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Contents

Foreword

Thirteen peer-reviewed papers appear in this publication, the first 12 of which were presented at two sessions sponsored by the TRB Committee on Pavement Management Systems, during the 1995 Annual Meeting.

Chen et al. describe a computer tool to assist engineers in developing pavement performance models for use in pavement management systems. Sebaaly et al. develop flexible pavement maintenance performance models from actual performance data. Wang and Zaniewski conduct sensitivity studies to demonstrate relationships among prediction models and rehabilitation needs and pavement conditions, which reveal large future savings in pavement rehabilitation through application of effective preventive maintenance actions. Kannemeyer and Visser evaluate the applicability of the HDM-III performance models to the development of a balanced maintenance expenditure program for roads in South Africa. Fekpe et al. present parameters for evaluating pavement loading impacts of alternative truck weight limits and enforcement levels. Mishalani and Koutsopoulos develop a spacial distress model to assess the condition of the infrastructure and its deterioration. Sekiguchi et al. use field surveys and statistical methods to analyze pavement structure data in Tokyo's pavement management system data base. Shahin et al. investigate the effects of altering the sample unit size and of reducing the number of distress types on the validity of the Pavement Condition Index of asphalt surfaces. Andres and Turo report on the partnering efforts between the Massachusetts Highway Department and the state's metropolitan planning organizations toward developing appropriate interagency pavement management systems. Mijuskovic et al. analyze the influence of staged pavement construction on the condition of the total pavement network. Broten and McNeely report on the history, usefulness, and success of Virginia's pavement management system for airfields. Wang et al. discuss the development of a user-friendly, graphical, interactive, multimedia pavement management system. Omar et al. present a model system for managing Japan's aging infrastructure considering direct impacts on users and indirect impacts on the economy. Readers of this record may wish to be aware that the proceedings of the TRB-sponsored Third International Conference on Managing Pavements, May 22–26, 1994, in San Antonio, Texas, are available from TRB.

Pavement Performance Modeling Program for Pennsylvania

XIN CHEN, STUART HUDSON, GAYLORD CUMBERLEDGE, AND ERIC PERRONE

The Pavement Performance Modeling Program (PPMP) developed for the Pennsylvania Department of Transportation by Texas Research and Development Foundation is described. PPMP is a MicroSoft Windows-based computer tool to assist engineers in developing pavement performance models for use in pavement management systems and updating these models annually as new data are input into the data base. The program can build both deterministic models and probabilistic models for an individual or group of pavement segments for each maintenance and rehabilitation (M&R) treatment. The program allows the user to define pavement performance indexes, grouping variables, and M&R treatments. Grouping variables are those that influence performance and are thus accounted for in the analysis. They include annual average daily traffic or equivalent single axle load (ESAL), functional class, pavement structure depth, maintenance level, and others at the discretion of the user. For deterministic models, five forms of equations are included. The independent variables can be pavement age or ESAL. Data and models can be plotted on screen and analyzed. The results from sample runs are presented and discussed.

Modeling of pavement performance is absolutely essential to pavement management on all levels, from project to national network (*1*). Pavement performance models can be developed based on pavement historical data. It is realized that many factors (i.e., pavement surface type, maintenance and rehabilitation (M&R) treatment, traffic, subgrade type, construction material, maintenance level, environment, climate etc.) have effects on pavement performance.

Pavement performance models can be broadly divided into group models and segment models. A group is a set of pavement segments defined by one or more variables. These variables are called performance grouping variables. For example, if pavement type and annual average daily traffic (AADT) are selected as grouping variables, pavement type is divided into two levels, flexible and rigid; and AADT into three levels, light, medium, and heavy, giving a total of six groups ($2 \times 3 = 6$). In terms of the analysis methods used in modeling, performance models can also be divided into deterministic models and probabilistic models. In pavement performance modeling, the most popular method for building deterministic models is regression analysis. For probabilistic models, the Markov chain process is the preferred method.

Pavement performance prediction is the most technologically difficult portion of pavement management (*2*) because of (a) the uncertainties of pavement behavior under changeable traffic load, environment etc., (b) the difficulty of quantifying many factors affecting pavements, (c) the error associated with using discrete testing points to represent the total pavement area when estimating pavement condition, and (d) the nature of the subjective condition survey. To develop the best models from the available data and update these models as more data become available is one of the most important tasks for engineers and researchers in pavement management. The development of pavement performance models involves extensive effort to create a data file (or a data base) by joining and calculating data from original data files. Currently, most researchers use a single model form to produce pavement performance models for all types of pavements (*2–5*). One reason is that no specific software has been available to allow relatively easy manipulation of a historical data base and development of models. A single model form may produce reasonable results, but may not get the best results due to the nature or variability of pavement performance in the real world.

The MicroSoft Windows-based Pavement Performance Modeling Program (PPMP) has been developed for the Pennsylvania Department of Transportation (PennDOT). The program provides a computer tool to assist PennDOT engineers in developing pavement performance models for use in their pavement management system (PMS) and updating these models annually as new data are input into the data base. There are five basic forms of models included in the program, They allow the user to try different types of models and select the best fit model for a specific situation. In this paper, the data used for developing PPMP is discussed, the components of the program are presented, the procedure used to produce performance models for PennDOT is described, and the models developed from sample data are analyzed and evaluated.

DATA DESCRIPTION

The road network of the Commonwealth of Pennsylvania is divided into approximately 150,000 road segments. Most data are stored in the Roadway Management System and the Maintenance Operations and Resources Information System. PennDOT uses IBM's Information Management Systems as its primary data base management system with MVS/ESA as the operating system. RMS contains 32 data bases and over 600 computer programs that generate 221 different inquiry and data input screens at computer terminals throughout the state.

The required data files for pavement performance modeling include (a) segment inventory, (b) pavement rehabilitation history, (c) asphalt concrete (AC) surface condition, (d) portland cement concrete (PCC) surface condition, and (e) pavement-related minor maintenance. Table 1 lists the data used for pavement performance modeling.

The pavement history file stores up to 10 layers of information. There are more than 200 surface, base, and subbase types coded in the file. The distresses stored in both the AC pavement condition file and PCC pavement condition file are two-digit codes representing

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TABLE 1 Data Used in Pavement Performance Modeling Data

Data Table	Data Used In Pavement Performance Modeling
Segment Inventory	length, width, lane count, federal functional class, truck percent, AADT, ESAL
Pavement History	layer year, layer code, layer thickness
AC Surface Condition	excess asphalt, raveling/weathering, block cracking, transverse/longitudinal cracking, alligator cracking, edge deterioration, bituminous patchings, potholes, widening drop-off, profile distortion, IRI, PSR
PCC Surface Condition	joint seal failure, longitudinal joint spalling, transverse joint spalling, faulting, bloken slab, bituminous patch, surface defects, rutting, IRI, PSR
Maintenance Activity	activity year, activity code

the severity and density of the distress. A total of 23 activities, from patching to surface treatments, are coded in the minor maintenance file. All condition survey data from 1983 through 1993 (approximately 1.5 million records) are available for performance modeling.

SYSTEM COMPONENTS

The project has three major objectives: (a) create a research data base so the modeling can be done efficiently and effectively, (b) develop statistical analysis procedures for developing various types of models, and (c) design user-friendly user interface so different approaches can be tried to obtain the best model fitting a specific data set. To achieve these objectives, six modules are designed for PPMP: user definition, data base, method base, modeling, analysis, and output. Figure 1 illustrates the components of the program.

User Definition Module

The user definition module defines (a) the deduct values for converting distress severity and density codes into individual distress indexes, (b) M&R treatments and maintenance level, (c) performance indexes, and (d) grouping variables.

In this module, distress codes are converted to individual distress indexes when the raw data are imported to PPMP. M&R treatments can be classified generally as thin overlay, medium overlay, and thick overlay or reconstruction, or specifically as detailed surface material types. Maintenance levels can be divided into no maintenance (Level 1), some maintenance (Level 2), or heavy maintenance (Level 3) between two major rehabilitation treatments. Pavement performance can have a single index (such as a cracking or rutting index) or composite indexes (such as an overall pavement index). The selection of grouping variables is essential to performance modeling in that it determines the number of models and the significance of the models to some extent.

To be flexible, models can be built to reflect county by county, district by district, a mix of counties and districts, or the whole state. The advantage of dividing the state into small regions such as counties or districts is that climate factor can be taken into account indirectly, since the climate of the whole state is more diversified than that of a county or a district. The disadvantage is that more computation effort is needed, and, in some cases, no model can be obtained from lack of data.

Data Base Module

Currently, the five data files used for modeling are downloaded from the PennDOT primary roadway data base in ASCII format and then imported to PPMP. A research data base is created and may be updated on a year-by-year basis. In addition to the original five files, the PPMP data base consists of more than 30 additional data files, such as distress deduct values, performance indexes, grouping variables, performance models, and so forth. A master file is created by joining and calculating the data from the original five files.

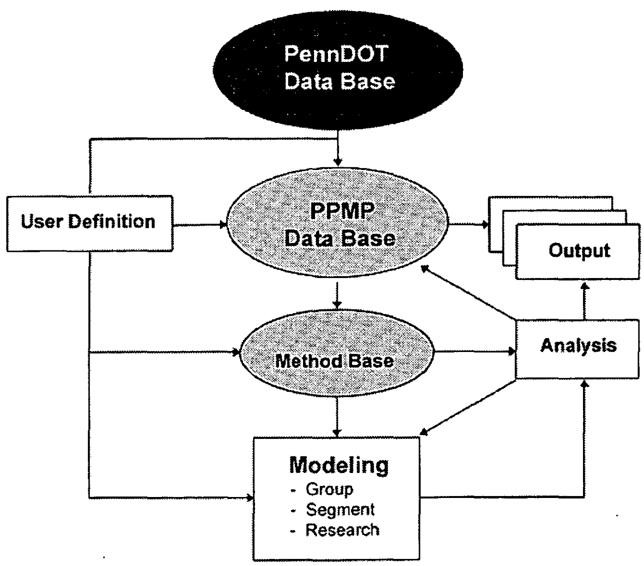

FIGURE 1 Components of PPMP.

Method Base Module

The method base is the key module of PPMP. It is the collection of various statistical analysis methods for pavement performance modeling. The current version of PPMP is composed of the following statistical methods: (a) least squares and constrained least squares methods for generating deterministic models, (b) probabilistic analysis for building Markov chain models, and (c) optimization algorithms for selecting best models. The method base allows the user to try different data transformation methods and types of models to get the best models possible.

Modeling Module

The modeling module provides two ways to build pavement performance models: group models and research models. A group model can be built once for all performance indexes, groups, and M&R treatments. A research model can be built with any combination of variables, for example, interstate highways, flexible pavements, heavy maintenance, AADT from 5,000 per lane to 10,000 per lane, and so forth. In any case, for deterministic models, the independent variables can be either age or cumulative equivalent single axle load (ESAL). Currently, the following forms of models are included in the program:

$$y = \alpha + \beta x \quad (1)$$
$$y = \alpha e^{-\beta x} \quad (2)$$
$$y = 100 - \alpha e^{\beta x} \quad (3)$$
$$y = 1/(\alpha + \beta x) \quad (4)$$
$$y = \alpha + \Sigma \beta x^i \; (i = 2 \ldots) \quad (5)$$

where

y = performance index;
x = independent variable, either pavement age or cumulative ESAL; and α, β, and β_i $(i = 2 \ldots)$ = model parameters to be estimated.

The user of the program can build the foregoing models for any set of data; the model that fits the data best can then be selected.

For probabilistic modeling, the Markov chain model is included in the current version of the program. In building the Markov chain model, each performance index can be divided into a maximum of five states (e.g., excellent, good, fair, poor, and bad). It is assumed that pavements can change only to an equal or worse condition under routine maintenance in a period of 1 year (i.e., routine maintenance cannot improve the condition). The following equation is used to estimate the transition probability of the Markov chain model for any performance index after an M&R treatment is performed:

$$p_{ij}(k) = m_{ij}(k)/n_i(k) \quad (6)$$

where

$p_{ij}(k)$ = transition probability from state i to state j after M&R treatment k;
$m_{ij}(k)$ = number of segments moved from state i to state j in a period of 1 year after M&R treatment k; and
$n_i(k)$ = number of segments in state i before M&R treatment k.

Analysis Module

The purpose of the analysis module is to plot the raw data and the models built by the modeling module; analyze the data, outliers, and models; and select the best model. In some groups, for some performance indexes, the models built from the available data may be unrealistic. This module provides a practical tool for the user to determine whether the models can be used or adjusted, in addition to the test of statistical significance. In some cases, models cannot be obtained due to lack of data. From a network M&R planning point of view, models for some groups may be desired. In such cases, the models can be made subjectively based on the available models similar to these cases and supplemented with engineering judgment.

Output Module

The generic output module produces various reports for the performance models, such as listings, summaries, graphs, and so forth. In addition, it can also generate various file formats, such as ASCII, dBase, Paradox, and Excel, which can be accessed by network optimization programs and project life cycle cost analysis programs.

M&R TREATMENTS

In PPMP, an M&R treatment is the combination of a level of the thickness of a surface layer and the material type of the surface layer. The level of layer thickness can be divided into thin, medium, and thick, and may differ from one material type to another. The pavement type under a surface layer can be flexible or rigid if the surface layer is an overlay, or none if the surface is a new construction or reconstruction.

There are more than 160 types of surface layers used in Pennsylvania. If the average number of levels for all these layers is two, there are more than 320 M&R treatments ($160 \times 2 = 320$). Although the program allows the user to define unlimited M&R treatments, it may not be necessary to develop models for all the treatments.

Figure 2 depicts the screen for defining an M&R treatment. First, the surface layer groups are defined, the number of levels of the layer thickness is specified, and the limiting values for each level are determined. Currently, seven layer groups are used. ID2, ID3, FB1, FJ1, and FJ4 are flexible pavement surface layers; PCC and CRC are rigid pavement surface layers. The major differences among ID2, ID3, FB1, FJ1, and FJ4 are aggregate gradation and asphalt content. The structure numbers of ID2 and ID3 are 0.44; those of FJ1 and FJ4 are 0.2; and that of FB1 is 0.2 (*6*). Next, layer codes are grouped. This is done separately for AC pavements and PCC pavements.

PERFORMANCE INDEXES

The PPMP allows users to define their own performance indexes for modeling. In developing performance models for Pennsylvania, five performance indexes provided by PennDOT are used. All indexes range from 0 to 100, with 100 being excellent condition. SDI (Surface Distress Index), SFI (Surface Friction Index), and SI (Strength Index) are linear functions of condition ratings. RI (Ride Index) is a

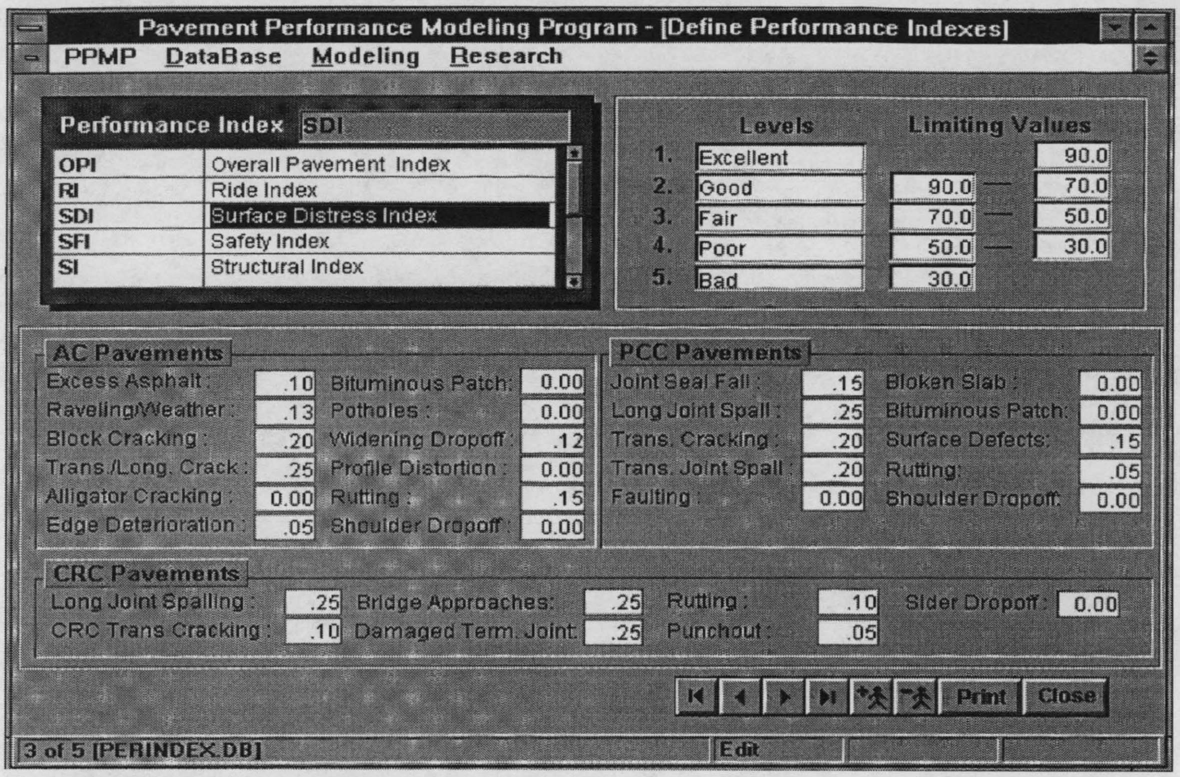

FIGURE 2 Performance definition screen.

