 base installations = \$1600/ installation.

nience. The current disc storage charge is \$22/1000 sectors/month. The installation data base is approximately 700 sectors of disc space. This results in an annual charge of \$1848.

Since the Autovon telephone lines of the study installation will not support teleprocessing equipment, a commercial telephone line was necessary. The monthly charge for the service was \$29, or \$349/year. No long-distance service was required since the computer vendor has an "800" telephone number.

The terminal equipment for the PET was a Teletype 43 terminal with a 30-character/s acoustical modem. This equipment can be purchased for \$1500. Since the terminal supports three systems (two in addition to PAVER), only one-third of the cost was assigned to the PET as an initial cost (\$500). Approximately one box of paper per year is needed to support the PAVER system, at a cost of \$21. These costs are summarized in Table 4.

#### Estimated Data Comparison

In the case of the current operating method, the activities performed during a normal year have been categorized into six groups. The costs for these groups (see Table 5) were estimated through discussion with the Building and Grounds Division chief and the breakdown of costs and times given in Table 2. The annual cost has been divided into \$8415 for labor and \$2948 for computer costs. The computer support costs are calculated as follows:

Tape loading: (2 times/week) (52 weeks/year) (\$6/mount) = \$624.

Update tape: (2 tapes) (35 times/year) (\$6/mount) = \$420.

\$624 + \$420 = \$1044.

Tape storage: (2 tapes) (\$0.25/day) (365 days/year) = \$183.

On-line storage (disc space): \$22/1000 sectors/month.

Average data base size is 8000 sectors. Assume tape loaded to disc 2 months/year. Annual cost = (\$22) (8) (2) = \$352.

\$1044 + \$183 + \$352 = \$1579.

Phone line cost = \$348.

Paper cost = \$21.

Computer time cost = \$1000.

Total cost = \$1579 + \$348 + \$21 + \$1000 = \$2948.

To reduce overall costs, a tape mount system was assumed to be used in normal annual operation of the PAVER system.

Estimated costs for PAVER implementation can thus be summarized as follows (the initial costs are based on the initial cost of the PET given earlier in this paper):

| Category                     | Cost (\$) |
|------------------------------|-----------|
| Initial cost                 | 39350     |
| Annual labor cost            | 8415      |
| Annual computer support cost | 2948      |

#### Benefits

##### Tangible

Analysis of specific projects indicated that the use of PAVER could reduce the cost of maintenance and have an effect on long-term cost avoidance. One specific project was the Branch IWASN Section 04 (Washington Boulevard). As obtained from the installation Contracting Office, the bid price for reconstruction of this section was \$50 417.25. This section was scheduled for an overlay; however, based on its rate of deterioration (from a second pavement-condition-index inspection), it was estimated that the overlay would last only 5 years. The reconstruction, on the other hand, was estimated to have a design life of 25 years. The overlay price would have been approximately \$12 173 based on current competitive bid prices. Over the design life of the reconstruction, five overlays would have to have been placed, which would have resulted in a total cost of \$60 865 without inflation. This represents a cost avoidance of \$10 448. Other cost avoidances are likely to occur due to timely maintenance through the use of readily available information from PAVER. To quantify this cost avoidance, however, several years of data are needed. Therefore, a conservative cost avoidance of only \$10 500 is estimated to occur on an annual basis.

##### Intangible

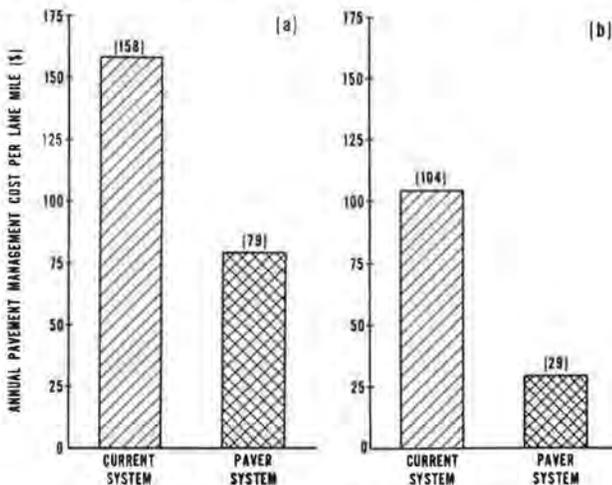
One of the major benefits of using the PAVER system

Table 6. Summary of economic analysis.

| Method                    | Inflation Rate (%) | Present Value (\$) | EUAC (\$) | EUAC per Lane Mile (\$) | Total Benefits (\$) | Present Value - Total Benefits (\$) | EUAC per Lane Mile Including Benefits (\$) |
|---------------------------|--------------------|--------------------|-----------|-------------------------|---------------------|-------------------------------------|--|
| Estimated Data Comparison |                    |                    |           |                         |                     |                                     |  |
| Current                   | 0                  | 101 160            | 18 962    | 89                      | 0                   |                                     |  |
|                           | 5                  | 117 850            | 22 090    | 104                     | 0                   |                                     |  |
|                           | 10                 | 137 904            | 25 849    | 122                     | 0                   |                                     |  |
|                           | 15                 | 161 955            | 30 358    | 143                     | 0                   |                                     |  |
| PAVER                     | 0                  | 106 027            | 19 874    | 93                      | 84 000              | 22 027                              | 20   |
|                           | 5                  | 117 027            | 21 936    | 115                     | 84 000              | 33 027                              | 29   |
|                           | 10                 | 130 246            | 24 414    | 115                     | 84 000              | 46 246                              | 41   |
|                           | 15                 | 146 099            | 27 385    | 129                     | 84 000              | 62 099                              | 55   |
| PET Data Comparison       |                    |                    |           |                         |                     |                                     |  |
| Current                   | 0                  | 153 752            | 28 820    | 136                     | 0                   |                                     |  |
|                           | 5                  | 179 120            | 33 575    | 158                     | 0                   |                                     |  |
|                           | 10                 | 209 600            | 39 288    | 185                     | 0                   |                                     |  |
|                           | 15                 | 246 156            | 46 140    | 218                     | 0                   |                                     |  |
| PAVER                     | 0                  | 162 286            | 30 420    | 143                     | 84 000              | 78 286                              | 69   |
|                           | 5                  | 173 729            | 32 565    | 154                     | 84 000              | 89 729                              | 79   |
|                           | 10                 | 187 479            | 33 142    | 166                     | 84 000              | 103 479                             | 92   |
|                           | 15                 | 203 969            | 38 233    | 180                     | 84 000              | 119 969                             | 106  |

Notes: EUAC = equivalent uniform annual cost.  
Data based on interest rate of 10 percent and analysis period of eight years.

Figure 2. Comparison of annual pavement management cost per lane mile: (a) PET data comparison and (b) estimated data comparison.



is that the Major Commands have a uniform method of comparing the pavements at all installations. This will help determine the distribution of maintenance funds and help establish an overall level of service for the installation. This uniform rating will also increase the communication between the Major Commands and the installation engineers. In addition, the pavement user will experience greater safety, comfort, and reduced vehicle maintenance because of better overall pavement condition.

At the installation and Major Command level, the PAVER system also adds a great deal of analytic power through programs such as ECON and the M&R Guidelines (1). From the PET data, it appears that a time savings of about 2.5 h can be expected for an economic analysis calculation. This reduced computation time is a benefit to the user. Increased accuracy of the analysis is also expected.

Major benefits experienced at the installation during the PET were that the work-requirements and M&R guidelines reports were of great use in developing contract documents. These reports provided quantities and cost estimates of the maintenance activities, which could take a considerable amount

of time to calculate by hand. The quantities were then used in the project preparation phase. The time savings are reflected in the last inputs in Table 2. It was felt that it would have been impossible to turn out the required number of year-end projects (36) without the PAVER system. Having the data stored saved a considerable amount of time in locating documents and reduced the time of field measurements, since only spot checking was necessary.

