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

GEORGE B. WAY, JOHN EISENBERG, AND RAM B. KULKARNI

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 fourto 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 then 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 determine 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 (ΔR) 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:

$$\Delta\%C_n = 0.55(\Delta\%C_p) + 0.031(\Delta\%C_p{*}\%C) + 0.01(R_g)^2 + 0.05(R_g{*}\%C) - 0.0059(\%C)^2 + 0.186 \quad R^2 = 0.70 \quad SE = 0.64 \quad F = 84 \tag{1}$$

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:

$$R_n = 0.138(R) + 2.65(R_g)^2 - 0.047(R_g{*}R) - 0.125 \quad R^2 = 0.54 \quad SE = 10.4 \quad F = 38 \tag{2}$$

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:

$$\Delta\%C_n = 0.51 + 0.069(\%C) + 0.52(\Delta\%C_p) - 9.9934(D_L)^2 - 0.003(\%C)^2 + 0.068(\Delta\%C_p)^2 \quad R^2 = 0.68 \quad SE = 0.71 \tag{3}$$

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 Desert	Low ADT Transition	Low ADT Mountains	Medium ADT Desert	Medium ADT Transition	Medium ADT Mountains	High ADT Desert	High ADT Transition	High ADT 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:

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

where

- R_N = change in roughness one year after an overlay (typically a negative number, which is added to R_3 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 Actual	Ride Calculated	Percentage Cracking Actual	Percentage Cracking 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	Calculated Ride Given 1973	Calculated Ride 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	Calculated Percentage Cracking Given 1974	Calculated Percentage Cracking 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^2	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^2	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^2	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^2 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^2 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^2	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^2	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^2	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^2	0.6239	0.8993
	SE	13.6	0.4
	A	-7.2	0.1
	B	1.23	0.95
	Coefficient of variation	17	18
2	R^2	0.4489	0.9257
	SE	22.3	1.2
	A	35.6	-0.7
	B	0.57	1.1
	Coefficient of variation	23	16

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