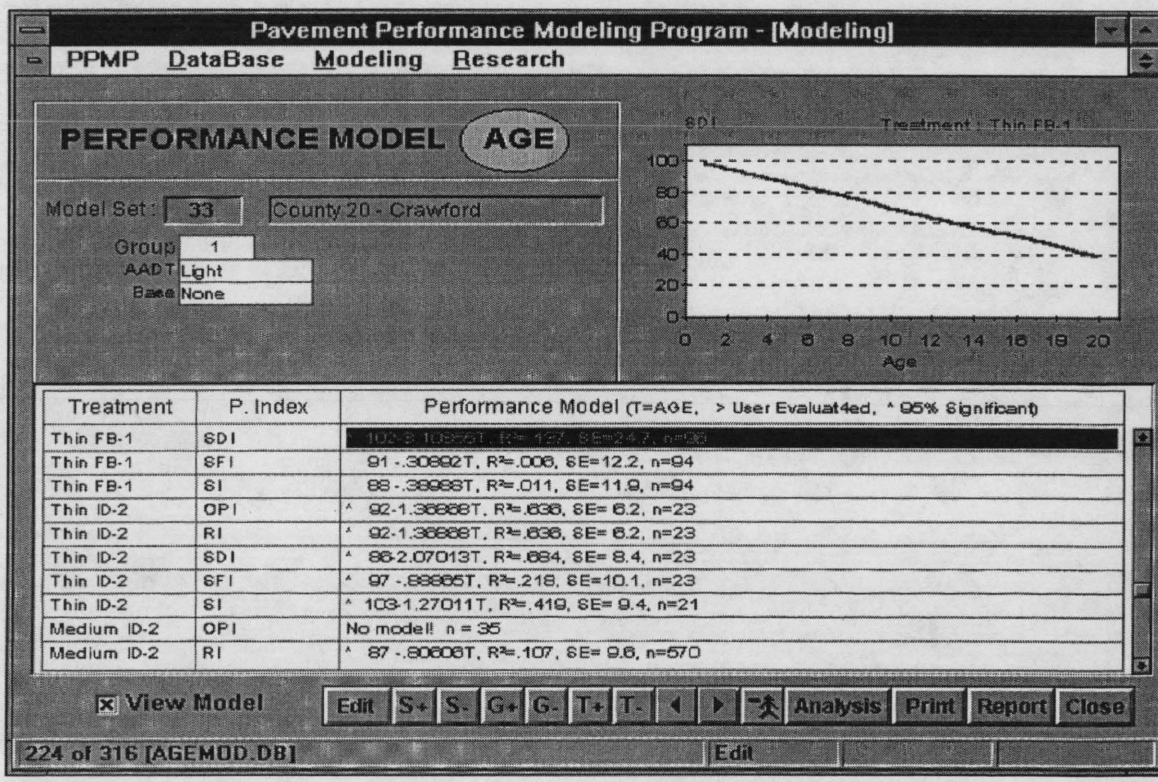

FIGURE 3 Deterministic models screen.

nonlinear function of roughness measurements; OPI (Overall Pavement Index) is a linear function of RI, SDI, SFI and SI (*7*). Figure 2 shows the pick list of the five performance indexes with that selected, thus showing its levels and definition as one of the indexes (SDI):

SDI = 0.1 (Excess Asphalt) + 0.13 (Raveling/Weathering) + 0.2 (Block Cracking) + 0.25 (Trans./Long. Cracking) + 0.05 (Edge Deterioration) + 0.12 (Widening Dropoff) + 0.15 (Rutting).

MODEL DEVELOPMENT

Figure 3 illustrates a portion of the group models developed using the default form of Equation 1. Shown on the upper part of the screen is the group definition for the current active record. The program identifies three types of models: (a) those marked with an asterisk (*) are statistically significant though the R^2 may be small; (b) those marked with nothing are not statistically significant; and (c) those marked "No model" indicate the slope parameter with a positive sign, which is unrealistic and unacceptable.

Figure 4 depicts the screen for comparing all types of models, and a model before and after outliers are removed. The degree (n) of polynomial models can be specified by the user. In cases where no model can be obtained, a user-defined model can be easily constructed.

It is important that a priori condition should be met by a performance model. If a performance model cannot meet a priori condition, it should be deleted from the model set. The a priori conditions for a performance model outlined by Lytton (*1*) can be used to evaluate the usability of a model. The most important a priori condition for the models built by this program are (a) The *initial value* of a model should be less than or equal to 100; (b) The *slope* of a model should be negative (for those with positive slopes, "No model" indications are presented as shown in Figure 3); and (c) the *final value* should be greater than or equal to zero.

ANALYSIS OF RESULTS

To test the significance of grouping variables and performance indexes, a set of runs with different combinations of grouping variables were made without removing any data points from the original data set. Significance level was set to $\alpha = 0.05$. Tables 2 and 3 list the grouping variables and the M&R treatments used in the analysis, respectively. For each run, two grouping variables—AADT and pavement type—are included because they are important for pavement performance modeling. Since there is no information about the relationship between pavement structural number and pavement age, the pavement depth of the whole structure (excluding the surface layer that is used as M&R treatment) is used as a grouping variable. It was found that, as a grouping variable, AADT is more sensitive than ESAL; and for all runs with cumulative ESAL as an independent variable, less than 20 percent of the models turned out to be significant.

Because the models of performance indexes against age are much better than those against ESAL, models against age are used for analysis. Table 4 lists the summary of the results of eight runs. In Table 4, Columns 2 through 6 list the grouping variables; Column

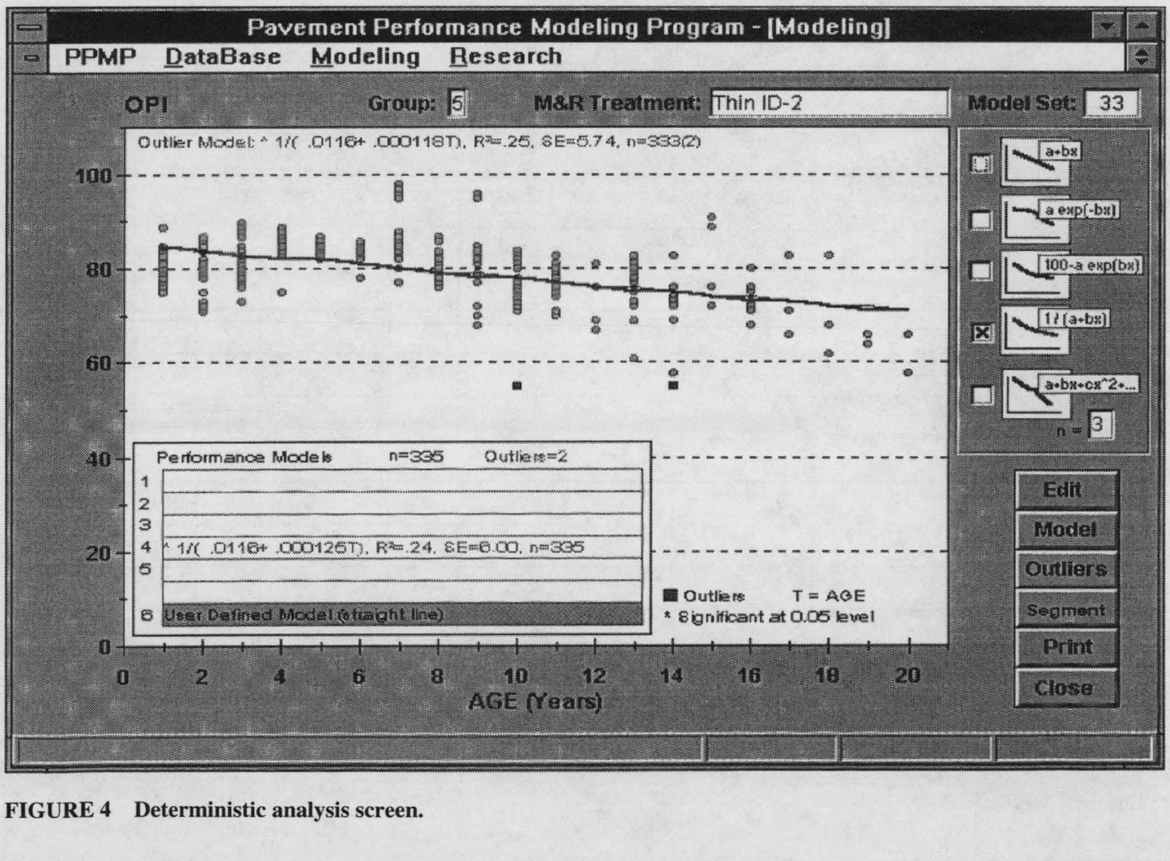

FIGURE 4 Deterministic analysis screen.

TABLE 2 Grouping Variables

Grouping Variables	Levels	Description
AADT	Light Medium Heavy	<1000 per lane 1000 - 4999 per lane ≥ 5000 per lane
Pavement Type	Flexible Rigid None	
Maintenance Level	Level 1 Level 2 Level 3	no maintenance minor maintenance such as patching major maintenance such as surface treatment
Functional Class	Rural Arterial Rural Collector Rural Local Urban Arterial Urban Collector Urban Local Ramp	
Structure Depth	Thin Medium Thick	<30 inches (76 cm) 30 - 49 inches (76 - 127 cm) ≥ 50 inches (127 cm)

TABLE 3 M&R Treatments

M&R Treatments	Levels	Description
ID2, and ID3	Thin Medium Thick	< 2 inches (5 cm) 2 - 5 inches (5 - 13 cm) ≥ 5 inches (13 cm)
FB1	Thin Thick	< 3 inches (7.6 cm) ≥ 3 inches (7.6 cm)
PCC	Thin Thick	< 8 inches (20 cm) ≥ 8 inches (20 cm)
FJ1, FJ4, CRC	One level	

TABLE 4 Number of Significant Models

No (1)	AADT (2)	Pavement Type (3)	Maintenance Level (4)	Functional Class (5)	Structure Depth (6)	No. of Models (7)	OPI (%) (8)	RI (%) (9)	SDI (%) (10)	SFI (%) (11)	SI (%) (12)	Average (%) (13)	Average $R^2 \geq 0.50$ (%) (14)
1	●	●				100	75	75	65	65	55	67	5
2	●	●	●			125	58	42	49	33	37	44	13
3	●	●		●		125	58	58	49	29	31	45	18
4	●	●			●	100	66	66	58	54	50	59	4
5	●	●	●	●		415	40	40	35	18	25	32	27
6	●	●	●		●	215	60	47	51	37	40	47	15
7	●	●		●	●	270	46	52	37	22	28	37	18
8	●	●	●	●	●	450	40	42	35	17	25	32	29

OPI = Overall Pavement Index
RI = Ride Index
SDI = Surface Distress Index
SFI = Surface Friction Index
SI = Strength Index

TABLE 5 Examples 1–4

Examples	Model Type	Original		After Outliers Removed		
		R-Square	SEE	No. Outliers	R-Square	SEE
1.	1) $\alpha+\beta x$	0.33	13.65	0		
Group #1	2) $\alpha e^{-\beta x}$	0.29	13.83	0		
SDI	3) $100-\alpha e^{\beta x}$	0.43	14.12	1	.44	11.64
Thin ID2	4) $1/(\alpha+\beta x)$	0.23	14.27	0		
N = 29	5) $\alpha+\sum \beta_i x^i$ (i=2...n)					
	n=2	0.36	13.62	1	0.48	10.89
	n=3	0.37	13.83	1	0.52	10.72
	n=4	0.38	13.92	**1**	**0.53**	**10.79**
	n=5	0.45	13.45	0		
	n=6	0.47	13.48	0		
2.	1) $\alpha+\beta x$	0.20	6.67	2	0.22	6.50
Group #2	2) $\alpha e^{-\beta x}$	0.21	6.70	2	0.23	6.53
OPI	3) $100-\alpha e^{\beta x}$	0.09	6.86	5	.10	6.46
Thin ID2	4) $1/(\alpha+\beta x)$	0.23	6.76	1	0.24	6.66
N = 471	5) $\alpha+\sum \beta_i x^i$ (i=2...n)					
	n=2	0.25	6.47	5	0.29	6.01
	n=3	0.25	6.47	5	0.29	6.01
	n=4	0.27	6.39	**5**	**0.30**	**5.98**
	n=5	0.28	6.38	5	0.30	5.98
3.	1) $\alpha+\beta x$	0.4	9.84	**4**	**0.62**	**6.19**
Group #3	2) $\alpha e^{-\beta x}$	0.34	9.77	4	0.62	6.21
SFI	3) $100-\alpha e^{\beta x}$	0.38	11.37	4	0.51	6.81
Thin ID2	4) $1/(\alpha+\beta x)$	0.27	9.80	3	0.47	7.24
N = 79	5) $\alpha+\sum \beta_i x^i$ (i=2...n)					
	n=2	0.46	9.40	3	0.54	6.86
	n=3	0.52	8.89	2	0.61	6.31
	n=4	0.52	8.94	3	0.62	6.28
	n=5	0.52	9.00	3	0.62	6.31
4.	1) $\alpha+\beta x$	0.29	6.87	2	0.34	5.83
Group #10	2) $\alpha e^{-\beta x}$	0.27	6.92	2	0.33	5.87
SDI	3) $100-\alpha e^{\beta x}$	0.32	7.06	2	0.32	5.93
Medium ID2	4) $1/(\alpha+\beta x)$	0.25	6.99	2	0.33	5.93
N = 153	5) $\alpha+\sum \beta_i x^i$ (i=2...n)					
	n=2	0.42	6.28	2	0.52	5.00
	n=3	0.44	6.16	**3**	**0.62**	**4.46**
	n=4	0.47	6.06	2	0.52	5.00

7 lists the total models obtained (number of models per index multiplied by the number of indexes); Columns 8 through 12 present the percentage of significant models for each performance index.

As indicated in Table 4, the number of significant models decreases, but the number of models with $R^2 \geq 0.5$ increases with an increase in the number of grouping variables. As far as performance indexes are concerned, OPI and RI are more significant than SDI, SFI, and SI. For groups with three grouping variables, the best combination is AADT, pavement type, and structure depth. For groups with four grouping variables, the best combination is AADT, pavement type, functional class, and structure depth. In general, structure depth is more significant than maintenance level and functional class.

To evaluate the types of models used for pavement performance modeling, M&R treatment ID2—"Thin ID2" and "Medium ID2" (Thick ID2 is unavailable) from the runs of No. 4 and No. 8 defined in Table 4—were selected for detail analysis. Tables 5 and 6 list the results of eight examples (Table 5 from No. 4 runs and Table 6 from No. 8 runs).