PAVER also offers the user access to factual data about the condition of the pavement system. Under the current system, these data are the subjective opinion of the pavement engineer. Provision of the objective data allows for more accurate calculations and sounder management decisions. The PAVER system will also provide a means for a new pavement engineer to become familiar with the overall network condition and inventory in a short time.

**Results**

The results of the economic analyses for the PET data comparison and the estimated data comparison are presented in Table 6. The present-worth analysis was performed for an eight-year analysis period, assuming a 10 percent interest rate. The analysis was repeated for inflation rates of 0, 5, 10, and 15 percent, respectively. The following is a brief definition of the terminology used in Table 6:

1. Initial cost--A one-time cost realized at the beginning of the analysis period,
2. Present value--The cost in today's dollars of the initial cost plus the discounted amount of future costs,
3. Equivalent uniform annual cost (EUAC)--The present value amortized over the analysis period (present value multiplied by capital recovery factor),
4. EUAC per lane mile--The EUAC divided by the total number of lane miles of pavement inventoried, and
5. Total benefits--The total amount of tangible benefits (in this case, cost avoidance) realized over the analysis period (the total benefits are not discounted).

**SUMMARY AND CONCLUSIONS**

This paper presents an economic analysis of the

PAVER prototype evaluation test at a military installation. Two analyses were performed: (a) an analysis based strictly on the data collected during the four-month PET (PET data comparison) and (b) an analysis based on average annual estimated data (estimated data comparison). The estimated data were based on the PET data and input from the Buildings and Grounds Division chief at the study installation.

The results of the economic analyses for the PET data comparison and the estimated data comparison are given in Table 6. Figure 2 graphically summarizes the results of the two methods of data comparison for an analysis period of eight years, an interest rate of 10 percent, and an inflation rate of 5 percent. The results of the PET data comparison clearly show that the annual cost of pavement management with PAVER is approximately 50 percent of the cost of the current system. The results of the estimated data comparison show that

the annual cost of pavement management with PAVER is approximately 30 percent of the cost of the current system.

#### ACKNOWLEDGMENT

The views expressed in this paper are ours and do not necessarily reflect the views of the U.S. Department of the Army or the U.S. Department of Defense.

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## Development of a Statewide Pavement Maintenance Management System

KAMRAN MAJIDZADEH, MICHAEL S. LUTHER, AND MICHAEL LONG

A framework for a statewide pavement maintenance management system (PMMS) is presented that describes the general system approach, criteria for monitoring pavement conditions, methods and types of data to be collected routinely to define pavement conditions, suggested computer data systems needed to support and implement the PMMS, and a system for establishing project priorities. The overall objective of such a system is to develop and implement maintenance management schemes for optimum selection of various repair strategies based on cost-effective analyses that consider repair needs and priorities. In this PMMS, criteria are applied to determine what portion of the total state network is likely to need maintenance and therefore should be monitored. Monitoring parameters in the form of maintenance needs indicators or "trigger values" are identified. These indicators—present serviceability index, skid number, age, and traffic—are also used to develop recommended sampling or monitoring frequency intervals. Procedures have been developed for nondestructive testing and analyses of structural remaining life for pavements that show structural distress of a certain extent and severity. These pavements are classified by using a visual pavement condition rating system, which provides a uniform method for assessing pavement conditions on a statewide basis. Finally, the PMMS includes a framework for establishing project priorities based on need and condition and presents guidelines to aid in formulation and evaluation of maintenance alternatives. The proposed system uses roughness, skid resistance, deflection, and traffic data currently maintained by most state transportation agencies. It is structured to facilitate implementation with minimal difficulty to a user agency and makes maximum use of the experience and judgment of agency engineers. Finally, it is modular, permitting easy future modifications and improvements to various aspects of the system as they become available.

Highway departments across the nation are experiencing serious monetary problems as aging highways and increasing rates of pavement deterioration are placing larger demands on pavement maintenance requirements. The nation's pavements are deteriorating faster than they are currently being rehabilitated, which results in increasing numbers of pavements needing repair. This situation is aggravated, to a large extent, by obsolete state funding structures that are unable to yield sufficient revenue in times of high inflation and reduced motor-fuel consumption as well as budgetary cutbacks

at the national and state levels. As a result, many states have been forced to defer such repair, thus allowing many pavements to deteriorate to an even poorer condition that makes future rehabilitation more extensive and costly.

In light of such fiscal pressures, most state transportation agencies have recognized the need to establish a systematic, rational procedure for identifying pavement repair needs and priorities and selecting cost-effective design alternatives. The pavement maintenance management system (PMMS) framework appears to be providing a solution to highway agency problems, and many agencies have already developed and implemented such systems as a management tool to aid in prioritizing those projects that are in need of rehabilitation (1-4). A PMMS also provides a medium for feedback in which the consequences of past actions can be incorporated into the decision process; it facilitates consistency and uniformity in funding allocations to optimize and achieve the best values possible for public funds and provides improvements and cost savings by means of improved organization and coordination of activities.

Although the primary benefit of a PMMS is economic, in that improved maintenance management will provide greater values for dollars spent, the system also enhances the opportunities for optimal, correct decisions; this leads to improved technology and efficiency for various activities and provides capability to defend funding allocations. It is intended to provide a means for presenting information on in-service pavements that can be used to identify needs and program investments as well as design and construction requirements.

The essential requirements of a PMMS include the capacity for updating and modification as new data and better models become available, incorporation of alternative strategies, identification of optimum

alternatives, and the capability for making decisions based on rational procedures. A thoroughly sophisticated and rational system can be formulated so that it provides its own lists of feasible strategies. A PMMS should provide the means for strategy evaluation and optimization and/or comparison of the consequences of the individual strategies.

Another major requirement of any PMMS is to identify condition parameters of a pavement system or network. In the system developed for the Ohio Department of Transportation (DOT), the parameters are skid number (SN), deflection, roughness, texture, and visual distress. The PMMS will provide information on the effect of maintenance activity on the condition parameters as well as the current values of these parameters. The condition parameters must be evaluated reliably and economically.

In a sophisticated PMMS, decisions are based on quantified standards and constraints and data base and information subsystems are an important part of the system. The information subsystem must separate the information needed for the network-level analysis from those data essential to project-level management. Caution is needed and steps must be taken to ensure that the PMMS is not drowned by the volume of data. One cannot expand project-level data bases to encompass the network.

The quality of a PMMS depends on the quality of the data and information available. A viable system should make optimum use of the types of data that are maintained or can be acquired by the user agency, including (a) field and laboratory data, (b) default values, (c) equipment and manpower, and (d) geometric, traffic, condition, unit cost, and materials data.

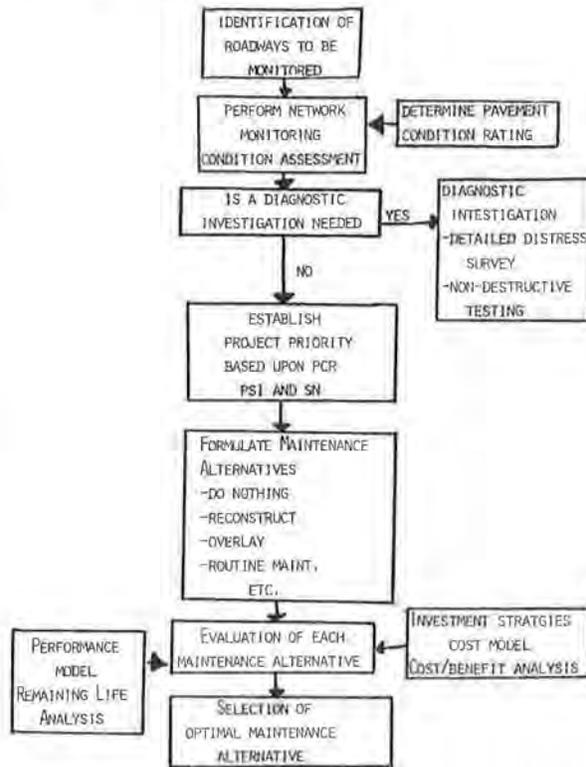
A PMMS is also considered a management tool for performance analysis and is designed to solicit subjective user responses. Therefore, the performance analysis is based, in part, on users' subjective views of the facility. This analysis is associated with serviceability-performance measurements and models.