Tables 5 and 6 indicate that polynomial models built by the constrained least squares method perform much better than other types of models since any shape of performance curves can be generated using polynomial models. Polynomial models have been used successfully to build pavement performance models (*2,5*). It is obvious that R^2 increases and standard error of estimate (SEE) decreases with the increase of n before outliers are removed. Statistically, the

TABLE 6 Examples 5 and 6

Example	Model Type	Original		After Outliers Removed		
		R-Square	SEE	No. Outliers	R-Square	SEE
5.	1) $\alpha+\beta x$	0.54	7.15	4	0.66	5.03
Group #32	2) $\alpha e^{-\beta x}$	0.50	7.22	4	0.65	5.20
SDI	3) $100-\alpha e^{\beta x}$	0.43	9.02	5	0.41	5.63
Thin ID2	4) $1/(\alpha+\beta x)$	0.45	7.44	4	0.62	5.51
N = 190	5) $\alpha+\sum \beta_i x^i$ (i=2...n)					
	n=3	0.54	7.19	4	0.69	4.81
	n=4	0.59	6.78	4	**0.71**	**4.67**
	n=5	0.59	6.80	4	0.71	4.68
6.	1) $\alpha+\beta x$	0.48	5.75	0		
Group #32	2) $\alpha e^{-\beta x}$	0.51	5.81	0		
OPI	3) $100-\alpha e^{\beta x}$	0.19	6.26	0		
Thin ID2	4) $1/(\alpha+\beta x)$	**0.53**	**5.90**	0		
N = 204	5) $\alpha+\sum \beta_i x^i$ (i=2...n)					
	n=2	0.49	9.40	0		
	n=3	0.49	8.89	0		
	n=4	0.52	8.94	0		
	n=5	0.55	9.00	0		

larger the n, the better the model will be. It is difficult to estimate the best n for all cases. In most cases, reasonable models can be obtained with $n = 4$ after outliers are removed from the analysis, with the exception of Example 4 ($n = 3$). It can also be seen that polynomial models may not be the best models in some cases, as the results indicated in Example 3 and Example 6.

Of all the examples shown in Tables 5 and 6, R^2 increases greatly after outliers are removed from modeling, but care should be taken in removing outliers (*8*).

SUMMARY AND CONCLUSIONS

The Pavement Performance Modeling Program presented in this paper provides a powerful tool for developing pavement performance models for Pennsylvania. The program allows engineers and researchers to develop various performance models based on available data, to evaluate the data and the models, and to select the best model for use in a PMS.

The program is flexible enough to allow the user to define modeling scope, performance indexes, grouping variables, M&R treatments, and maintenance levels. Modeling scope can be a county, a district, a mix of counties and districts, or the whole state. Grouping variables include AADT or ESAL, functional class, pavement type, pavement structure depth, maintenance level, and so forth. The user can define individual performance indexes and comprehensive performance indexes. M&R treatments and maintenance level can be determined by grouping the detailed pavement surface types and the maintenance activities. For deterministic models, five types of models can be built, and outlier analysis can be performed. The program is user friendly with a graphical user interface in which the data and models can be plotted on screen and analyzed one by one.

From the preliminary analysis of the original data and the models developed using the program, it has been found that AADT is more significant than ESAL, and that the performance indexes OPI and RI are more significant than SFI, SDI, and SI. In general, polynomial models perform well in fitting the data, but they are not the best models in some cases.

ACKNOWLEDGMENTS

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REFERENCES

1. Lytton, R. L. Concepts of Pavement Performance Prediction and Modeling. Proc. 2nd North American Conference on Managing Pavements, Toronto, Ontario, Canada, 1987.
2. Johnson, K. D., and K. A. Cation. Performance Prediction Development Using Three Indexes for North Dakota Pavement Management System. In *Transportation Research Record 1344*, TRB, National Research Council, Washington, D.C., 1992.
3. Saraf, C. L., and K. Majidzadeh. Distress Prediction Models for a Network-Level Pavement Management System. In *Transportation Research Record 1344*, TRB, National Research Council, Washington, D.C., 1992.
4. Lee, Y., A. Mohseni, and M. I. Darter. Simplified Pavement Performance Models. In *Transportation Research Record 1397*, TRB, National Research Council, Washington, D.C., 1993.
5. Shahin, M. Y., et al. New Techniques for Modeling Pavement Deterioration. In *Transportation Research Record 1123*, TRB, National Research Council, Washington, D.C., 1987.
6. Highlands, K. L., G. F. Liddick, G. Cumberledge, and G. L. Hoffman. *The State-of-the Interstate Analysis and Reports Rationale and Role in PENNDOT.* Department of Transportation, Commonwealth of Pennsylvania.
7. *Roadway Management Manual.* Bureau of Bridge and Roadway Technology, Department of Transportation, Commonwealth of Pennsylvania, 1991.
8. Draper, N., and H. Smith. *Applied Regression Analysis.* John Wiley and Sons, Inc., New York, 1981.

Performance Models for Flexible Pavement Maintenance Treatments

PETER E. SEBAALY, STEPHEN LANI, AND ADAM HAND

Using actual pavement performance data, nine flexible pavement maintenance performance models were developed. The models relate the pavement's present serviceability index (PSI) to its age, materials properties, traffic loadings, and environmental conditions. To develop the performance models, data collected by the Nevada Department of Transportation (NDOT) personnel over the life of 123 projects were used. Statistically significant samples were drawn from these projects for each of the three maintenance techniques that NDOT commonly employs. The maintenance techniques include flush seals, sand seals, and chip seals. To produce statistically accurate predictions, performance models for each technique were developed separately for each of NDOT's three districts. In cases where a large number of projects were available, some projects were set aside for a model verification study. Using the data from the set-aside projects, the nine models were tested by comparing the predicted performance to the performance observed at the projects. These comparisons showed excellent correlations between the PSI values predicted by the models and those observed.

The increasing interest in pavement performance studies is a result of their representing the final link between theory and practice. As the pavement engineering profession strives for better design procedures and more enduring materials, the evaluation of the long-term pavement performance becomes a critical step for every agency. Predicting the actual performance of specific pavement sections under the combined action of traffic loading and environment factors can provide valuable data to the various departments of a highway agency.

The pavement design engineer can use such data to check the validity of the design procedure and the appropriateness of the various assumptions that are made during the design process. The materials engineer can verify whether a given type of material is appropriate for the expected level of load and anticipated environmental conditions. As a result, design and construction practices may be altered to produce longer-lasting pavements.

Pavement management engineers tend to gain the most from such studies. They are usually responsible for recommending various maintenance alternatives for specific applications. This is becoming an increasingly critical task since highway agencies at all levels (city, county, and state) are generally operating under a limited budget that requires effective prioritization to provide the highest level of public service. Pavement management engineers are also responsible for setting up a pavement management system (PMS) and managing the collected data. Long-term pavement performance studies that develop performance models will help the engineers to evaluate the effectiveness of the PMS and determine the usefulness of the collected data.

Four states (Arkansas, Iowa, Pennsylvania, and Washington) have recently completed studies to develop pavement performance curves (or equations) based on information in their existing data bases (*1*). All four of these states have chosen to use functional performance indicators. This is partially because functional performance indicators allow the states to establish and incorporate life cycle cost analysis into the models using their currently available data bases and existing PMS programs.

Arkansas used performance data to estimate a pavement's condition rating for the current year based on previous years' data. Components for pavement distress and ride are adjusted for traffic volumes. The pavement's condition rating is plotted against its age, on a yearly basis. From Arkansas's limited analysis it was concluded that even though the curves fit the data reasonably well, they would need to be revised to account for the effects of cumulative equivalent single axle loads (ESALs).

Iowa considered a more elaborate model that addressed some of the more obvious factors that could affect the performance of the overlay, such as thickness, aggregate durability, and base and subgrade characteristics. Sites were selected and divided by service levels and pavement type (rigid and composite). The model did allow the Iowa Department of Transportation to make some generalizations regarding material selection, but it also had several shortcomings. They included no allowance for maintenance and rehabilitation techniques (other than overlay), limited distribution of data points for loading and age, initial present serviceability indexes (PSIs) were all assumed to be constant, and only a few obvious variables that could affect the pavement performance were considered.

Pennsylvania generated performance curves from the roughness and traffic data for each of 22 monitored sites. The curves considered only rigid and composite pavement sections and while they do allow a reasonable prediction of PSI, the data considered were very limited.

Washington developed its curves based on the 5 years of data available in its data base. Washington considered a larger number of variables than the other three states. In all of its models, age was determined to be the most significant independent variable. Other variables such as overlay type showed generalized trends, but were not as significant.

None of the existing models considers the performance of pavement maintenance techniques. Therefore, there is a need to develop models that can be used to predict the performance of maintenance techniques used on flexible pavements as a function of traffic, environment, and pavement structural data. This research project dealt with the development of performance models for the flexible pave-

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ments maintenance techniques most commonly used by the Nevada Department of Transportation (NDOT).

SELECTION OF PROJECTS

A review of NDOT's historical maintenance records indicated that the most commonly used maintenance techniques include the following:

- Flush seal: applying an emulsion or liquid asphalt to the roadway at a prescribed rate.
- Sand seal: applying an emulsion or liquid asphalt to the roadway surface at a prescribed rate and applying a sand cover.
- Chip seal: applying a binder to a roadway at a prescribed rate, and covering the binder with rock screenings (chips). The binder is usually an emulsion with latex (LMRCRS-2 or LMCRS-2h). Emulsion without latex or a liquid asphalt may also be used for certain applications.

Following the selection of maintenance techniques, project selection guidelines were established. In establishing the project selection guidelines, one must keep in mind the overall objective of the research. As mentioned earlier, the developed models should be used to predict the future performance of the selected techniques. These models will use statistical analyses of actual PMS, environmental, structural, and materials data. Therefore, several minimum requirements must be satisfied to make the statistical analysis appropriate. The following criteria were selected as guidelines for project selection (*2*):

- A minimum of 20 replicate projects must be included.
- Each project should be at least 3 km long.
- Traffic data must be available for each selected project.
- Materials data must be available for each selected project.
- PMS performance data must be available for each selected project.

The existing NDOT district lines were used as regional boundaries (Figure 1), and projects were selected for each maintenance technique within each district. The project selection criteria were strictly followed with very few exceptions; some projects that were just under 3 km long were accepted due to the limited number of available projects. The projects selected for each treatment were as follows:

- Flush seal: 37 projects
- Sand seal: 38 projects
- Chip seal: 47 projects

DATA MANAGEMENT

Three categories of data were of interest: structural, environmental, and PMS. From each of these general categories, a list of factors that could possibly affect the performance life of the pavement system was derived. To be unbiased, the lists of factors were developed before any of the actual data sources were examined.

Structural Data

The structural data consisted of two parts: the first part was primarily the specific material and construction information used with the technique being examined; the second part consisted of structural and material information of all previous construction activities. If available, as-built plans are used to obtain the structural information. If the as-built plans are unavailable, copies of the contracts together with field notes and lab test results are used to determine the exact materials and quantities used.

Environmental Data

Nevada's diverse climatic conditions play a large role in the design, construction, and maintenance treatments throughout the state. Realizing that environmental factors can have a significant impact on pavement performance, it was decided that these factors should be included in the analyzed data sets. The available sources are the National Oceanic and Atmospheric Administration (NOAA) and the NDOT PMS system (*3*). The NOAA data are by far the most complete in terms of accuracy and amount of information, but are very limited in their coverage. NOAA data can only be obtained where there is an observation station; this left most of the road system in the state with no information.

The NDOT PMS system also contains weather data, and while they have several limitations, they cover the entire road system in Nevada. These data are limited by their not coming from actual observed field conditions; they are generated by a statistical model based on 30 years of NOAA weather data. The model divided the state into five zones and took NOAA data for all points within each zone and extrapolated them over the rest of the zone based upon elevation. The model can predict minimum and maximum average yearly temperatures, average number of wet days per year, average annual precipitation, and average number of freeze-thaw cycles per year at any location based on its elevation.

Pavement Management System Data

NDOT has had an operational PMS since 1980. While the system has undergone several changes in the last 13 years, most of the data in the system are available for all years since 1981 (*3*). The PMS contains ride data, condition data, traffic data, reduced calculated fields, calculated pavement ratings, and the weather data as described earlier. Using these data, the PSI is calculated from the AASHTO equation as follows (*4*):

$$\mathrm{PSI} = 5.03 - 1.91 \log_{10}(1 + SV) - 1.38\, RD^2 - 0.01\, (C + P)^{0.5} \quad (1)$$

The PSI ranges from zero to 5 with zero being the worst and 5 the best. A new pavement will generally not score above a 4.5, and pavements are generally not allowed to drop below 2 depending upon their system classification.

MODEL DEVELOPMENT

After the data for all the selected projects were collected and entered into the appropriate computer formats, the model development task was initiated. This task was a multifaceted operation that involved a great deal of testing as well as regression analysis. The purpose of the model development was to provide a conceptually simple method for examining the functional relationships among variables.

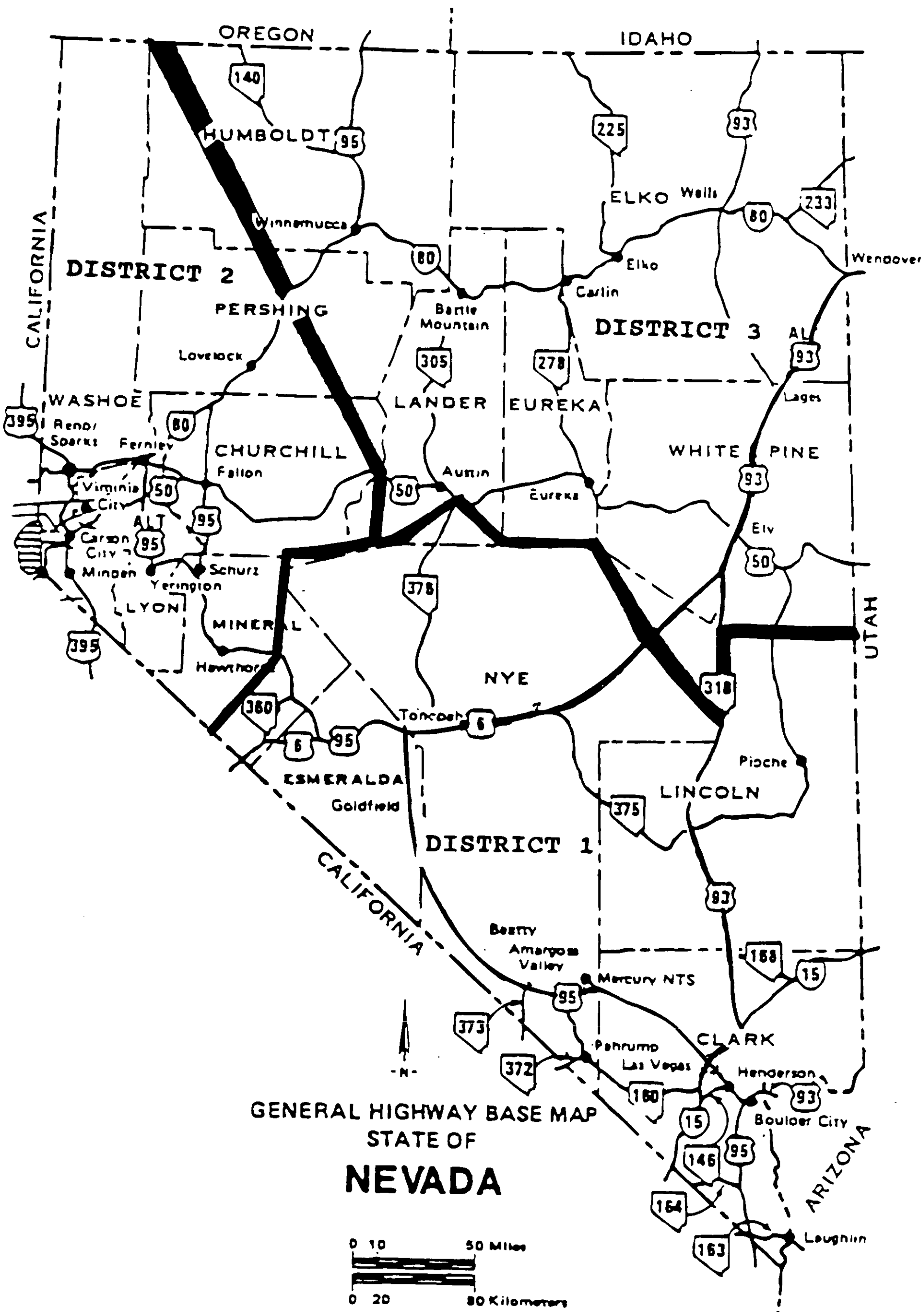

FIGURE 1 Geographical boundaries of NDOT districts.

The task was divided into the following processes: statistical analysis, data review, regression analysis, model review, tests of reasonability, model modification, additional regression analysis, model testing, and final model selection.

All of the statistical and regression analyses were performed using the SAS programming language (*5*). The SAS software is a combination of programs originally designed to perform statistical analyses of data, conduct complex data management, and provide a high-level programming language.