**FRAMEWORK FOR A STATEWIDE PMMS**

In the current processes used by many state transportation agencies for programming pavement maintenance and improvement, engineering judgments and available funding are key elements. In many instances, no guidelines are available to district personnel regarding levels of deterioration, alternative repair strategies, or types of economic analyses to be considered in preparing maintenance recommendations. Standard repair policies or techniques have become accepted over the years without the use of documented performance data to indicate that the standard repairs perform satisfactorily. Criteria for identifying when nonstandard repair techniques should be considered are often lacking. Assessment of maintenance needs is sometimes nonuniform, since the judgment of one state district regarding pavement conditions may differ from that of another. This can result in maintenance not being performed for pavements where needs or cost benefits are the greatest and can thus produce nonuniform pavement conditions on a statewide basis.

The Ohio DOT has for more than a decade sponsored activities to develop and implement a comprehensive PMMS to minimize and/or eliminate the types of problems outlined above. Its primary research efforts have been directed toward developing a pavement monitoring system and prioritizing maintenance needs. As part of this research program, a recently completed study sponsored by the Ohio DOT (5) developed a proposed maintenance management program, shown in Figure 1, which incorporates the following components:

Figure 1. Framework of proposed PMMS.



1. Application of criteria to determine what portion of the total network is likely to need nonroutine maintenance and therefore should be monitored;

2. Procedures for nondestructive testing and analyses of structural remaining life for pavements that show structural distress of a certain extent and severity;

3. A framework for identifying alternative maintenance options and evaluating, when appropriate, standard maintenance options;

4. A methodology for selecting optional alternatives; and

5. A procedure for establishing project priorities.

The guidelines used in formulating the PMMS shown in Figure 1, which are considered essential to the development and implementation of a viable statewide PMMS, include the following:

1. The proposed system should use currently available user agency data, including roughness, skid resistance, deflection measurements, and traffic.

2. The proposed system should be structured so that it can be implemented with minimal difficulty by the user agency.

3. The system shall make maximum use of the experience and judgment of agency pavement engineers.

4. The system shall be modular to permit easy future modifications and improvements to various aspects of the program as they become available.

**PMMS COMPONENTS**

The PMMS developed as part of the Ohio DOT study (5) included four primary components:

1. Development of network monitoring criteria,

including monitoring parameters, maintenance-needs indicators or "trigger values", present serviceability index (PSI) and SN testing requirements and deterioration rates, and other parameters to be used for monitoring criteria;

2. A pavement condition rating (PCR) system, including a system for rating visual distress, field verification of the PCR, and identification of the need for structural investigation;

3. A system for determining project priorities and selecting the optimal repair method, including prioritization based on condition, formulation of maintenance alternatives and economic analysis, and selection of optimal maintenance alternatives; and

4. Suggested data systems, including pavement section files, pavement condition files, and scheduling.

These components are discussed briefly in the following sections.

#### Development of Network Monitoring Criteria

Monitoring criteria constitute that methodology or logic for deciding which portion of the total roadway network has deteriorated to the point where it is likely to need maintenance. Those criteria are a crucial part of the overall PMMS, since only those pavements identified by the monitoring criteria as being deteriorated will be considered by subsequent steps in the program. The criteria establish the type of pavement condition data to be collected for the network, identify data-collection intervals, and establish the magnitude of the data-collection program needed to support the management program. Two important aspects of the monitoring criteria are evaluating pavement conditions on a systemwide basis and defining unacceptable pavement conditions in terms of the parameters used to evaluate the pavements.

The PMMS included the identification of monitoring parameters and, as a key part of establishing monitoring criteria, a pavement evaluation and rating procedure was developed for defining unacceptable pavement conditions. Maintenance-needs indicators or trigger values were identified as SN, PSI, age, and traffic. Although it is not the principal purpose of a PMMS to identify slippery pavements, an SN of less than 30 was suggested for use in the monitoring criteria.

The research investigation indicated that PSI is a fairly reliable maintenance-need indicator or trigger parameter. The trigger values of PSI were selected on the statistical correlation of the estimated need for maintenance by district engineers and PSI values (based on roughness measurements) for various pavements in that district. Eighty-eight test sections were formed by combining continuous sections from roadways that had the same surface and base types and similar average daily traffic (ADT). Other criteria in defining the sections included a length of 1-4 miles, whether maintenance had been performed since the last PSI measurement (sections or portions of sections were omitted where overlays had been placed since the last PSI measurement), and a definite change in the visual condition of the pavement. In field visitations, district engineers were asked to answer yes, no, or maybe to the question, Will the pavement probably require some type of nonroutine maintenance within the next two years in order to maintain an acceptable level of serviceability and/or structural integrity? Based on the statistical relation between this estimated need for maintenance and measured PSI, trigger values for PSI were developed by using 1980 PSI data, as given below (an asterisk indicates facility types not

studied, for which data were extrapolated from data for high-type, two-lane roads):

| System                                  | PSI by Pavement Type |           |          |
|---|----------------------|-----------|----------|
|   | Rigid                | Composite | Flexible |
| Interstate                              | 3.30                 | 3.40      | 3.40*    |
| Multilane and high-type, two-lane roads | 3.10                 | 3.20      | 3.20     |
| Low-type, two-lane roads (ADT < 1000)*  | 2.90                 | 3.00      | 3.00     |

Since PSI and SN were to be used as the basis for roadway monitoring, these parameters must be routinely collected on an inventory basis for the entire network. Suggested sampling intervals or monitoring-frequency intervals were established by studying the decline of PSI and SN with traffic and age, as illustrated in Figures 2-4. Representative deterioration rates of PSI and SN were defined for each pavement type, and these rates were used to establish suggested sampling intervals by calculating the required amount of time needed to produce a "statistically significant" change in mean PSI and SN values by using the Student's t-test to calculate that time interval. The recommended test intervals for PSI and SN data collection developed in our study (5) are given in Table 1. Monitoring-frequency requirements should account for climatic and traffic variations experienced in different regions of the state. Finally, since many states do not yet have a fully implemented PMMS and obtaining and compiling PSI data for a statewide roadway network takes considerable time and funds, parameters other than PSI might also be used in the monitoring criteria. In the study cited above, it was found that age and traffic volume "B + C" could be used as trigger parameters in the absence of PSI data. For rigid pavements, it was found that traffic volume is a better indication of maintenance needs than age.

By incorporating PSI and SN deterioration rates into the PMMS, corrective action can be initiated for pavements with high deterioration rates before they decline to a poor condition and pavements with low deterioration rates can be investigated to identify designs that produced good performance. Deterioration rates can be analyzed on a district or statewide basis to identify those regions or routes that are experiencing the highest deterioration and to obtain an estimate of the magnitude of future maintenance needs. A summary of the monitoring criteria is presented in Figure 5.

#### PCR System

The monitoring phase of a PMMS program involves evaluation of the current physical condition of pavements in the field. A rating procedure, identified as pavement condition rating, was developed (5) that reflects the physical condition of the pavement. The PCR method is based on visual inspections of pavement distress and includes standard descriptions of distress types and the process for defining distress severity and extent. The Ohio DOT PCR system was developed after a review of visual rating systems developed by other agencies (1,6,7).

The PCR system yields a numerical index that reflects composite effects of distress types, severity, and extent on the overall pavement condition. The computation of PCR is based on the summation of deduct points for each type of observable distress. Total deduct points are subtracted from 100 to yield the PCR. The scale used for PCR is shown in Figure 6. A PCR of 100 is assigned to a pavement that has no observable distress. The deduction for each distress type is calculated by multiplying distress weight times the weights for the severity and extent

of the distress. Distress weight is the maximum number of deductible points for each different distress type. The PCR is calculated as follows:

$$PCR = 100 - \sum_{i=1}^n \text{deduct}_i \quad (1)$$

where n is the number of distress types and  $\text{deduct}_i$  = weight for distress x weight for severity x weight for extent.

Figure 2. PSI deterioration.

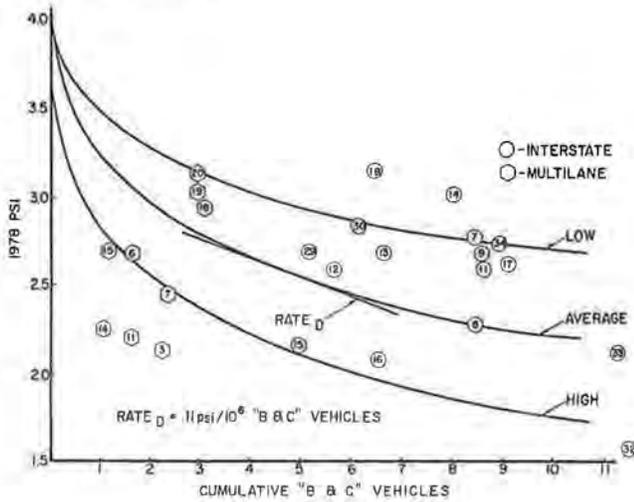


Figure 3. SN deterioration for asphalt-surfaced pavements.

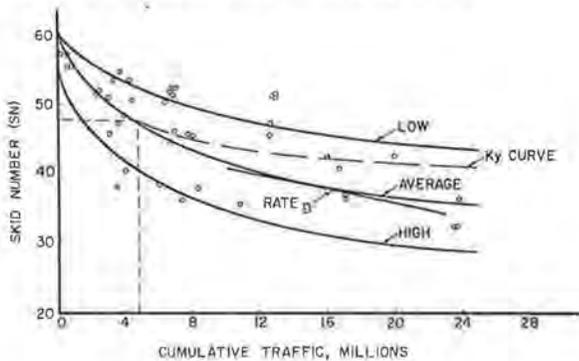
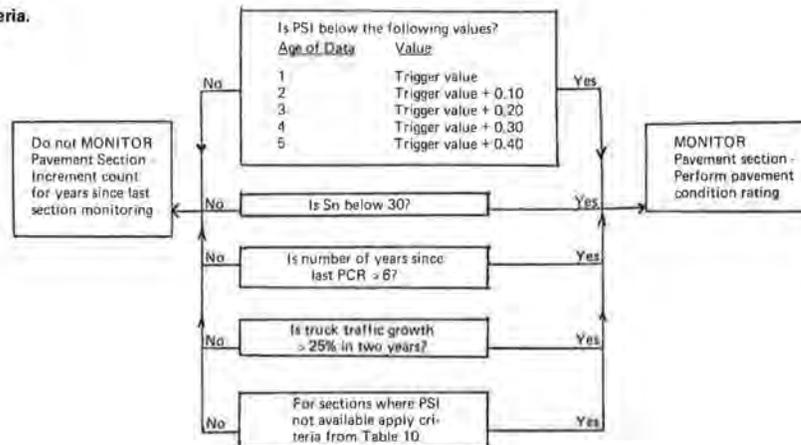


Figure 5. Summary of monitoring criteria.



The proposed PCR method developed (5) reflects the judgment of Ohio DOT engineers polled in the study and has been implemented in Ohio. However, as it becomes implemented on a statewide basis, expanded studies will be undertaken to measure test uniformity among PCR raters and to develop necessary adjustments to deduct values.

The Ohio DOT has incorporated a procedure through which the PCR system is used to identify and investigate pavements that may have inadequate load-carrying or structural capacity. Pavement sections that exhibit a certain degree of structural-related distresses and whose overall condition has reached a certain level of deterioration are investigated for structural integrity. The results of various research studies we have done over the past decade allow for the nondestructive evaluation of the structural condition of a pavement. The proposed PMMS program provides an opportunity for implementation of these nondestructive evaluation techniques (8,9). Diagnostic investigation criteria developed

Figure 4. SN deterioration for concrete-surfaced pavements.

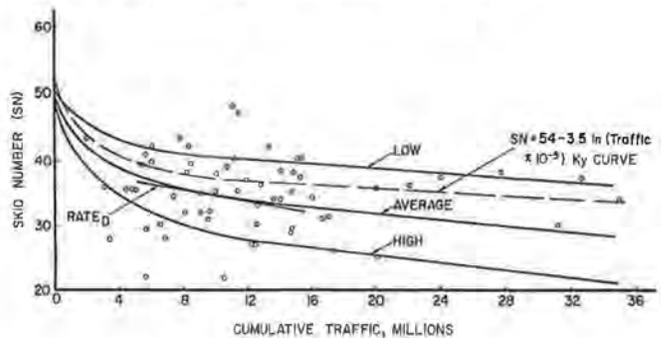


Table 1. Recommended sampling intervals.

| Maintenance Classification | PSI                                   | SN                                    |
|----------------------------|---------------------------------------|---------------------------------------|
| Interstate                 | Each direction every three years      | Each direction every two years        |
| Multilane                  | Each direction every four years       | Each direction every three years      |
| Two-lane High type         | Alternate directions every four years | Alternate directions every four years |
| Low type                   | Alternate directions every five years | Alternate directions every five years |

as part of the Ohio DOT study (5) are given below:

| System                            | Sum of Deduct Points for Structural-Associated Distress | PCR |
|-----------------------------------|---|-----|
| Interstate                        | >25   | <60 |
| Multilane and high-type, two-lane | >25   | <55 |
| Other two-lane                    | >30   | ≤50 |

System for Determining Project Priorities and Selecting Optimal Repair Method

Priority programming is probably the most important aspect of the PMMS. According to the proposed method, maintenance priority is established in terms of the condition parameters roughness (PSI), skid resistance (SN), and distress (PCR). Our research study (5) concluded that pavement distress should be considered most important in the priority system and roughness and skid resistance somewhat less important. Traffic is also considered as another parameter in the priority system. Urban districts generally consider traffic to weigh heavily in priority, whereas rural districts are less concerned about it.

The prioritizing procedures establish a maintenance urgency category (MUC), which is a simple method that simultaneously considers distress, roughness, and skid resistance to establish priority. Figure 7 shows the project prioritization criteria, and Figure 8 shows the determination of MUC. The table below defines the PCR groups:

| Group No. | PCR Value | Condition    |
|-----------|-----------|--------------|
| 1         | <40       | Very poor    |
| 2         | 40-55     | Poor         |
| 3         | 56-64     | Fair to poor |
| 4         | 65-74     | Fair         |
| 5         | 75-89     | Good         |
| 6         | 90-100    | Very good    |

Figure 6. PCR scale.

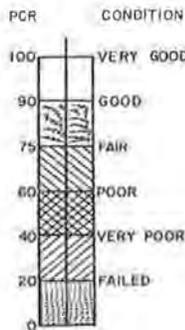
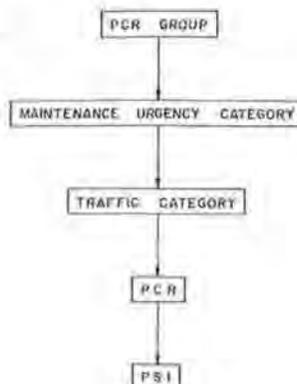


Figure 7. Project prioritization criteria.



For sections within the same MUC category, traffic is used to establish priority. For sections within the same PCR group, MUC, and traffic category, the actual values of PCR and, if necessary, the PSI are used to establish final section priority. The data given in Table 2 illustrate the priority system.