The box-plot statistical analysis was performed as a final check on the data prior to performing an actual regression analysis (*6*). This analysis calculates the mean and standard deviation for the PSI data for each pavement section. These values are then used to generate the acceptable range of data (e.g., plus or minus one standard deviation). Any observation that falls outside the acceptable range is considered an outlier. If data points appeared to be outliers, they were carefully examined for accuracy and reasonableness; any possible interactions among the independent variables were also carefully scrutinized to understand the possible physical representations and implications.

Regression Analysis

The General Linear Model procedure was used to develop a linear regression equation. In the first regression analysis, all the possible variables were considered. For each variable considered, a test statistic (t-value) was determined as part of the analysis. The test statistic is a representation of the significance of the individual variable in the model tested against the variable equal to zero. Variables that could be removed from the model were determined as those that had only a 5 percent chance of being significantly important to the model.

With those variables removed, a second regression analysis was performed on the remaining variables. In addition to checking the t-value of the individual variables, the Type I and Type II sums of squares of each variable were examined. The sums of squares provide an indication of any variables that may possibly be interrelated or interacting with each other. The Type I sums of squares indicate a variable's significance when considering removing the effects of the other variables, and the Type II sums of squares indicate that variable's significance after accounting for the effects of the other variables. Large differences in the t-values for variables indicate a possible interaction with other variables. If possible interactions were found, interaction terms were added to the regression analysis.

Another parameter that was of considerable importance was the sign of a variable's coefficient. In much of the previous pavement performance studies, signs were opposite of common belief or practice (*1*). For example, a positive coefficient for the 18-kip ESAL's term indicates that higher ESALs on the pavement section would generate a higher PSI. Although the models may appear to fit the data well, engineers tend to shy away from models that do not hold to traditional sign conventions. There were only a few cases in which sign conventions presented a problem in this study. In some cases, this was the result of outlier data points or misunderstood data information. The problem was corrected by simply removing the outliers. In other cases, the reversed signs were the result of a missed interaction term.

In most regression analyses, the fit of the model is described by an R-squared (R^2) value. The R^2 value is based on sample correlation coefficients that indicate the strength of the developed relationship between the response variable (PSI) and the independent variables (ESALs, AC type, aggregate rate, etc.) when compared to the observed data. R^2 may then be interpreted as the proportion of total variability in the dependent variable that can be explained by the independent variables. The R^2 can range from zero to one with the higher number indicating a better fit of the model to the actual data.

Model Testing and Selection

The tasks of model testing and model selection are interrelated. While R^2 indicates the model's fit to the analyzed data, it was more important to know how well the model can fit data not included in the analyzed data set. For the model to be accurate, it must be used within the range of parameters that were used during the development step. In other words, a model is valid only within the range of values from which it was developed. Every effort was made to maintain a data set that was representative of the entire range of variables that could be encountered on a particular project.

Verification projects were chosen at random from the original candidate list, and the data were examined to ensure that they met the required criteria for the model or models being considered. The independent variables were input into the developed regression models, and the PSIs predicted by the models were plotted against the actual recorded PSIs. Figures 2 through 7 show examples of the

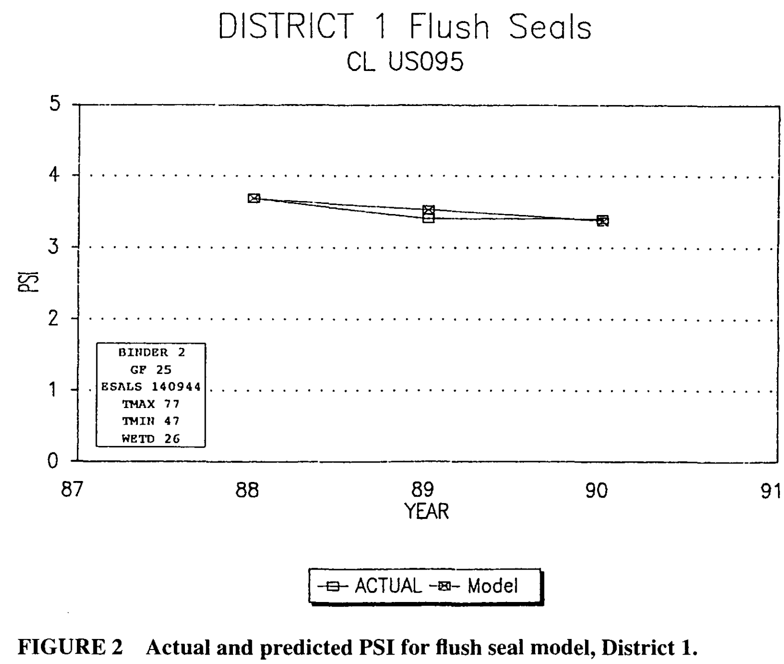

FIGURE 2 Actual and predicted PSI for flush seal model, District 1.

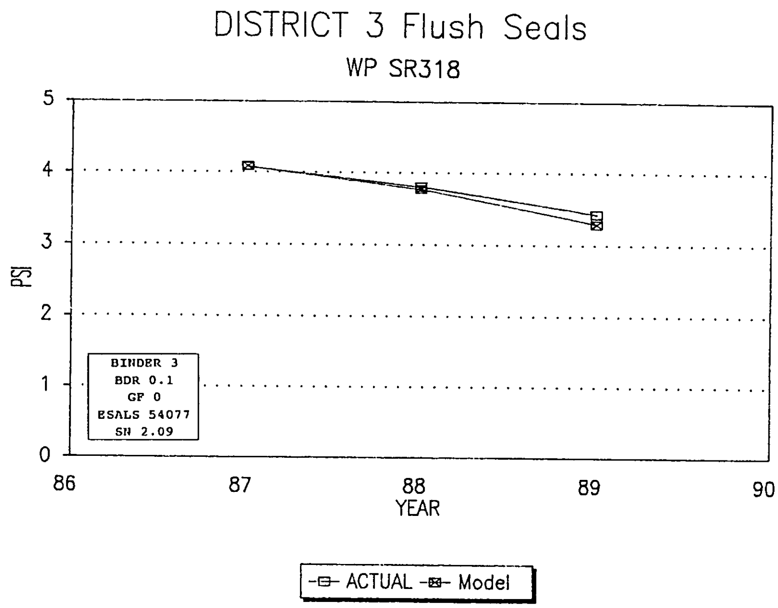

FIGURE 3 Actual and predicted PSI for flush seal model, District 3.

model verification data. In some cases, there is an excellent agreement between the actual data and the models (see Figures 2, 3, and 5). In other cases, the models showed more stable data trends than the actual data. In the case of chip seal in District 3 (Figures 6 and 7), the actual data indicate that the PSI of the pavement increases with time while the model showed a steady decrease in the PSI. There is no logical explanation of why the PSI should increase on these projects except that the collected data for that year may not be representative of the entire section.

The worst agreement between the actual data and the models was obtained in the case of sand seal in District 2 (Figure 4). This model has a relatively low R^2 value, which indicates that the model does not fit the data very well. The R^2 for this model is 0.6, which means that 40 percent of the variability in the data cannot be explained by

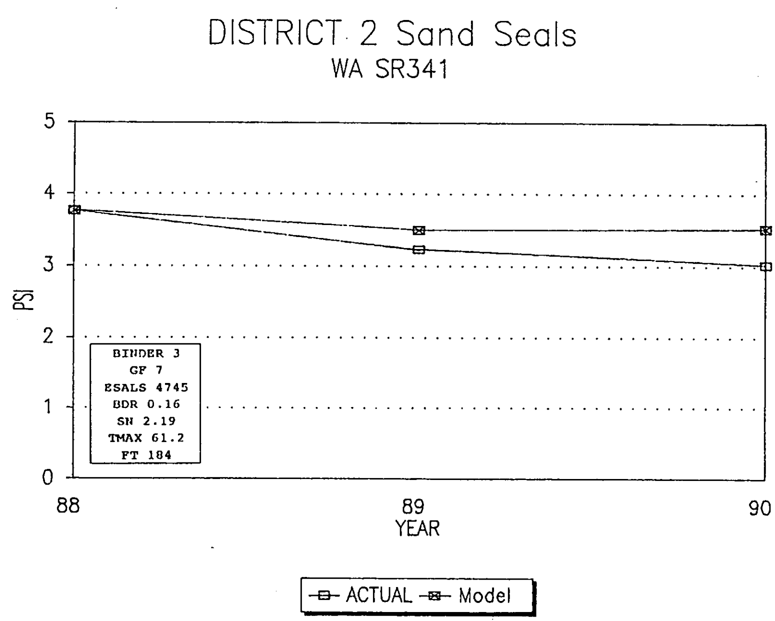

FIGURE 4 Actual and predicted PSI for sand seal model, District 2.

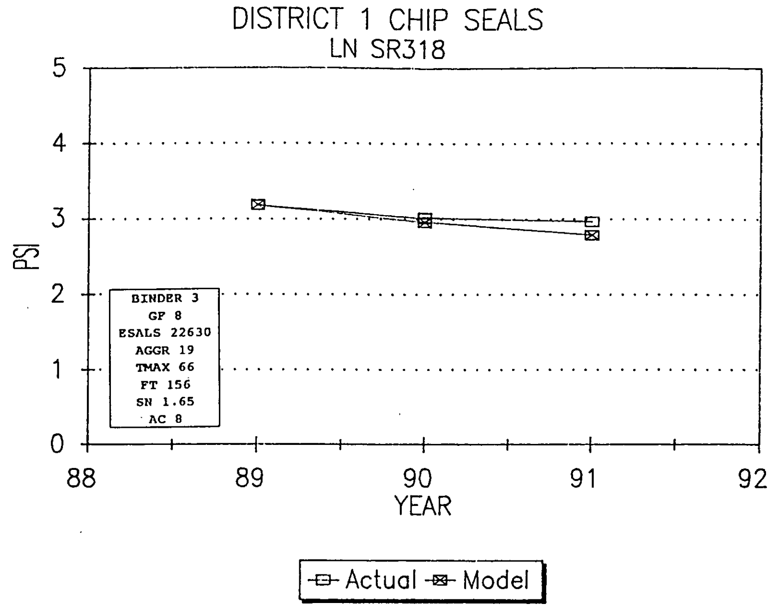

FIGURE 5 Actual and predicted PSI for chip seal model, District 1.

the model. The flush seal model in District 1 has an R^2 of 0.58, which is also low compared to the other models. However, in the case of the flush seal District 1 model, the agreement between the model and the actual data was excellent (Figure 2). This indicates that when the model has a low R^2 value (below 0.8), its performance becomes unpredictable. In other words, a model with an R^2 value below 0.8 may give excellent prediction for one project while showing poor prediction for another one. Based on this criterion, the flush seal (District 1) and the sand seal (District 2) models presented in this paper should be used with extreme caution. Tables 1 through 9 summarize the verified models for all techniques and all NDOT districts.

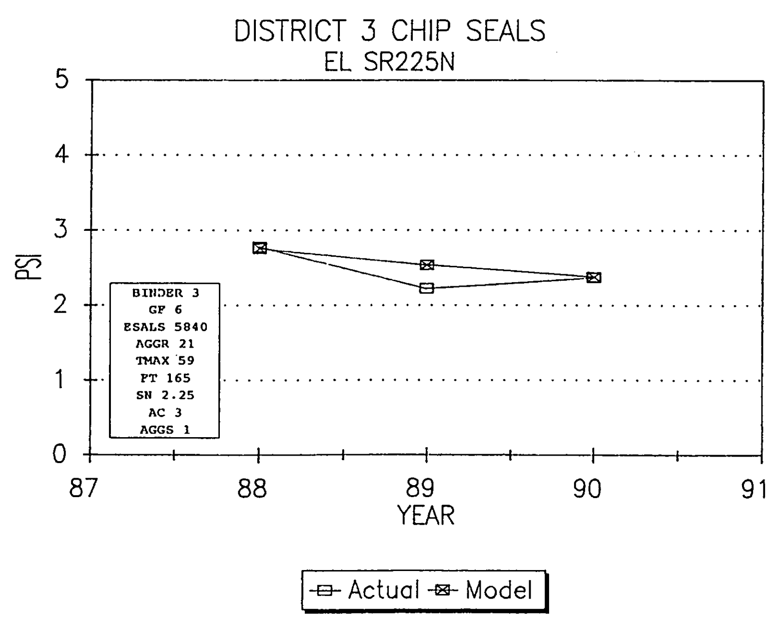

FIGURE 6 Actual and predicted PSI for chip seal (EL SR 225N) model, District 3.

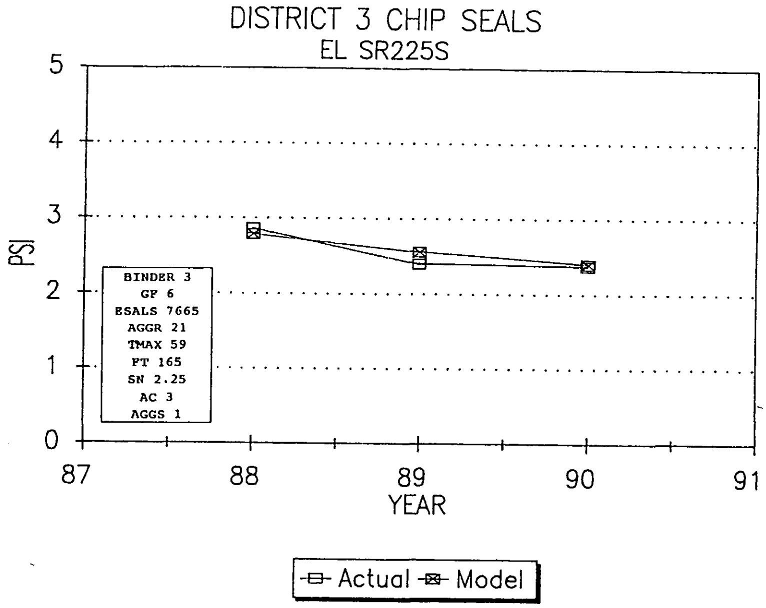

FIGURE 7 Actual and predicted PSI for chip seal (EL SR 225S) model, District 3.

Definition of Variables

The following represents a list of the variables included in the models shown in Tables 4 through 9.

AC: Type of binder used in the first structural layer below the flush, sand, and chip seals.

AGGR: Aggregate spread rate for chip and sand seal projects, lbs/yd^2 ($0.54\ kg/m^2$).

AGGS: Maximum nominal aggregate size used in chip seal projects.

BDR: Binder application rate for flush, sand, and chip seal projects, gal/yd^2 ($4.6\ L/m^2$).

Binder: Type of binder used in maintenance project.

CFT: Cumulative value of freeze-thaw cycles. It is obtained as cycles per year multiplied by year of project, (see FT for freeze-thaw cycle information).