Suggested Data Systems

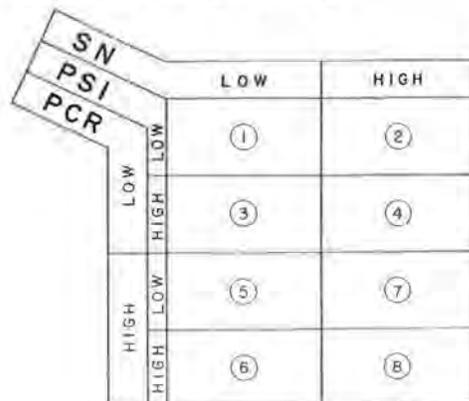
Implementation of both monitoring criteria and the condition rating aspects of a PMMS would require collection and analysis of large amounts of data. New data banks and additional programming efforts may be needed in some states to manipulate these data effectively and fully implement such a PMMS.

Three data files--the pavement section file, the pavement condition file, and the priority maintenance file--can constitute the primary data storage and retrieval program for a PMMS as described here. The pavement section file would contain data on section location, pavement type, traffic, age, and maintenance performed. Much of this information is usually available from current user agency files. The pavement section file would require program logic to create permanent sectioning of the roadway network; the logic should be similar to that currently used by the agency to define sections for skid testing and other purposes. Age of surface and maintenance class would also be contained in this file, which can be updated in the future as records of major maintenance performed are computerized.

The pavement condition file contains the PSI, SN, PCR, and deflection data for each roadway section and also program logic to determine PSI and SN deterioration rates for each section based on general trends. The pavement condition file would contain location data from the pavement section file and all information needed to identify pavement sections for monitoring, determined by using the monitoring criteria presented previously.

One important use of the pavement condition file would be to generate reports to assist maintenance engineers in evaluating repair needs. Such a report might essentially summarize all information col-

Figure 8. Maintenance urgency category.



LEVELS

PCR - Low is below 65 - Interstate  
 60 - Multilane & high-type two-lane  
 55 - Low-volume two-lane

PSI - Low is within bottom 10 percentile per maintenance class per pavement type statewide

SN - Low is <30

Rank within PCR group for each maintenance class:

- 1) Urgency category;
- 2) Traffic category within same urgency category;
- 3) PCR within same traffic category;
- 4) PSI for section with same PCR.

Table 2. Sample priority list for maintenance class 1, Interstate.

| Section No. | ADT per Lane | SN | PSI | PCR | PCR Group | MUC | Traffic Category | Ranking |
|-------------|--------------|----|-----|-----|-----------|-----|------------------|---------|
| 1350        | 5 000        | 40 | 2.5 | 80  | 5         | 8   | B                | 14      |
| 1352        | 9 000        | 33 | 1.6 | 57  | 3         | 2   | B                | 3       |
| 1354        | 3 000        | 53 | 2.2 | 69  | 4         | 8   | C                | 12      |
| 1356        | 7 500        | 23 | 1.6 | 49  | 2         | 1   | B                | 1       |
| 1357        | 2 200        | 59 | 2.6 | 71  | 4         | 8   | C                | 13      |
| 1359        | 10 000       | 28 | 2.5 | 72  | 4         | 6   | A                | 8       |
| 1360        | 3 500        | 57 | 2.5 | 74  | 4         | 8   | B                | 10      |
| 1362        | 5 500        | 49 | 2.4 | 62  | 3         | 2   | B                | 4       |
| 1364        | 6 100        | 29 | 2.2 | 68  | 4         | 6   | B                | 9       |
| 1366        | 7 200        | 42 | 2.5 | 64  | 3         | 4   | B                | 7       |
| 1367        | 4 200        | 54 | 2.0 | 53  | 2         | 2   | B                | 2       |
| 1369        | 1 700        | 36 | 1.6 | 60  | 3         | 2   | C                | 5       |
| 1381        | 2 900        | 55 | 2.2 | 67  | 4         | 8   | C                | 11      |
| 1383        | 12 000       | 29 | 2.3 | 59  | 3         | 3   | A                | 6       |

Note: Assume 10 percent PSI = 2.10.

lected by the maintenance management program about a particular roadway section.

The priority maintenance file would contain a listing of pavement sections and maintenance priorities established in accordance with the system presented previously. Section priorities can be assigned on both a statewide and districtwide basis, and sections should be listed by route so that district engineers can formulate maintenance projects by grouping together continuous sections of similar PCR groups or priority ranges. This file can be easily assembled by taking data from the pavement condition file and computing the priority by using the criteria shown in Figure 6. There should be cumulative mileage calculations for priority listings to enable early identification of total state or district network mileage for each PCR group or any given priority. Such listings should be completed by late fall or early winter of each year and be given to district personnel, together with the recommended trigger value of statewide priority ranking for maintenance planning, so that agency personnel can begin planning maintenance projects for sections that have priorities above the established value.

#### ACKNOWLEDGMENT

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## Prediction of Pavement Maintenance Expenditure by Using a Statistical Cost Function

SUE McNEIL AND CHRIS HENDRICKSON

Effective management and control of pavement maintenance expenditures are becoming increasingly important as the magnitude of these costs increases. The use of a statistical cost function as a means of inexpensively and quickly

forecasting the level of pavement maintenance expenditure is described. The statistical cost function predicts the level of real expenditures as a function of (a) traffic levels, measured in equivalent single 18 000-lb axle loads, and (b)

pavement age, measured as the number of years since the pavement was last resurfaced. Calibration of the cost model was performed for several turnpikes in the northeastern United States. The function was found to yield average errors of less than 10 percent in application to a turnpike section from 1956 to 1979 and also to an entire turnpike in 1980. Application of the cost function to different regions or roadway types may require parameter recalibration to reflect these different conditions.

In recent years, there has been a greater concern with planning and control of expenditures for roadway maintenance. There are several reasons for this interest:

1. Such expenditures are large; approximately \$94 billion was spent on highway maintenance and operation by all units of government in 1978 (1).
2. The paved highway system is not expanding as rapidly as in the past, so the average pavement is becoming older and, presumably, more expensive to maintain.
3. Budgets have been restricted in recent years, and this has spurred efforts to improve maintenance practices.

A critical element in the planning and control of maintenance expenditures has been the preparation of cost estimates. Traditionally, an engineering cost estimate of maintenance expenditure is used for estimating budgets and planning. Such estimates are obtained by summing the products of input quantities and their unit rates. For example, an organization might use the average cost per mile for shoulder maintenance multiplied by the number of miles of shoulders as part of an estimate of maintenance costs. These cost rates are derived from observed costs and quantities, are intended to be specific to a given situation, and rarely take into account factors such as weather, pavement age, and vehicle use. These cost estimates are calculated with the implicit assumption of linear proportionality between the input factors and the total cost.

A statistical cost function is an alternative for estimating maintenance costs for budgets and planning. This paper investigates the use of a statistical cost function for routine roadway maintenance based on turnpike data. This cost function relates roadway expenditures to traffic levels and pavement age, although other explanatory variables are considered.

Appropriate and accurate statistical cost functions would be quite useful in roadway management. First, they can be used to prepare cost estimates. Second, organizations are frequently faced with explaining large cost overruns, and a cost function may be used to indicate the origin of these overruns, such as particularly heavy traffic. Cost control may also be facilitated by checking that costs are not accumulating faster than scheduled or that a particular section of road does not have unwarranted costs, possibly due to mismanagement. Finally, the insights provided by a cost function are difficult to obtain any other way. The function captures the marginal effect of a change in any of the explanatory variables (such as roadway traffic) and the relations between such variables.

Despite these advantages, statistical cost functions have some limitations. The functions cannot be reliably extrapolated outside the range of the data used for calibration. When variables are not explicitly included, the transferability of the model is severely limited. For example, the use of turnpike data in this paper restricts the application of the cost function because each turnpike in the sample is always maintained to a high standard, maintenance is rarely deferred, and there are few

problems of underdesign or overloading. The model is only reliably transferable to roads that are comparably managed. Similarly, the model might not be directly transferable to areas that do not have climatic, topographic, or soil conditions similar to those of the areas used for calibration. However, coefficients may be estimated for these conditions with appropriate data.