ESALS: Cumulative value of 80-kN equivalent single axle loads calculated by multiplying the daily 80-kN ESALS by 365 and a

TABLE 1 Verified Flush Seal District 1 Model

Flush Seal District 1 Model	Number of Observations	R-Squared
$PSI = 36.03 + C1 + 2.8e\text{-}7ESALS - 0.18YEAR - 0.42TMAX + 0.14TMIN - 0.25WETD - 1.89e\text{-}13ESALS^2$	540	0.58
Variables	**Range**	
ESALS	365 - 2614313	
YEAR	1 - 3	
TMAX	58 - 81	
TMIN	27 - 50	
WETD	22 - 48	
Binder Type	**Constant (C1)**	
SS-1H	0.00	
MC-70	0.57856986	

TABLE 2 Verified Flush Seal District 2 Model

Flush Seal District 2 Model	Number of Observations	R-Squared
PSI = 3.27 + C1 + 2.86e-6ESALS - 0.56SN - 0.13YEAR	72	0.91
Variables	**Range**	
ESALS	365 - 1018350	
YEAR	1 - 3	
SN	1.00 - 3.48	
Binder	**Constant (C1)**	
SS-1H	0.00	
CRS-1	1.047347855	

TABLE 3 Verified Flush Seal District 3 Model

Flush Seal District 3 Model	Number of Observations	R-Squared
$PSI = 11.96 + C1 + C2*BDR - 7.00e\text{-}7ESALS - 5.62SN - 0.34YEAR - 1.79e\text{-}13ESALS^2 + 0.92SN^2$	288	0.88
Variables	**Range**	
ESALS	10950 - 3759135	
YEAR	1 - 3	
SN	1.30 - 4.18	
BDR	0.05 - 0.18	
Binder Type	**Constant (C1)**	
CRS-1	-0.0488592	
MC-250	24.7892223	
SS-1H	0.00	
Binder Type	**Constant (C2)**	
CRS-1	0.00	
MC-250	-228.2079830	
SS-1H	-1.7459573	

TABLE 4 Verified Sand Seal District 1 Model

Sand Seal District 1 Model	Number of Observations	R-Squared
PSI = -6.43 + C1 + 30.52BDR - 1.32e-6ESALS - 0.13YEAR	256	0.86
Variables	**Range**	
ESALS	9125 - 324120	
YEAR	1 - 5	
BDR	0.08 - 0.34	
Binder Type	**Constant (C1)**	
LMCRS	-0.50753824	
SS-1H	3.73656624	
CRS-1	6.51841777	
CRS-2H	-0.24556977	
ARA-A	7.83622503	
ARA-B	0.00	

TABLE 5 Verified Sand Seal District 2 Model

Sand Seal District 2 Model	Number of Observations	R-Squared
PSI = 6.23 + C1 - 0.51BDR + 3.95e-6ESALS + 0.24SN - 0.045TMAX - 9.8e-4CFT - 0.50YEAR + 3.50e- 10ESALS^2+ 0.12YEAR^2 - 8.93e-6ESALS*SN	314	0.60
Variables	**Range**	
ESALS	365 - 97455	
YEAR	1 - 5	
TMAX	46 - 69	
SN	1.10 - 2.99	
CFT	95 - 915	
BDR	0.11 - 0.25	
Binder Type	**Constant (C1)**	
MC-250	-0.232861533	
SS-1H	0.191551524	
CRS-1	0.00	

TABLE 6 Verified Sand Seal District 3 Model

Sand Seal District 3 Model	Number of Observations	R-Squared
PSI = 11.34 + C1 + 7.25BDR + 5.25e-6ESALS + 0.38SN - 0.15TMAX - 4.3e-3CFT + 0.26YEAR - 3.91e-13ESALS^2 + 0.069YEARS^2 - 2.47e-6ESALS*SN	224	0.80

Variables	Range
ESALS	365 - 1769885
YEAR	1 - 4
TMAX	59 - 67
SN	1.40 - 4.03
CFT	154 - 756
BDR	0.10 - 0.36

Binder Type	Constant (C1)
SS-1H	-1.55768840
LMCRS-2H	-1.41913898
MC-70	1.10471980
MC-800	-0.34928839
MC-250	0.00

growth factor and adding to the previous year, beginning with year zero of a project.

FT: Total number of freeze-thaw cycles that a pavement may experience over the course of one year.

TMAX: Maximum average yearly temperature that a pavement section may experience.

TMIN: Minimum average yearly temperature that a pavement section may experience.

SN: Structural number prior to application of a maintenance technique.

WETD: Total number of wet days.

YEAR: Service year of the project. The year of construction is represented by year zero.

SUMMARY AND CONCLUSIONS

Nine performance models were developed to cover all maintenance techniques for all three NDOT districts. The three techniques for which models were developed are flush seals, sand seals, and chip seals.

The majority of the models have R^2 values above 70 percent, indicating a very good fit between the models and the data. The verification study showed an excellent correlation between the measured PSI values and computed values for test sites that were not in the original data base.

Based on the analysis of the data and the verification study, the following recommendations can be made.

- The flush seal model for District 2 has only 12 reduced observations, which were obtained from four projects. Therefore, even though the R^2 for this model is very high (0.91), the model should be used with extreme caution because of the model's extremely limited data base.
- The models should not be used for situations that are outside the boundaries of the original data base. If a certain combination is desired that is outside the boundaries of the data base, an effort should be made to approximate the desired data with the closest variables that exist within the data base. For example, if a binder type is recommended for a flush seal project, and that binder is unavailable in the model's data base, then a binder that most closely resembles the desired binder, in performance characteristics, should be chosen.
- The developed models should undergo extensive implementation efforts and be updated annually during the first 3 to 5 years to accommodate the rapidly changing trends in material specifications and pavement performance monitoring.

TABLE 7 Verified Chip Seal District 1 Model

Chip Seal District 1 Model	Number of Observations	R-Squared
PSI = 1.20 + C1 + C2 + C3 - 2.89e-7ESALS + 0.027AGGR - 0.013TMAX - 8.6e-3FT + 0.78SN - 0.28YEAR + 0.023YEAR^2	284	0.84
Variables	**Range**	
ESALS	1095 - 523410	
AGGR	19 - 33	
TMAX	66 - 80	
FT	53 - 156	
SN	1.35 - 3.76	
YEAR	1 - 6	
Binder Type	**Constant (C1)**	
MC-800	1.021811264	
CRS-2/CRS-2H	0.135232398	
LMCRS-2	0.00	
AC	**Constant (C2)**	
60-70	1.660032310	
85-100	0.829006560	
SC-4	0.899722220	
SC-800	0.325306063	
MC-800	1.207444910	
AR-4000	0.00	
Binder - AC Combination	**Constant (C3)**	
MC-800 & 60-70	-0.705516471	
MC-800 & SC-800	0.284393056	
All other combinations	0.00	

TABLE 8 Verified Chip Seal District 2 Model

Chip Seal District 2 Model	Number of Observations	R-Squared
$PSI = -2.86 + C1 + C2 + C3 + C4 - 1.02e\text{-}4ESALS - 0.015AGGR + 0.075TMAX - 2.98e\text{-}3FT + 0.125SN - 0.33YEAR + 0.005YEAR^2$	234	0.87
Variables	**Range**	
ESALS	365 - 1647245	
AGGR	20 - 38	
TMAX	58 - 73	
FT	100 - 183	
SN	1.68 - 6.17	
YEAR	1 - 4	
Binder Type	**Constant (C1)**	
CRS-2/CRS-2H	1.281414527	
LMCRS-2	1.475765738	
AR-2000	0.00	
AC	**Constant (C2)**	
85-100	1.166532005	
120-150	-0.098528394	
SC-800	0.869102804	
AR-2000	0.143673193	
AR-4000	0.00	
AGGS	**Constant (C3)**	
3/8"	0.579529646	
1/2"	0.00	
Binder - AC Combination	**Constant (C4)**	
CRS-2/CRS-2H & 120-150	0.554234128	
CRS-2/CRS-2H & AR-4000	0.283288225	
All other combinations	0.00	

TABLE 9 Verified Chip Seal District 3 Model

Chip Seal District 3 Model	Number of Observations	R-Squared
PSI = -24.04 + C1 + C2 + C3 + C4 + 4.90e-7ESALS - 0.38AGGR + 0.83TMAX - 0.042FT - 1.32SN - 0.60YEAR + 0.056YEAR^2	150	0.92
Variables	**Range**	
ESALS	2190 - 1188805	
AGGR	20 - 30	
TMAX	57 - 67	
FT	145 - 216	
SN	1.65 - 5.41	
YEAR	1 - 4	
Binder Type	**Constant (C1)**	
CRS-2/CRS-2H	1.02686865	
LMCRS-2	0.27622556	
MC-3000L	0.00	
AC	**Constant (C2)**	
85-100	-0.13911552	
120-150	-6.00223816	
SC-800	-5.11758889	
MC-800	-8.10408550	
AR-4000	-4.16186176	
AR-1000	0.00	
AGGS	**Constant (C3)**	
3/8"	3.62555754	
1/2"	0.00	
Binder - AC Combination	**Constant (C4)**	
CRS-2/CRS-2H & 120-150	0.13670219	
CRS-2/CRS-2H & MC-800	-1.68689386	
All other combinations	0.00	

REFERENCES

1. Bendar, J. J. Pavement Performance Curves: Four Case Studies. *Public Roads*, Vol. 53, No. 3, Dec. 1989, pp. 90–99.
2. Lani, S., P. E. Sebaaly, and G. Fernandez. *Development of Performance Curves for Pavement Rehabilitation and Maintenance Treatments.* Report No. 1092-1. Nevada Department of Transportation, Carson City, 1993.
3. *Pavement Management System: System Description and Application.* Nevada Department of Transportation, Carson City, 1980.
4. *AASHTO Guide for Design of Pavement Structures.* American Association of State Highway and Transportation Officials, Washington, D.C., 1986.
5. *SAS User's Guide: Statistics Version*, 5th ed., SAS Institute, Inc., N.C., 1985.
6. Montgomery, D. C. *Design and Analysis of Experiments*, 2nd ed. John Wiley and Sons, New York, 1984.

Sensitivity of Pavement Network Optimization System to Its Prediction Models

KELVIN C.P. WANG AND JOHN ZANIEWSKI

The prediction models in the network optimization system (NOS) are exhibited in the form of transition probability matrices (TPMs) in the newly implemented NOS (AZNOS) in the Arizona Department of Transportation. Due to variability in pavement performance parameters over time, it is necessary to study the effect of the influencing factors causing this variability. One such factor is annual expenditure on pavement rehabilitation, which is determined with the help of AZNOS results. In addition, rehabilitation budgets recommended by AZNOS are determined by the existing pavement network conditions, performance standards, and, more importantly, the prediction models through the use of the linear optimization routine. Even though it is evident that variations of transition probabilities from and to particular condition states will affect the recommended rehabilitation budgets from AZNOS, there is a lack of quantitative analysis in this relationship. AZNOS performance models' sensitivity to variations in transition probabilities and current pavement conditions is analyzed. This sensitivity study demonstrates the inherent relationship among prediction models (TPMs), rehabilitation needs, and current pavement conditions. This analysis also reveals an important property of AZNOS that large future savings in the pavement rehabilitation program may be obtained through the applications of effective preventive maintenance actions to existing pavements.

The major update of the Arizona Department of Transportation (ADOT) network optimization system (AZNOS) resulted in improved model structure and performance prediction (*1–4*). In addition, a model of pavement probabilistic behavior was developed in the process of implementing the new system (*2*). The pavement prediction models in AZNOS are exhibited in the form of transition probability matrices (TPMs), which determine the probabilities of pavements to progress from any condition state to all condition states in 1 year. Two major parameters—ride quality (*roughness*) and surface distress (*cracking*)—coupled with the third parameter *index to first crack*, determine the structure of the pavement condition states. The roughness and cracking parameters are also the barometers for pavement performance in NOS. Figure 1 illustrates in (a) and (b) the history of roughness levels and cracking levels for high-traffic interstate highways in the Arizona desert. The variations of the network's performance depicted in the figure in relation to roughness and cracking are due to a number of factors, one of which could be the actual budget allocated for the yearly rehabilitation. The transition probabilities used in the model are estimates based on past pavement performances (*2*). The transition probabilities directly affect the behavior of the prediction models in the optimization process and ultimately influence the results of the financial recommendations of AZNOS. As ADOT has more than 10-years' experience in using NOS in its pavement rehabilitation program, and rehabilitation expenditure is determined with the help of NOS results, it is reasonable to believe that the transition probabilities in the prediction models need further analysis. This paper presents the analysis of sensitivities of AZNOS to the variations in the transition probabilities, or TPMs, and actual pavement conditions.

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THEORETICAL BACKGROUND

The Markov process is a time-independent stochastic description of event development. Pavement behavior is modeled with the Markov process in 1980 in ADOT's pavement management system (*5*). The Markov property is equivalent to stating that the conditional probability of any future event, given any past event and the present state, is independent of the past event and depends only on the present state of the process. The conditional probability for the process to transition from one state to another is called transition probability. The transitions are also called steps. Therefore, the n-step transition probability $p_{ij}^{(n)}$ is defined as the conditional probability that the random variable X, starting in state i, will be in state j after exactly n steps, or time units.

A convenient notation for representing the transition probabilities is the matrix form

$$P^{(n)} = \begin{bmatrix} P_{00}^{(n)} & \cdots & P_{0M}^{(n)} \\ \vdots & & \vdots \\ P_{M0}^{(n)} & \cdots & P_{MM}^{(n)} \end{bmatrix} \tag{1}$$

$P^{(n)}$ is the n-step TPM. As applied in ADOT's NOS, the transition process of the pavement condition states conforms to the finite-state Markov chain process. Future pavement condition is dependent only on the current pavement condition. The performance model used in the NOS is based on transition probability matrices. A transition probability, $p_{ij}(a_k)$, is assumed to be equivalent to the proportion of roads in state i that move to state j in 1 year if the kth rehabilitation action is applied. It defines the probability of transition from one condition state to another in 1 year under one of the rehabilitation actions, including routine maintenance.

Chapman-Kolmogorov equations provide a method for computing the n-step transition probability matrix from a single-step transition probability matrix as used in NOS:

$$P^{(n)} = P \cdot P \ldots P = P^{n} \tag{2}$$

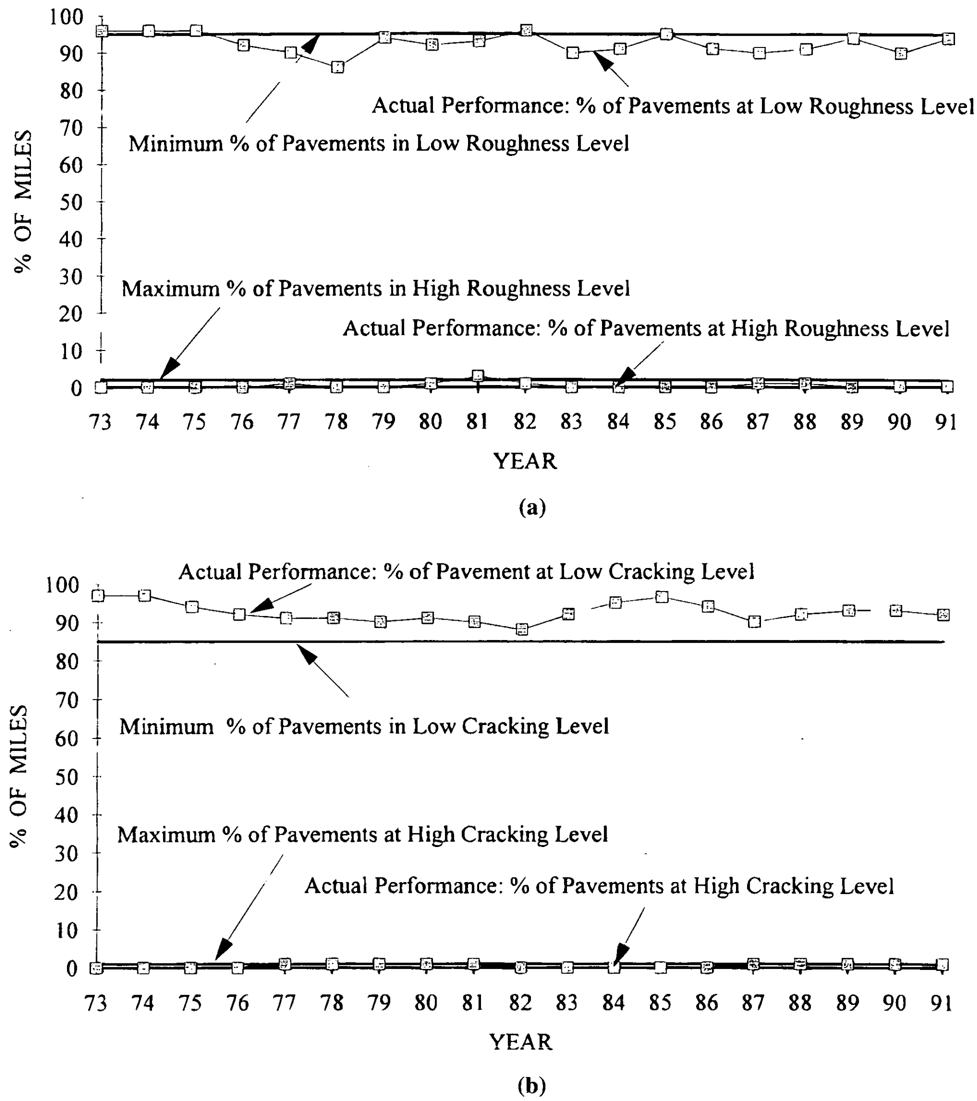


FIGURE 1 Pavement history of roughness (a) and cracking (b), high-traffic road category of Arizona Interstate network, AC pavement (2663 km) with ADT >10,000.

Therefore, the transition probabilities of pavement condition for a period of n years can be obtained based on the existing one-step transition probabilities of pavement condition. This allows a probabilistic prediction of pavement behavior over the life of the pavement structure. As shown in Equation 2, the transition probabilities for n number of periods or years can be calculated by multiplying the one-step or the original TPM n times. The following pavement probabilistic behavior equation for one rehabilitation action in vector form is established based on Equation 2 (*2*):

$$P^{(n)} = \begin{cases} P^{(n)}_{\text{routine}} & n \le v \\ P^{(v)}_{\text{routine}} \cdot P^{(1)}_{\text{rehab}} \cdot P^{(n-v-1)}_{\text{after rehab}} & n > v \end{cases} \qquad (3)$$

where

$P^{(n)}$ = n-step TPM;

$P^{(n)}_{\text{routine}}$ = n-step TPM before the rehabilitation when $n \le v$;

$P^{(v)}_{\text{routine}}$ = v-step TPM when the rehabilitation is applied;

$P^{(1)}_{\text{rehab}}$ = the one-step TPM based on the effectiveness of the rehabilitation at the period of v immediately after the application; and

$P^{(n\text{-}v\text{-}1)}_{\text{after rehab}}$ = (n-v-1)-step TPM after the rehabilitation.

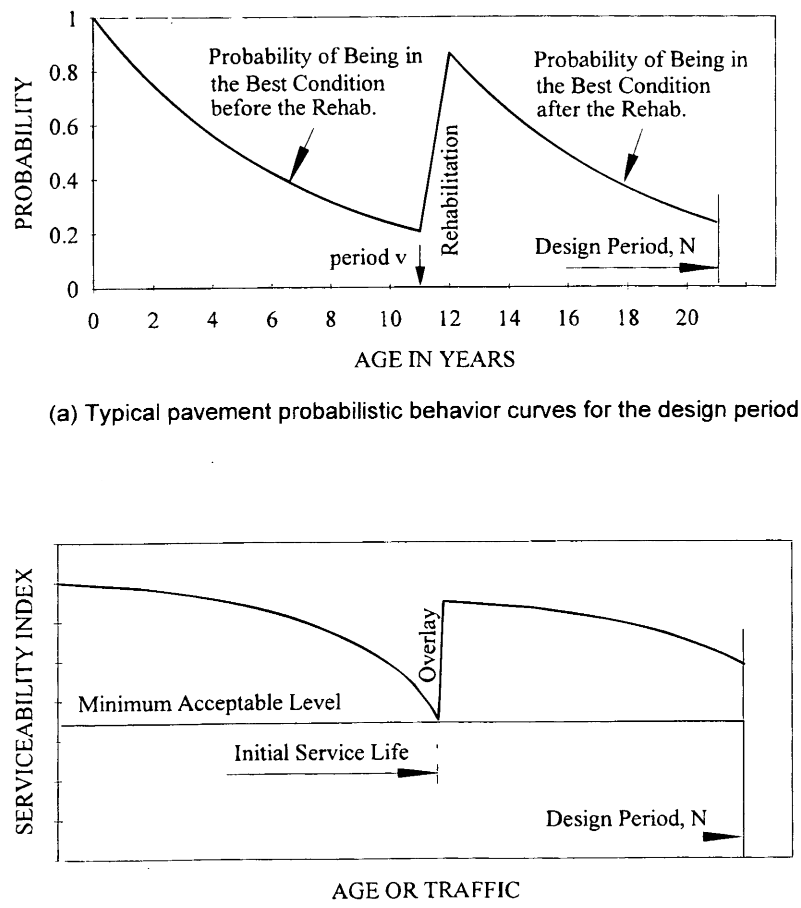
As indicated in the Equation 3, three TPMs are needed to conduct the analysis of long-term probabilistic behavior for the entire design period during which one rehabilitation is applied. The data generated based on Equation 3 can be used to plot pavement probabilistic behavior curves (PBCs). Pavement PBC is defined as the probability of being in a given condition state over time. Therefore, each condition state can have its own set of PBCs. An important performance standard set by ADOT is the minimum percentages of roads in the best condition state with the lowest roughness and cracking levels. Figure 2 in (a) illustrates typical long-term PBCs for the best condition state of design period *N* for interstate pavement for this condition state. The vertical axis represents the probability of pavements remaining in the best condition state. Figure 2 in (b) presents a traditional pavement performance curve. Note the sag shape of the probabilistic behavior curve in (a) versus the crest shape of the performance curve in (b).

DESIGN OF SENSITIVITY STUDY

It is important to perform sensitivity analysis to investigate the effect on the optimal solution provided by the simplex method if the parameters take on other possible values. Usually, there will be some parameters that can be assigned any reasonable values without affecting the optimality of the solution. However, there may also be parameters with likely alternative values that would yield a new optimal solution. In the case of AZNOS, the optimal solution is expressed in the form of budget needs of pavement rehabilitation for each year in the planning horizon. It is certain that variations in the independent variables, such as transition probabilities, will affect the optimal solutions. The transition probabilities used in the AZNOS models are *estimates* used to predict future conditions. The approach used to develop these estimates are based on past pavement performances (*2*). Therefore, the basic objective of this analysis is to quantitatively identify the sensitivities of budget needs from AZNOS to variations in the independent variables, such as the transition probabilities, so that care can then be taken in the estimation of the parameters. In addition, this analysis may also provide quantitative data on the effective implementation of rehabilitation actions to existing pavement networks to improve future rehabilitation programs and reduce costs.

Transition Probabilities in NOS

Two submodels are used in the original NOS: steady state and multiperiod. It has been demonstrated that the multiperiod model is more practical for the management of statewide pavement networks (*3*). The following formulations indicate the main mathematical structure of the multistage AZNOS relating to the interested parameters.



(a) Typical pavement probabilistic behavior curves for the design period.

(b) Illustration of pavement performance and prediction.

FIGURE 2 Pavement probabilistic behavior curve and pavement performance curve.

The objective is to minimize

$$\sum_{l=1}^{T-1}\sum_{i,k} w_{i,k}^{l} \cdot d_l \cdot c(i,k) \tag{4}$$

subject to

$$\sum_{k} w_{j,k}^{l} = \sum_{i,k} w_{i,k}^{l-1} \cdot Pij(a_k), \text{ for } 1 < l \le T \tag{5}$$

$$\sum_{k} w_{j,k}^{l} = q_i \tag{6}$$

$$\sum_{j,k} w_{j,k}^{l} \le P_1(l) \cdot \gamma_i, \text{ for } i \in I, j \in j_1(i), 2 \le l \le T \tag{7}$$

$$\sum_{j,k} w_{j,k}^{l} \ge P_2(l) \cdot \varepsilon_i, \text{ for } i \in J, j \in j_2(i), 2 \le l \le T \tag{8}$$

where

- $w_{j,k}^{l}$ = the proportion of roads of a given road category that are in condition state *j* at the beginning of the *l*th time period of horizon *T*, and to which the *k* preservation action is applied;
- $p_{ij}(a_k)$ = pavement transition probability from condition state *i* to *j* due to the rehabilitation action *k*;
- $c(i,k)$ = cost matrix for pavements in condition *i* receiving action *k*;
- d = present worth of one dollar spent during *l*th time period;
- q_i = current proportion of roads in *i*th condition state;
- $p_1(l)$ = a multiplier ≥ 1 to permit a higher than γ_i proportion of roads in undesirable states at the *l*th time period;
- $p_2(l)$ = a multiplier ≤ 1 to permit a higher than ϵ_i proportion of roads in undesirable states at the *l*th time period; and
- γ_i and ϵ_i = performance standards set by ADOT management.

Equation 5 forms the core of pavement performance prediction in NOS. It presents the very basic relationship between transition probabilities and condition prediction in the classical formulation of linear programming in a Markov chain. This equation has also been proved to be compatible with Equation 3 used to define pavement probabilistic behavior curves (*2*).

It is clear that when current conditions of the pavement network q_i and performance standards γ_i and ϵ_i are known, transition probabilities $p_{ij}(a_k)$ determine the condition transitions of the network shown in Equation 5. Ultimately, rehabilitation needs ($w_{j,k}^{l}$) are resolved through the use of linear programming based on values of given parameters, including $p_{ij}(a_k)$.

Data Selection

Sensitivity analysis is a statistical study to determine the sensitivity of dependent variables, such as $w_{j,k}^{l}$ and $\sum_{l=1}^{T-1}\sum_{i,k} w_{i,k}^{l} \cdot d_l \cdot c(i,k)$, to variations in independent variables, such as the transition probabilities $p_{ij}(a_k)$, q_i, *and* γ_i and ε_i over reasonable ranges. This analysis involves investigating the effect on the optimal solution by making changes in the values of these model parameters.

The prediction models' sensitivities to performance standards γ_i and ϵ_i were carefully analyzed by Wang et al. (*4*). In this analysis, performance standards were increased incrementally in the form of maximum percentages of roads at high roughness and cracking levels and minimum percentages of roads at low roughness and cracking levels. The corresponding rehabilitation needs in the form of an AZNOS budget recommendation were also increased along the higher standards. Based on the data presented to ADOT management on the analysis of statewide pavement rehabilitation needs (*4*), ADOT set the performance standards for Arizona pavement networks. Therefore, it has been determined that pavement performance standards are used as given data. Since the focus of the study is on the prediction models or the transition probabilities, cost matrix $c(i,k)$ is also used as given data in this analysis.

As a result, current pavement conditions q_i and transition probabilities $p_{ij}(a_k)$ are the only independent sets of parameters in the AZNOS model that need further analysis in this paper. As shown in Equation 5, variations in the transition probabilities play the determining role in the transition of pavement condition states. Therefore, the sensitivity analysis will concentrate on the roles of prediction models or TPMs in determining AZNOS budget recommendations. As performance predictions are made from existing pavements, the current pavement conditions directly affect the result of optimization. As such, current pavement conditions q_i are also used as independent parameters for this analysis.

There are 15 road categories in Arizona. Each road category was determined based on its traffic level, geographical region, and rainfall. Each road category can be perceived as a highway subnetwork. There are performance prediction models for each subnetwork. The road category (subnetwork) of high traffic, desert interstate highways is chosen for this sensitivity analysis since it has the largest pavement area among the 15 road categories and carries the traffic load for the Phoenix metropolitan area and adjacent regions. Therefore, the rehabilitation needs for this network are very large compared with other networks. Six rehabilitation actions are shown below with corresponding costs for this interstate subnetwork:

Rehabilitation Action	*Cost($)*
ROUTINE	0.12
SEAL COAT	1.38
ACFC;ACSC	2.30
ACFC + AR;ARAC;2″AC + FC	6.90
2″AC + FC + AR;3″AC + FC(W/O AR)	10.35
4″AC + FC;4″/5″AC + FC	13.80

ACFC and ACSC stand for asphalt concrete friction course and asphalt concrete surface course, respectively. AR is asphalt rubber. ARAC is asphalt rubber plus asphalt concrete. The preset pavement performance standards for this interstate network are 95 percent for minimum percentage of roads in the low roughness level, 2 percent for maximum percentage of roads in high roughness level, 85 percent for minimum percentage of roads in the low cracking level, and 1 percent for maximum percentage of roads in high cracking level.

Data Requirements and Analysis

The independent variables in the prediction equations must be statistically linear and contain a minimum collinearity between independent variables, for the following reasons (*6*):

- The magnitudes of the effects from varying the individual nonlinear independent variables would not be directly comparable.
- As collinearity must be minimized for any meaningful analysis, and nonlinear regression techniques are deficient to identify

collinearity, the use of nonlinear analysis could seriously limit confidence in the results.

- There are no existing procedures for conducting sensitivity analyses on nonlinear models.

It is clear that the relationships among parameters in AZNOS are all linear. In addition, current conditions q_i and transition probabilities $p_{ij}(a_k)$ are independent of each other. However, there exist properties for both q_i and $p_{ij}(a_k)$ that may pose difficulties in meeting the minimum collinearity requirement:

$$\sum_i q_i = 1 \tag{9}$$

$$\sum_j P_{ij}(a_k) = 1 \tag{10}$$

Apparently, parameters q_i in $\sum_i q_i$ or $p_{ij}(a_k)$ in $\sum_j p_{ij}(a_k)$ are not completely independent of each other. Instead, as a result of the requirements in Equations 9 and 10, the degrees of freedom for both sets of parameters are reduced by one. This property should be taken into consideration in the analysis design.

In this sensitivity analysis, the dependable variables include the proportion of roads in condition state j at the beginning of lth time period, and to which the k preservation action is applied ($w^l_{j,k}$), and total agency cost $\left[\sum_{l=1}^{T-1}\sum_{i,k} w^l_{i,k} \cdot d_l \cdot c(i,k)\right]$, which is the objective function. The independent variables include transition probabilities $p_{ij}(a_k)$ and current conditions q_i. Table 1 shows the current pavement conditions for the road category of desert interstate highways. There are 45 condition states, determined by three factors: ride level (roughness), distress level (cracking), and index to first crack. The index to first crack was conceptually an estimate of the time between the construction or rehabilitation of the pavement to occurrence of the first crack. However, this index is used in both the original NOS and AZNOS to select a TPM based on the most recent rehabilitation. There are five levels of the index to first crack based on the type of rehabilitation treatment as shown in Table 1, corresponding to five levels of rehabilitation actions. For example, 18.25 percent of the pavement area was in Condition State 1 (low roughness and cracking levels, and never rehabilitated except for routine maintenance). There were 20.44 percent of the pavements in Condition State 19 (low roughness and cracking levels, and the last rehabilitation is Action Number 4). It should be noted that pavements with the most recent treatment of Action 2 or 3 converge to Conditions 10 to 18 after the action is applied. Condition States 10 to 18 fall within Index to First Crack 2. However, these two treatments of seal coat and ACFC are different in their effectiveness, resulting in the two different transition probabilities for Actions 2 and 3 for the year that the actions are applied. With the exception of seal coat and ACFC, a probability of 1 is assumed for the transition from any condition state to the condition state with low roughness and cracking levels during the year the rehabilitation action is applied. Table 2 presents the complete sets of transition probabilities, or transition probability matrices under routine maintenance, for the subnetwork under study.

The majority of pavement (65.81 percent) is at the levels of low cracking and low roughness, or the best condition state (see Table 1). In addition, the majority of pavements receive only routine maintenance. Because 20.44 percent of pavements are in Condition 19, it is determined to start the analysis by varying the transition probabilities from Condition State 19 to States 19, 20, 22, and 23. The second analysis includes simultaneously varying the transition probabilities from States 1, 10, 19, and 28. Data relating to State 37

TABLE 1 Current Pavement Conditions in Percentage of Area for the 45 Condition States, Road Category of High-Traffic and Desert Interstate Highways in Arizona (1992)

Ride Level	Distress Level	Index[a] 1		Index 2		Index 3		Index 4		Index 5		Total
		CS[b]	% of Area	CS	% of Area	CS	% of Area	CS	% of Area	CS	% of Area	
Low	Low	1	18.25	10	10.54	19	20.44	28	15.17	37	1.41	65.81
Low	Medium	2	5.27	11	1.93	20	3.60	29	2.31	38	0.26	13.37
Low	High	3	1.67	12	0.64	21	0.77	30	0.90	39	0.00	3.98
Medium	Low	4	3.08	13	0.90	22	2.06	31	2.44	40	0.77	9.25
Medium	Medium	5	1.67	14	0.39	23	0.39	32	0.26	41	0.00	2.71
Medium	High	6	1.29	15	0.26	24	0.64	33	0.13	42	0.00	2.32
High	Low	7	0.13	16	0.00	25	0.13	34	0.13	43	0.13	0.52
High	Medium	8	0.51	17	0.13	26	0.00	35	0.00	44	0.00	0.64
High	High	9	1.40	18	0.00	27	0.00	36	0.00	45	0.00	1.40
Total			33.28		14.79		28.03		21.34		2.57	100.00

[a] Index stands for index to first crack.
[b] CS stands for condition states.