The cost function presented in this paper for roadway routine maintenance models the aggregate costs of such activities as joint cleaning and sealing, crack filling, drainage maintenance, and minor patching. Essentially, these costs pertain to the road surface. Drainage maintenance is included because of the interrelation between the maintenance of adequate drainage and a good pavement surface. The data were obtained directly from turnpike records and include labor, materials, supervision, and equipment costs.

In this paper, these costs are modeled as an algebraic function of a series of explanatory variables that influence the extent of maintenance required. Broadly, the objective was to develop a function that can be used for forecasting routine maintenance costs and gives some insight into the relations between costs and explanatory variables. The model is also used to test the following hypotheses:

1. Maintenance costs increase as traffic increases, measured as larger vehicle volumes and heavier axle loads.
2. Maintenance costs increase as the pavement surface becomes older.
3. Maintenance costs increase in years or locations with more severe weather.

Although these hypotheses are widely accepted, estimation of cost models can yield numerical estimates of the amount of cost increases due to increases in traffic, pavement age, and roadway area.

The organization of the paper is as follows: (a) an examination of the trends in routine maintenance costs over time with and without inflation; (b) a discussion of the model formulation, including the appropriate factors to be included and the sources of data; (c) a description of the estimation results and their implications; (d) a description of the use of the model for prediction and some experiences in validating the model; and (e) conclusions.

#### TRENDS IN MAINTENANCE EXPENDITURE OVER TIME

Pavement maintenance expenditures have increased rapidly in recent years, but the extent to which various factors are responsible for this increase is not clear. The solid line in Figure 1 shows the cost per lane mile of maintenance for a 30.2-mile section of the Ohio Turnpike between 1956 and 1979. With few exceptions, the cost of maintenance has increased nearly every year. Over the 24-year period, expenditure increased by more than 500 percent, from \$644/lane mile in 1956 to \$3917/lane mile in 1979.

One major cause of this cost increase has been inflation in the prices of the labor and materials used in the maintenance process. The extent of this inflation can be seen in the increase in the Federal Highway Administration Highway Maintenance and Operation Cost Index (FHWA MOC) (1). This index indicates the relative costs of typical maintenance inputs in terms of the cost of these inputs in a base year, 1967. The price of these typical inputs has increased more than 260 percent during the period from 1956 to 1979.

Although inflation has been a major cause of maintenance cost increases, it has not been the only

Figure 1. Maintenance cost per lane mile for section of Ohio Turnpike from 1956 to 1979.

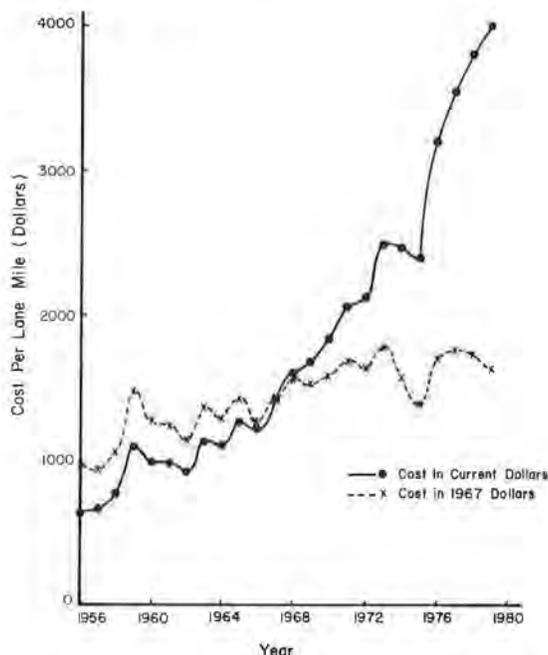
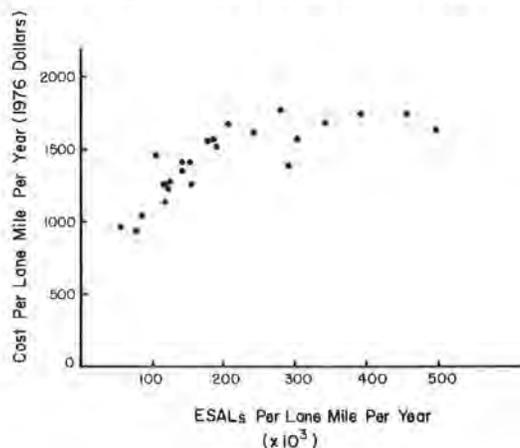


Figure 2. Maintenance cost per lane mile versus traffic using the road.



factor responsible. If the effects of input price inflation are removed by multiplying the costs in any year by the ratio of the FHWA MOC index in 1967 to the FHWA MOC index in that year, then maintenance expenses are converted to constant 1967 dollars. The broken line in Figure 1 shows the resulting trend in maintenance expenditures per lane mile in constant 1967 dollars on the same section of the Ohio Turnpike. Like the curve in which the effects of inflation are not removed, the costs tend to increase over time but at a much slower rate. Over the 24-year period shown in Figure 1, costs increase by 68 percent rather than by the 500 percent that occurs when the effects of input price inflation are not removed. In addition, the variation of these real costs is quite large from year to year.

Clearly, factors other than inflation influence the level of maintenance expenditure. For example, if the traffic that uses a section of road is found from historical records and converted to the number of 18 000-lb equivalent single-axle loads (ESALs)

that use the road by using the appropriate factors (2) to account for the relative damage caused by heavier vehicles, the cost of routine maintenance tends to increase with increasing ESALs, as shown in Figure 2. Unfortunately, graphical analysis is insufficient to capture the simultaneous effects of several factors; therefore, multivariate statistical techniques are used here to model the effects of various factors on maintenance costs.

STATISTICAL MODEL OF PAVEMENT MAINTENANCE COST EXPENDITURE

Numerous factors could be postulated as affecting the amount of pavement maintenance expenditures. Among these factors might be the current state of the pavement (including age, pavement type, and the adequacy of past maintenance), stress on the pavement (including traffic and weather effects), the procedures used for maintenance (including the amount of mechanization and the type of materials used), and various institutional factors (including type of management structure, availability of funds, and wage levels of workers). Due to lack of data and knowledge, it would be difficult to include all of these various factors in a cost model. Accordingly, only a few of the most important of these factors can be included. Other factors are either assumed to be constant (so that their influence does not change from one year to the next) or are of lesser importance in determining costs.

For the statistical cost model, the only explanatory variables used are those representing the traffic-related stress and the age of the pavement. As noted above, it is hypothesized that maintenance expenditures will increase with increases in either of these two explanatory factors, which are denoted AGE and ESAL. AGE, the age of the pavement, is measured by the average number of years since a pavement section was last resurfaced. This variable is a proxy for the deterioration of the pavement over time due to the action of the weather. ESAL, the number of equivalent 18 000-lb axle loads, is used as the measure of traffic-related stress. The axle-load equivalency factors (ESALs) represent the relative effects on the pavement of axle loads of different weights (2). The number of ESALs reflects the effects of both increased vehicle volumes and heavier axle loads.

Several other factors and different measures of traffic were explored as additional or alternative explanatory factors. A later section in this paper describes the results of these alternative model forms. As noted above, a major problem with adding these additional factors is obtaining adequate data and variation in the explanatory factors to identify their actual effects.

In order to calibrate the model of maintenance expenditure, detailed data on expenditures, traffic volume (in ESALs), and pavement age were required, even without considering additional factors. Unfortunately, the accounting practices and volume-counting programs of many highway agencies do not report these three pieces of information at a sufficiently disaggregated level to permit meaningful model estimation. Fortunately, many turnpikes routinely gather the required information. Traffic volumes by vehicle types and pavement sections are recorded as part of toll-collection records. Maintenance expenditures and dates at which resurfacing is undertaken are also available by section since the original construction of the turnpikes. The various sources of the calibration data are given in Table 1, where observations are assembled for each year and each roadway section. McNeil and Hendrickson (3) include a detailed description of the

Table 1. Sources of turnpike data.

| Turnpike      | No. of Sections | Years of Data | No. of Lanes | Total Length (miles) | No. of Observations |
|---------------|-----------------|---------------|--------------|----------------------|---------------------|
| Ohio          | 8               | 1956-1979     | 4            | 241                  | 192                 |
| Pennsylvania  | 5               | 1978-1979     | 4            | 469                  | 10                  |
| West Virginia | 1               | 1955-1979     | 2            | 88                   | 25                  |

sources and the preparation of the data.