TABLE 2 Transition Probabilities Under Routine Maintenance for High-Traffic and Desert Interstate Highways in Arizona (Truncated from 6 Decimals to 4)

From[a]	To[b]	Tran Prob[c]	From	To	Tran Prob	From	To	Tran Prob	From	To	Tran Prob	From	To	Tran Prob
1	1	0.8540	10	10	0.8372	19	19	0.8477	28	28	0.8577	37	37	0.8547
1	2	0.0545	10	11	0.0465	19	20	0.0700	28	29	0.0300	37	38	0.0800
1	4	0.0836	10	13	0.1093	19	22	0.0786	28	31	0.1086	37	40	0.0572
1	5	0.0078	10	14	0.0070	19	23	0.0037	28	32	0.0037	37	41	0.0080
2	2	0.7237	11	11	0.6452	20	20	0.8137	29	29	0.8237	38	38	0.8512
2	3	0.1447	11	12	0.0645	20	21	0.1091	29	30	0.0991	38	39	0.0855
2	5	0.0833	11	14	0.2258	20	23	0.0748	29	32	0.0748	38	41	0.0619
2	6	0.0482	11	15	0.0645	20	24	0.0025	29	33	0.0025	38	42	0.0013
3	3	0.7391	12	12	0.9211	21	21	0.8209	30	30	0.8209	39	39	0.8709
3	6	0.2609	12	15	0.0789	21	24	0.1791	30	33	0.1791	39	42	0.1291
4	4	0.8707	13	13	0.8835	22	22	0.8276	31	31	0.8676	40	40	0.8513
4	5	0.0532	13	16	0.1068	22	23	0.1010	31	32	0.0610	40	41	0.0929
4	7	0.0722	13	17	0.0097	22	25	0.0659	31	34	0.0659	40	43	0.0515
4	8	0.0038			N/A	22	26	0.0055	31	35	0.0055	40	44	0.0043
5	5	0.6429	14	14	0.7868	23	23	0.7970	32	32	0.7970	41	41	0.7992
5	6	0.1607	14	15	0.0336	23	24	0.0273	32	33	0.0273	41	42	0.0237
5	8	0.0357	14	17	0.1724	23	26	0.1699	32	35	0.1699	41	44	0.1720
5	9	0.1607	14	18	0.0072	23	27	0.0059	32	36	0.0059	41	45	0.0051
6	6	0.8732	15	15	0.8230	24	24	0.8229	33	33	0.8229	42	42	0.8229
6	9	0.1268	15	18	0.1770	24	27	0.1771	33	36	0.1771	42	45	0.1771
7	7	0.9400	16	16	0.8681	25	25	0.8590	34	34	0.8590	43	43	0.8935
7	8	0.0600	16	17	0.1319	25	26	0.1410	34	35	0.1410	43	44	0.1065
8	8	0.9456	17	17	0.9577	26	26	0.9607	35	35	0.8662	44	44	0.9711
8	9	0.0544	17	18	0.0423	26	27	0.0393	35	36	0.1338	44	45	0.0289
9	9	1.0000	18	18	1.0000	27	27	1.0000	36	36	1.0000	45	45	1.0000

[a] Condition states to be transitioned from.
[b] Condition states to be transitioned to.
[c] Transition probabilities.
Note: refer to Table 1 for the corresponding roughness level, cracking level, and index number for each condition state.

were not used because a relatively small number of pavements (1.41 percent) were in this particular state. The third analysis includes varying the transition probabilities and current condition states in relation to data for the transitions from State 19 to States 19, 20, 22, and 23.

RESULTS OF SENSITIVITY ANALYSES

Varying Transition Probabilities from State 19

In Table 2 the transition probabilities from states at low roughness and cracking levels to states at the same levels fall within the range of 0.8372 to 0.8577. These probabilities play a critical role in keeping the pavements in the best condition states. Probabilities related to Index to First Crack 3 were selected in this analysis by varying the probabilities in the order shown in Table 3. Six runs were conducted. It should be noted that the transition probabilities for pavements in State 19 to stay in State 19 were varied from 0.6 to 0.7, 0.8, and 0.9 with the increment of 0.1, and from 0.9 to 0.95 and 0.99 with the increments of 0.05 and 0.04. Different increments were used to vary the probabilities because in initial AZNOS runs when transition probabilities were lower than 0.8, there were only small variations among the different AZNOS budget recommendations. That is to say, the AZNOS-based budget recommendations stay relatively stable when the probability to stay in the best state is smaller than 0.8. Figure 3 is a three-dimensional chart for these six runs. The following data show the budget recommendations of the six AZNOS runs based on the transition probabilities in Table 3 (in millions of dollars):

TABLE 3 Variations of Transition Probabilities from State 19 to States 19, 20, 22, and 23

Run Number	$p_{19,19}(1)$	$p_{19,20}(1)$	$p_{19,22}(1)$	$p_{19,23}(1)$
1, (TPM 1)	0.600	0.175	0.175	0.050
2, (TPM 2)	0.700	0.135	o.135	0.030
3, (TPM 3)	0.800	0.095	0.085	0.001
4, (TPM 4)	0.900	0.050	0.050	0.000
5, (TPM 5)	0.950	0.025	0.025	0.000
6, (TPM 6)	0.990	0.005	0.005	0.000

Run 1	Run 2	Run 3	Run 4	Run 5	Run 6
(TPM 1)	(TPM 2)	(TPM 3)	(TPM 4)	(TPM 5)	(TPM 6)
\$99.949	\$98.673	\$97.051	\$86.525	\$78.783	\$67.545

Based on the data above and the data in Table 3 and Figure 3, it is evident that a small increase for the transition probabilities to stay in the best state from 0.8 may introduce sizable savings in pavement rehabilitation costs.

Simultaneously Varying Transition Probabilities from Multiple States

The second analysis was conducted through the simultaneous varying of the transition probabilities from States 1, 10, 19, 28, and 37 to all the possible states as indicated in Table 4. Six runs were conducted on the six sets of transition probabilities. The following data show the AZNOS budget recommendations from the six runs (in millions of dollars):

Run 1	Run 2	Run 3	Run 4	Run 5	Run 6
(TPM 1)	(TPM 2)	(TPM 3)	(TPM 4)	(TPM 5)	(TPM 6)
\$124.076	\$116.951	\$106.951	\$68.713	\$33.216	\$23.209

This analysis reveals that a compounding effect occurred as a result of the simultaneous change of the transition probabilities. When the probabilities changed from 0.8 to 0.99, the budget recommendations from AZNOS were reduced drastically from \$106.51 million to \$23.209 million. Figure 4 illustrates the recommended rehabilitation costs for each action and each set of transition probability matrices.

Varying Transition Probabilities and Current Conditions

The third analysis focused on actual pavement Conditions 19 to 25 and their related transition probabilities. For each set of transition probabilities in Table 3, six runs of AZNOS were conducted based on six sets of pavement condition data. Six proportions of roads in State 19 with low roughness and cracking levels were used as follows: 0.04, 0.07, 0.1, 0.13, 0.16, and 0.204. The last proportion (0.204) represents the actual pavement condition in 1991. The other proportions of pavement condition data were adjusted proportionally to their actual pavement conditions in Table 1. Figure 5 shows

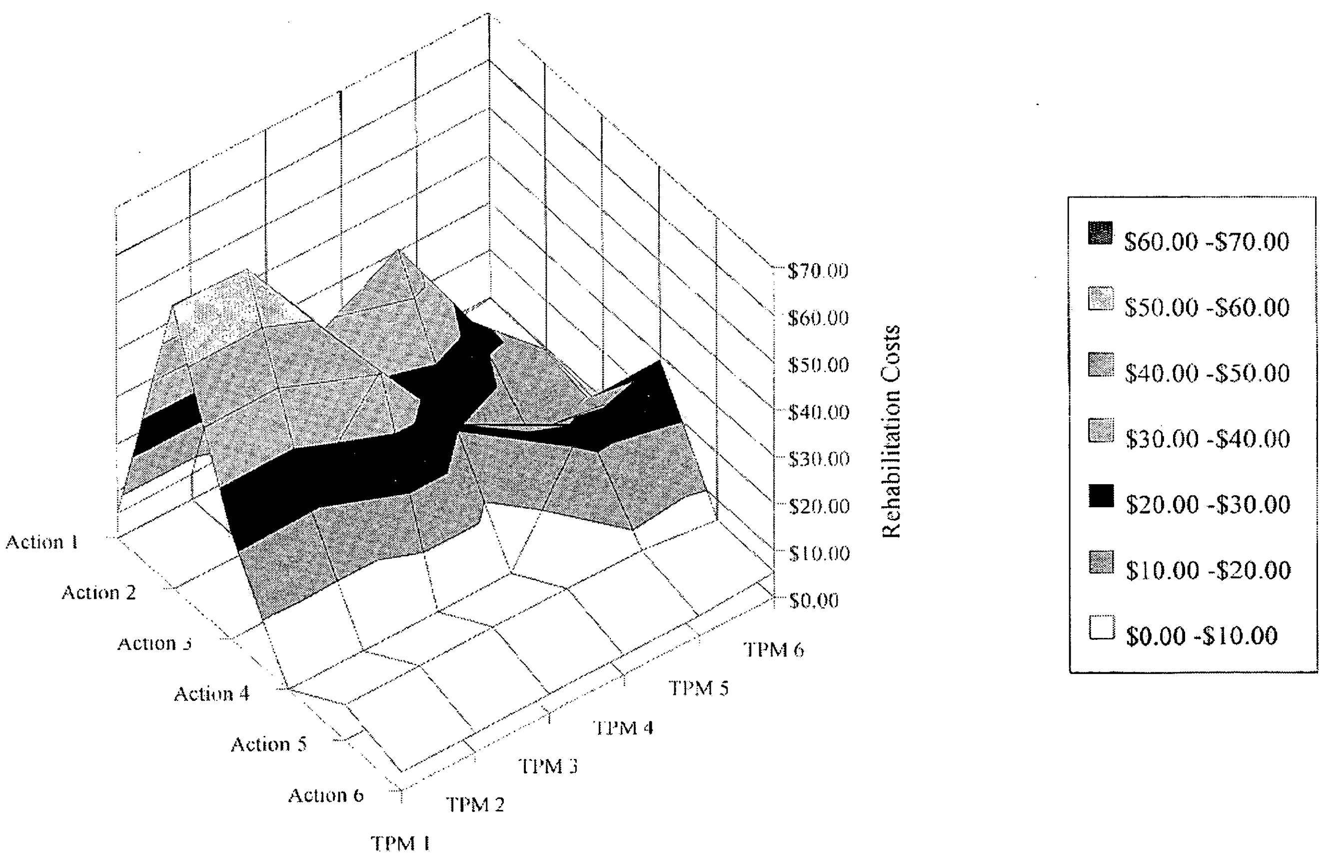


FIGURE 3 Sensitivity of AZNOS to six sets of transition probabilities from State 19 to all other states ($million).

TABLE 4 Variations of Transition Probabilities from States 1, 10, 19, 28, and 37 to All Possible States

Run Number	Probabilities to stay in the best state	Probabilities to state 2/ 11/20/29/37	Probabilities to state 4/ 13/22/31/40	Probabilities to state 5/ 14/23/32/41
1, (TPM 1)	0.600	0.130	0.240	0.030
2, (TPM 2)	0.700	0.100	0.180	0.020
3, (TPM 3)	0.800	0.070	0.120	0.010
4, (TPM 4)	0.900	0.030	0.060	0.010
5, (TPM 5)	0.950	0.015	0.035	0.000
6, (TPM 6)	0.990	0.003	0.007	0.000

the results of this analysis through the use of a three-dimensional surface. CC1 to CC6 represent the six sets of pavement condition data. Figure 5 indicates that the changing proportions of pavement conditions have limited effects on recommended budget needs when the transition probabilities to stay in Condition 19 were smaller than 0.8. However, when the transition probabilities to stay in the best state changed from 0.8 to 0.99, for each set of pavement condition data, a large decline in recommended budget needs was exhibited. The sharp declining slope toward the right-front corner of the three-dimensional surface in Figure 5 demonstrates the compounding effect of improved pavement condition and higher transition probabilities for pavements to stay in the best condition with low roughness and cracking levels.

CONCLUSION

The higher the transition probabilities for pavements to stay in the best condition state, the less proportions of pavements will transition to worse states. As a result, a smaller budget will be needed. It is also evident that the better the pavement conditions, the smaller the needed budget will be for future pavement rehabilitation. These two properties were quantitatively analyzed in this paper by using AZNOS. An interesting property was also revealed in the analysis: when transition probabilities were increased from 0.8, budget needs for pavement rehabilitation based on AZNOS were drastically decreased, disproportionally against the increasing rate of the probabilities. As transition probabilities were determined based on past pavement performance in Arizona, this newly revealed property encourages preventive pavement improvements to reduce future rehabilitation needs. This property also illustrates that a modest increase in costs for preventive maintenance may well generate large future savings. Therefore, efforts to improve current pavement roughness and cracking levels, which will be used to update future TPMs as past pavement performance data, will ultimately improve the lifelong cost-effectiveness of rehabilitation programs for pavement networks. It should be pointed out that this paper does not discuss the sensitivities to cost matrices and discount rates. These two factors also play important roles in determining long-term pavement rehabilitation costs.

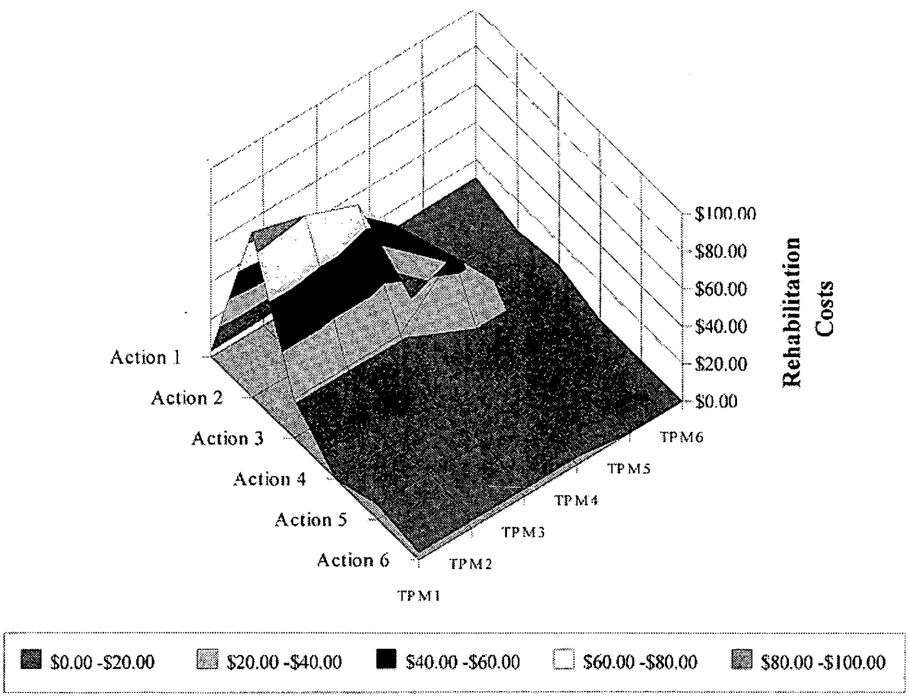


FIGURE 4 Sensitivity of AZNOS to simultaneous variations of transition probabilities from States 1, 10, 19, and 28 to all other states ($million).

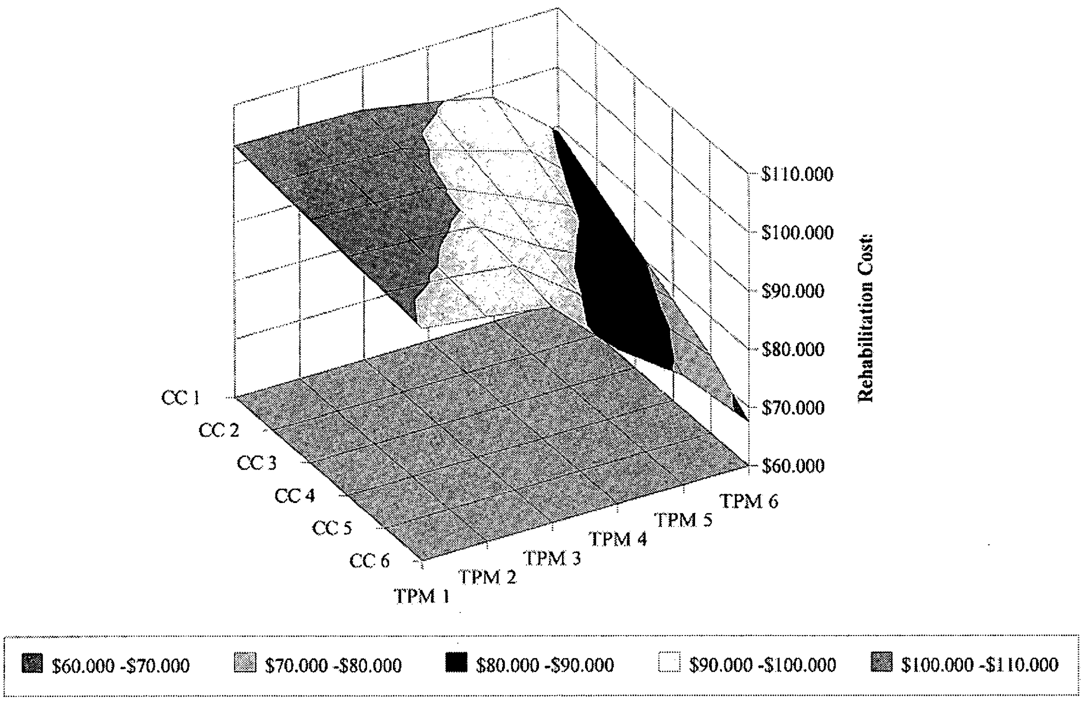


FIGURE 5 Sensitivity of AZNOS to simultaneous variations in transition probabilities from State 19 and current-condition States of 19 to 24 ($million).