The choice of a model form is also necessary in specifying a statistical cost function. A linear and a log-linear functional form are used for the cost function presented here. These forms may be interpreted as first-order approximations to any function, but both forms have some disadvantages. The linear function implies that there are constant marginal effects that are independent of the level of any variable. A marginal effect in this model is the change in real maintenance cost associated with a unit change in traffic or pavement age. The log-linear form, so called because it is linear in the logarithms of the variables, does not allow any of the variables to take a zero value.

#### ESTIMATION RESULTS

The models were estimated as follows: For the linear equation,

$$\text{COST} = 596 \text{ OH} + 3525 \text{ WVA} - 476 \text{ PA} + 0.0019 \text{ ESAL} + 21.7 \text{ AGE} \quad (1)$$

(1.50)      (5.29)      (0.35)      (3.93)      (1.93)

and, for the log-linear equation,

$$\ln(\text{COST}) = 4.22 \text{ OH} + 4.94 \text{ WVA} + 4.58 \text{ PA} + 0.37 \ln(\text{ESAL}/100) + 0.066 \ln(\text{AGE}) \quad (2)$$

(7.19)      (8.71)      (7.44)      (5.38)      (2.06)

where COST is the cost per lane mile (in 1967 dollars) for routine pavement maintenance; OH, WVA, and PA are dummy variables; and the remaining variables are as already defined. OH, WVA, PA take on the value one if the data refer to the Ohio, West Virginia, or Pennsylvania Turnpikes, respectively, and zero otherwise. For example, if the data are for a section of the Pennsylvania Turnpike, then OH = 0, WVA = 0, and PA = 1. The t-statistics are shown in parentheses under the appropriate coefficient in the above equations. For both models, the R<sup>2</sup> value was 0.88.

To account for variables not included and random effects, an error term is assumed in Equations 1 and 2 for calibration of the coefficients. These error terms were assumed to be first-order serially correlated (4). A first-order serial correlation implies that errors in one period carry over to the next. Various physical or economic effects cause this serial correlation. For example, the influence of bad weather in one year increases expenditures in that year and subsequent years. Similarly, the general economic climate may cause a reduction in expenditures from year to year. Statistical tests on preliminary estimates of the models indicated the existence of a first-order serial correlation of this type. The correlation coefficients for successive error terms were 0.93 for the linear model (Equation 1) and 0.78 for the log-linear model (Equation 2) in the final model. These correlations are useful for prediction, as described later in this paper.

Several of the estimation statistics reported above indicate the statistical properties of the models. The R<sup>2</sup> or goodness-of-fit measure is relatively high (R<sup>2</sup> = 0.88) for both models, which

indicates a reasonable level of explanation for variations in costs. With the exception of the constants for Pennsylvania and Ohio in Equation 1, the hypothesis that the coefficient equals zero (implying that no relation exists) can be rejected with a 95 percent confidence level for all coefficients based on the values of t-statistics.

The partial derivatives of Equations 1 and 2 with respect to each of the variables give the greatest insight into the model. For the linear model, these derivatives represent the change in cost for a unit change in the variable. In all cases, the signs of the partial derivatives are positive; so, as ESAL and AGE increase, the cost of maintaining a section increases. This result is consistent with the first two hypotheses proposed in the introductory section of this paper; the third hypothesis is discussed in the next section. The equations do not permit economies of scale to be assessed with respect to the number of lanes and the length of the section, since multicollinearity prevented the separation of these effects.

The constants may be interpreted as fixed costs, such as supervisory costs. Different constants were estimated for each turnpike to represent the differences in management and scale. In the case of Pennsylvania, the low t-statistics for the constant and the fact that only 10 observations were available prevent any reliable conclusions being made regarding the differences in constant costs between the turnpikes.

The equations indicate that, as the age of the pavement increases, and the road is subjected to the cumulative effects of weather and time, the cost of maintenance increases. Simply stated, the linear equation (Equation 1) implies that, for each additional year of pavement age, the maintenance costs increase by \$21.7/lane mile. In the log-linear equation (Equation 2), the marginal increase in maintenance cost varies with the pavement age, the length of the section, and the traffic, but an X percent increase in age will result in an increase of 0.066 x X percent in cost. For example, suppose a section is four years old and costs \$2000 annually to maintain. Without considering serial correlation, one would expect that in another year the maintenance cost would increase by \$22 if the linear equation were used and \$33 if the log-linear equation were used, due to increased pavement age.

Figure 3 shows the cumulative increases in maintenance costs over 10 years due to the increase in age alone for initial maintenance costs of \$1000, \$2000, and \$3000 annually. Figure 3 also shows the disadvantages of both the linear and log-linear forms: The linear form has constant slope whereas the log-linear form is only valid for pavements that are more than a year old. In effect, the linear model represents the effect of age on average pavements, since no interactive terms between age and other variables are included.

Similarly, the equations also indicate that maintenance cost increases with increased vehicle weights and volumes, expressed in terms of ESALs. Figure 4 shows a graph of the maintenance cost for the linear and log-linear equations for an arbitrary section of the Ohio Turnpike over a range of typical ESALs.

Both the linear and log-linear equations, as approximations to the true functional form of the cost function, verify the first two hypotheses as originally proposed. The graphs included in this section illustrate the implications of the functional forms.

#### ALTERNATIVE MODELS

In estimating Equations 1 and 2, several alterna-

Figure 3. Cumulative increases in maintenance cost due to age.

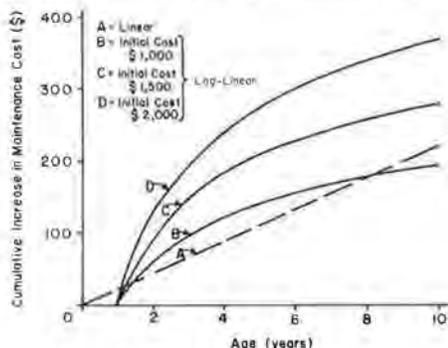
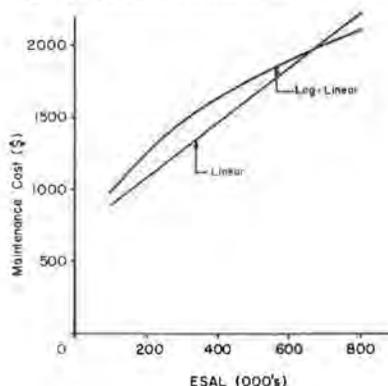


Figure 4. Maintenance cost versus traffic determined by using linear and log-linear equations for maintenance costs.



tives to ESAL values were used as indicators of traffic and additional variables were also included to account for weather. The models represented by Equations 1 and 2 gave the most reasonable fit, but the alternatives warrant some discussion.

ESALs were chosen as the best single measure of traffic that accounts for increases in both traffic volume and traffic weight. The total number of vehicles and the number of passenger-car equivalents (PCEs) using the pavement were also used. The model that used number of vehicles did not produce a good fit, and, although the fit with PCEs was comparable to that with ESALs, ESALs were believed to be a more appropriate measure of traffic. One reason for the comparable fit with ESALs or PCEs was that these two traffic measures were highly correlated in the calibration data set.

The problem of choosing a single measure for traffic could be overcome by estimating specific coefficients for each vehicle class. Unfortunately, there was insufficient variation in the data to permit accurate estimation of such coefficients. ESAL miles of travel seems to be a reasonable traffic measure, since it reflects both pavement stress and amount of traffic.

Equations 1 and 2 were also estimated so as to include two variables to account for weather effects: the number of freeze-thaw cycles (FR) and the amount of precipitation (PR) in each year. These variables were estimated from records of meteorological stations near each pavement section. Although the coefficient estimates had the correct signs and good t-statistics for these variables (except for FR in the log-linear equation), the R<sup>2</sup> only improved marginally (an increase of 0.006 for the linear equation and 0.008 for the log-linear equation) and

large changes in FR and PR resulted in only small changes in cost. Typically, a 10 percent change in either PR or FR implied less than a 1 percent change in cost. The variables FR and PR were not included in the final model because they pose a prediction problem for forecasting and show very little variability in the data used for calibration. Most of the variation in FR and PR occurs between states and is accounted for by the constants.

In addition to the two factors included in the final model—pavement age and traffic—other factors might also be expected to affect the level of pavement maintenance expenditure. Several of these factors were described earlier. Unfortunately, additional factors could not be added to the cost model because of the unavailability of data or the lack of variation of these factors in the calibration data set. In the latter case, the effect of the factor could not be distinguished from the constant term in the cost models. In transferring the cost models to different regions or pavement types, parameters in the model should be recalibrated to ensure good fit because these constant factors might be expected to change and thereby affect the amount of maintenance expenditure.

PREDICTION WITH THE COST FUNCTION

Prediction with either the linear or the log-linear equation may be carried out in two ways. The first method is the most obvious: Substitute for the expected values of the explanatory variables, AGE and ESAL, and obtain the predicted maintenance cost, as illustrated in the following example.

Suppose that an estimate of maintenance costs for section 1 on the Ohio Turnpike is required for 1980. In 1979, the average pavement age on section 1 was 4.8 years and the traffic level was 500 300 ESALs. In 1980, the pavement will be one year older (AGE = 5.8), and we shall assume that the same level of traffic occurs (ESAL = 500 300). With these values, the predicted maintenance expenditure in 1980 is

$$\tilde{COST} = 596 + 21.7 * AGE + 0.19 \times 10^{-2} ESAL = \$1672 \tag{3a}$$

by using the linear Equation 1 and, taking exponentials on Equation 2,

$$\tilde{COST} = 12.4 ESAL^{0.37} AGE^{0.006} = \$1789 \tag{3b}$$

by using the log-linear equation.

Simply substituting for ESAL and AGE results in predictors that are unbiased but not "best" in the sense that the variance is large. This method does not account for serial correlation of the error terms (i.e., "carry-overs" from year to year), but it is inappropriate for a "ball-park" figure.

A more refined prediction estimate may be obtained by considering the serial correlation, but at the cost of more computational effort. Assuming year n was the last year for which actual costs are known, the predicted cost in year i ( $\tilde{COST}_i$ ) is given by

$$\tilde{COST}_i = \tilde{COST}_i + \rho^{i-n} (COST_n - \tilde{COST}_n) \tag{4a}$$

when the linear equation is used and by

$$\ln \tilde{COST}_i = \ln \tilde{COST}_i + \rho^{i-n} (\ln \tilde{COST}_n - \ln \tilde{COST}_n) \tag{4b}$$

when the log-linear equation is used, where

$$\tilde{COST}_i = \text{best linear unbiased estimate of cost in year } i;$$

$\hat{COST}_i$  and  $\hat{COST}_n$  = predicted values of cost in the appropriate years, obtained by substituting for the explanatory variables;  
 $COST_n$  = observed value of cost in year  $n$ ; and  
 $\rho$  = correlation coefficient.

For the example described above, if it is known that  $\rho = 0.93$  for the linear equation and 0.78 for the log-linear equation and  $COST_{79} = \$1354$ , then, by using the linear equation,

$$\begin{aligned} COST_{79} &= \$1354, \hat{COST}_{80} = \hat{COST}_{80} + \rho(\hat{COST}_{79} - COST_{79}) \\ &= \$1672 + 0.93(1354 - 1656) = \$1391 \end{aligned}$$

and, by using the log-linear equation,

$$\begin{aligned} \ln \hat{COST}_{80} &= \ln \hat{COST}_{80} + \rho(\ln \hat{COST}_{79} - \ln COST_{79}) \\ &= 7.49 + 0.78(7.21 - 7.48) = 7.48 \end{aligned}$$

or  $\hat{COST}_{80} = \$1454$ .

After the functions were calibrated by using the data summarized in Table 1, two additional sets of expenditure observations were used to test the predictive ability of the models. Observations from 1956 to 1979 for the last maintenance section on the Ohio Turnpike (section 8) were kept as a hold-out sample to test the transferability of the model to locations similar to those used for estimation. Similarly, observations for all eight sections for Ohio for 1980 were obtained to test the usefulness of the model for forecasting maintenance costs for the sections used for calibration.

For section 8 of the Ohio Turnpike, fitted values were obtained by using the method described above and assuming that the actual cost in the previous year was known. The linear form resulted in an average absolute error in prediction of 9.5 percent for section 8 over the 24-year period. Similarly, the log-linear form resulted in an average absolute error of 8.5 percent. These errors indicate that the model is transferable to sections that have similar climate, traffic, and maintenance standards and that both equations produce similar predictions.

Two types of fitted values were calculated to compare with the 1980 data. The first is an estimation of the cost given the traffic in 1980, and the second is an estimate based on expected traffic for 1980. In practice, the latter method would be used since traffic levels are not known in advance. For illustrative purposes, a linear trend based on the traffic in 1978 and 1979 is used to estimate the 1980 traffic.

The average percentage prediction error and the average absolute percentage prediction are simple measures of the goodness of fit of the equations. The average percentage prediction error is an overall measure of the predictive ability. It is calculated by averaging the percentage difference between the actual and predicted values. The average absolute percentage prediction error is a measure of the magnitude of the expected prediction error in each section for each year. It is calculated by averaging the absolute value of the percentage difference between the actual and predicted values. If the equations underpredict some sections and overpredict others, then the average error of predictions will tend to be zero, whereas the average absolute error of prediction indicates the actual magnitude of the errors.

Assuming the traffic was accurately predicted or known, the average absolute error in predicting the maintenance cost was found to be 5.4 percent compared with an average absolute error of 9.6 percent

in the Turnpike budget estimate (converted to 1967 dollars). All of these reported errors have been calculated by using the linear equation, but similar results can be obtained by using the log-linear equation.

By using the predicted traffic, the average absolute error of prediction is 9.1 percent and the average error of prediction is 8.1 percent. These errors are lower than those of estimates made by the Turnpike authority, which indicates that the model can be used successfully to predict maintenance costs. When either the known or the predicted traffic for 1980 was used, the average error of prediction was also below the error in the Turnpike budget estimates. Results are similar for both the linear and log-linear models.

The comparison of errors described here is in terms of real (1967) dollars. In practice, forecasts of the rate of inflation are also uncertain, so budget forecasts in current dollars have an additional source of error. For 1980, however, a simple quadratic projection of the FHWA MOC to 1980 resulted in a negligible error, and the errors reported here would be similar for real or current dollar forecasts.

## CONCLUSIONS

This paper has discussed the possible uses of statistical cost functions, described the trends that have occurred in routine pavement maintenance costs on the Ohio Turnpike over the past 24 years, and developed statistical cost functions for routine maintenance expenditures. The cost functions may be used to forecast expenditure in real dollars based on the age of the pavement and the level of traffic on the roadway. The linear and log-linear model forms gave comparable results.

In addition to traffic levels and pavement age, there are numerous other factors that influence the level of maintenance expenditure. However, several validation exercises suggest that the simple cost function may be adequate for many managerial purposes. When used to predict expenditures on a section of the Ohio Turnpike and to forecast expenditures on all sections of that Turnpike, cost functions yielded average absolute errors of less than 10 percent. Recalibration of the model parameters may be necessary in applying the models to different roadway types or circumstances.

Our conclusion is that statistical cost functions can be an effective mechanism for predicting maintenance costs. Highway agencies may consider actions to gather appropriate data and estimate similar models.

## ACKNOWLEDGMENT

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