ACKNOWLEDGMENTS

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REFERENCES

1. Wang, K. C. P., J. Zaniewski, and J. Delton. Microcomputer Implementation of Pavement Network Optimization System. *ASCE Journal of Computing in Civil Engineering*, Vol. 7, No. 4, Oct. 1995, pp. 495–510.
2. Wang, K. C. P., J. Zaniewski, and G. Way. Probabilistic Behavior of Pavements. *ASCE Journal of Transportation Engineering*, Vol. 120, No. 3, May–June 1994, pp. 358–375.
3. Wang, K. C. P., J. Zaniewski, G. Way, and J. Delton. Revisions to Arizona DOT Pavement Management System. In *Transportation Research Record 1397*, TRB, National Research Council, Washington, D.C., 1993, pp. 68–76.
4. Wang, K. C. P., J. Zaniewski, and J. Delton. Analysis of Arizona DOT's Pavement Management System. Presented at 73rd Annual Meeting of the Transportation Research Board, Washington, D.C., 1994.
5. Kulkarni, R., K. Golabi, F. Finn, and E. Alviti. *Development of a Network Optimization System.* Woodward-Clyde Consultants, San Francisco, Calif., 1980.
6. Rauhut, B. J., I. D. Darter, and O. J. Pendleton. *Early Analysis of Long-Term Pavement Performance General Pavement Studies Data: Lessons Learned and Recommendations for Future Analyses.* SHRP-P-680. Strategic Highway Research Program, Washington, D.C., 1994.

Calibration of HDM-III Performance Models for Use in Pavement Management of South African National Roads

L. KANNEMEYER AND A.T. VISSER

The applicability of the HDM-III performance models for predicting local observed deterioration was evaluated so that the timing, type, and cost of maintenance needs could be estimated and a balanced expenditure program developed for South African national roads. The validation procedure and the assessment methodologies used in the calibration of the environmental influences and the cracking, rutting, and roughness models are presented. For the HDM-III performance models evaluated, calibration values of less than one were obtained—except for rutting—indicating, in general, better performance on South African national roads than predicted. Based on the results obtained, it is concluded that after calibration the HDM-III performance models are capable of accurately predicting the observed deterioration on South African national roads, and it is recommended that these models be considered for incorporation into a balanced expenditure program for the national road network of South Africa.

The primary road network in South Africa has been established over the last half century and has been planned, constructed, and maintained to provide an acceptable level of service. However, the acute shortage of funds for roads in South Africa is endangering the integrity of this network, putting a considerable emphasis on rationalizing planning in the area of pavement maintenance and rehabilitation. Thus, pavement management, defined as *the total range of activities required to provide the pavement portion of the public works program* (*1*), has become more important.

An essential activity of pavement management is the modeling of the changes in pavement condition with accumulated use, generally known as pavement deterioration. The pavement management system used on national roads in South Africa does not yet incorporate these pavement deterioration prediction models. At present, the current condition of a pavement is used as a trigger for action to identify maintenance or rehabilitation projects for further evaluation. As illustrated in Figure 1, this method has a low probability of selecting the optimum rehabilitation strategy if the expected future deterioration of a pavement is not considered. Although both Pavements A and B in Figure 1 have the same level of riding quality after *T* years, their expected future deterioration differs to a large extent. This demonstrates the need to use deterioration prediction models in pavement management systems to predict the timing, type, and cost of future maintenance needs.

An extensive study (*2*) was executed to evaluate the applicability of models developed internationally for predicting the deterioration of the South African national road network. The study consisted of a literature review of international deterioration models developed from the deterioration results of in-service pavements under the normal traffic spectrum, avoiding models developed from accelerated testing with stationary devices. The reasons for avoiding these models are that the long-term effects are virtually eliminated (they are primarily environmental but also include effects of the rest periods or vehicle headway) and that the unrepresentative traffic loading regimes can distort the behavior of the pavement materials, which is often stress dependent (*3*). From the literature review, the HDM-III models were identified as possibly applicable and were subsequently validated through an analytical approach using the data obtained under the normal traffic and environmental conditions experienced over the past 15 years on the national road network of South Africa.

The aim of this paper is to demonstrate the suitability of the HDM-III performance models, once calibrated, for predicting the performance of South African national roads. After presenting the validation procedure and the assessment methodologies, the calibration of the environmental influences and the cracking, rutting, and roughness models are presented.

VALIDATION PROCEDURE

The approach embarked on during the validation was to first calibrate the environmental coefficient (m) for the different Thornthwaite moisture regimes in South Africa, then the HDM-III deterioration models. Based on the fact that the equations defining the different HDM-III deterioration models are of the exponential type, it follows that the accuracy of any prediction tends to decline as the time period increases. Thus, the value of the local calibration factor determined for each model would be valid only over the medium term. Based on this, it was decided to employ the same approach as that used in Chile to adapt the HDM-III models to their local conditions (*4*). The approach involved the periodic calibration of the models to correct the deterioration curves in such a way that they maintained good predictions over the pavement life. Since this was a very time-consuming process, the algorithm developed in Chile that performs the calibration was adopted for South African conditions, to automatically determine the calibration factor for each deterioration model of an individual pavement section.

The calculation method employed in the algorithm is based on the minimization of the difference between the values predicted by the HDM-III models and those measured (*4*). The procedure of cali-

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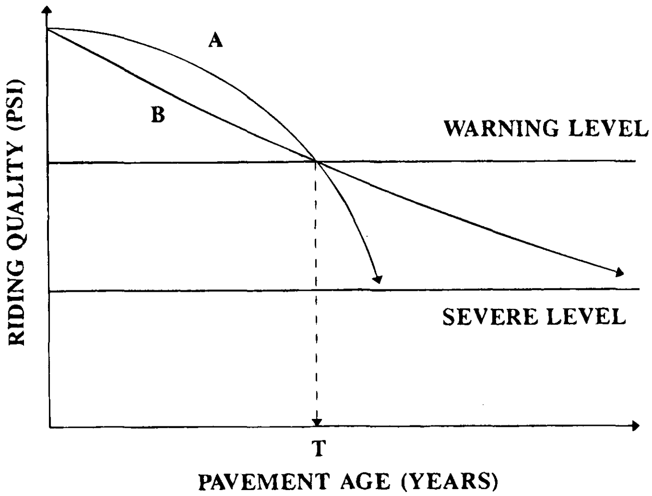

FIGURE 1 Expected future pavement deterioration.

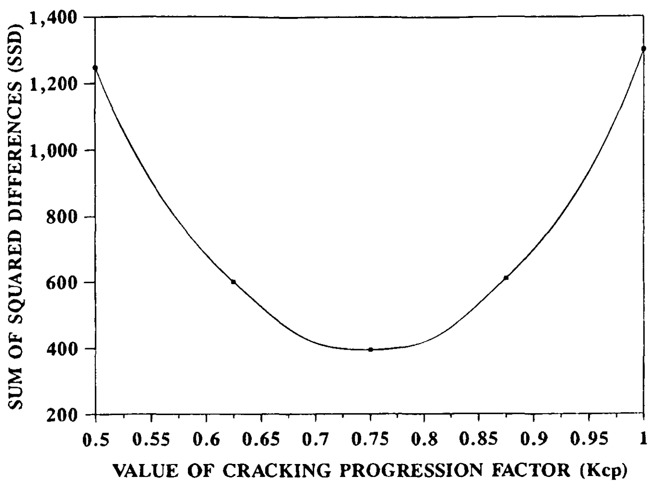

FIGURE 2 Illustration of sum of squared differences against calibration factor values.

bration involves the prediction of the change in a specific parameter over time for different calibration factor *ki* values, and then to calculate the corresponding difference between the predicted and measured values for each calibration factor value. These calculated differences are then used to determine the sum of the square of the differences that are then plotted against the specific calibration factor value. When plotted, the sum of squared differences are distributed in a parabolic shape, with a minimum at the optimum calibration factor value, as illustrated in Figure 2 for the cracking progression calibration factor (*kcp*). A parabolic curve is then fitted to the sum of squared differences (*SSD*) incorporating the calibration factor (*ki*) as follows (*4*):

$$SSD = aki^2 + bki + c \qquad (1)$$

where

SSD = sum of the squared differences,
ki = calibration factor, and
$a,b,$ and c = constants of equation obtained during the fitting of the curve.

The value obtained by taking the derivative of the equation above equals the calibration factor (*ki*) for which the *SSD* is the least, namely,

$$ki = \frac{-b}{2a} \qquad (2)$$

where

ki = calibration factor for which *SSD* is a minimum, and
a and b = constants obtained during the fitting of the parabolic curve.

The procedure above was repeated for all prediction models for each individual pavement section evaluated.

CORRELATION OF VISUAL ASSESSMENTS

Since the HDM-III model requires the area affected by cracking (all and wide) and raveling as a percentage value, the correlation of South African visual data (*2*) was needed to convert the degree and extent numerical ratings into percentage of area. Both the extent and degree numerical ratings were combined into a single value, defined as the area of indexed distress. The reason for adopting this approach is that it is believed that by incorporating both degree and extent in a single value, the value obtained will more accurately portray the pavement condition. The following conversion factor was used to convert the South African numerical ratings into the format required by the HDM-III model for each pavement section:

$$\mathrm{CRX} = \frac{\overset{N}{\Sigma} A_{TX} S_{TX}}{N} \qquad (3)$$

where

CRX = total area of indexed distress as a percentage of the surface area of the pavement section under evaluation;
A_{TX} = area of surface distress for a certain degree as a percentage;
S_{TX} = decimal factor assumed for converting the degree rating; and
N = number of visual segments in the pavement section under evaluation.

CORRELATION OF MECHANICAL MEASUREMENTS

Of the three mechanical measurements of importance on national roads, rutting as well as deflection was already in a format suitable for inclusion in the HDM-III model. The most important of these measurements, namely, the riding quality, had to be converted from present serviceability index (PSI) to quarter-car index (QI_m).

The roughness measurements in PSI, available on the data base, were used to calculate the mean PSI value for each section for a specific survey date. These mean PSI values were then correlated to the quarter-car index (QI_r) by using the correlation developed by Visser (*5*), and then to QI_m by using the relationship in Table 2.5 of Paterson (*3*). (Note QI_m equals 13 on the international roughness index).

$$QI_r = 92.63 - 56.93\ln(\text{PSI}) \tag{4}$$
$$QI_m = 9.5 + 0.9QI_r$$

where

QI_r = quarter-car index [profile RMSVA function of QI (counts/km)];
QI_m = quarter-car index [roadmeter estimate of QI roughness (counts/km)]; and
PSI = mean PSI value calculated for each pavement section.

MODIFIED STRUCTURAL NUMBER

The modified structural number (SNC), which includes the contribution of the subgrade (SN_{sg}), was calculated by using the following equation:

$$\text{SNC} = SN_i + SN_{sg} \tag{5}$$

where

SNC = modified structural number;
SN_i = initial structural number in first year of modeling;
SN_{sg} = contribution of the subgrade after Hodges et al. (*6*): = $3.51 \log_{10} \text{CBR} - 0.85 (\log_{10} \text{CBR})^2 - 1.43$; and
CBR = in situ California bearing ratio of subgrade in percentage.

The initial structural number (SN_i) was determined by using correlations developed by Rohde (*7*), whereby a pavement's structural number can be determined from its total thickness and the shape of the measured surface deflection bowl obtained from a falling weight deflectometer (FWD). The correlations are based on the general "two-thirds rule" suggested by Irwin (*8*) to explain the stress distribution and thus origin of deflections found below an FWD. This rule is based on the fact that approximately 95 percent of the deflection measured on the surface of a pavement originates below a line deviating 34 degrees from the horizontal. Based on this simplification, it can be assumed that the surface deflection measured at an offset of 1.5 times the pavement thickness originates entirely in the subgrade. By comparing this deflection with the peak deflection, the following index associated with the magnitude of deformation that occurs within the pavement structure was defined by Rohde (7):

$$\text{SIP} = D_0 - D_{1.5Hp} \tag{6}$$

where

SIP = structural Index of the pavement;
D_0 = peak deflection measured under a standard 40-kN FWD impulse load;
$D_{1.5Hp}$ = surface deflection measured at an offset of 1.5 times Hp under a standard 40-kN FWD impulse load; and
Hp = total pavement thickness.

To develop a relationship between FWD-measured surface deflection and a pavement's structural number, 7,776 pavement structures were analyzed using layered elastic theory. The SN for each pavement was calculated by using the following approach suggested by AASHTO (*9*):

$$\text{SN} = 0.04\Sigma a_g h_i (E_i/E_g)^{1/3} \tag{7}$$

where

SN = structural number;
a_g = material and layer strength coefficients, per inch;
h_i = layer thickness, mm (where $\Sigma\, h_i \leq 700$ mm);
E_i = resilient modulus of pavement layer; and
E_g = resilient modulus of standard materials in the AASHTO Road Test.

By comparing the calculated SN with the parameters previously defined, Rohde (*7*) obtained the following relationship between SN and SIP:

$$\text{SN} = k_1 \text{SIP}^{k2}\, \text{Hp}^{k3} \tag{8}$$

where

SN = structural number;
SIP = structural index of the pavement (in μm);
Hp = total pavement thickness (in mm); and
ki = coefficients as listed in Table 1.

The acceptability of SN determined according to the foregoing procedure was continuously verified by using the approach suggested by AASHTO (*9*). Where noticeable differences existed between the two methods (e.g., unrealistic high SN predicted according to procedure above, normally associated with unrealistic low deflections), the SN determined according to the AASHTO (*9*) approach was used.

Finally the initial structural number SN_i was defined as the structural number calculated according to procedure above less the contribution of any maintenance actions within the period between the date of the FWD measurements and the date used as the initial year of modeling:

$$SN_i = \text{SN} - 0.04\, \Sigma\, a_i h_i \tag{9}$$

where

SN_i = initial structural number in first year of modeling;
SN = structural number;
a_i = material and layer strength coefficients, per inch; and
h_i = thickness of overlay, reseal, and so forth, in mm.

ENVIRONMENTAL ROUGHNESS CALIBRATION FACTOR (*Kge*)

The environmental roughness calibration factor (*Kge*) is the exponential annual rate of increase in roughness due to environmental effects. The environmental roughness calibration factor (*Kge*) is calculated from the environmental coefficient (*m*) as follows:

$$Kge = m/0.023 \tag{10}$$

where

Kge = environmental roughness calibration factor; and
m = environmental coefficient.

Advice as to recommended values for the environmental coefficient (*m*) for various climatic regions is given in Table 8.7 of Paterson (*3*). However, Paterson (*3*) cautions that these recommended

TABLE 1 Coefficients for SN Versus SIP Relationships (7)

Surfacing type	k1	k2	k3
Surface Seals	0,1165	-0,3248	0,8241
Asphalt concrete	0,4728	-0,4810	0,7581

values are based on relatively few evaluations. Based on this and the advice given by Paterson during the Botswana calibration of HDM-III, it was decided to follow his recommended method for determining a value for the environmental coefficient (*m*) from roughness measurements.

The recommended procedure first runs HDM-III with the environmental roughness calibration factor set at zero. This establishes the contribution of traffic to the increase in roughness. The environmental coefficient (*m*) is then approximated by dividing the difference between the total increase and the increase due to traffic by the product of the mean roughness and the number of years since construction. In mathematical terms:

$$m = \frac{(Rm - Ri) - (Rp - Ri)}{(Rm + Ri) \times (T/2)} \tag{11}$$

where

Rm = measured roughness;
Ri = initial roughness;
Rp = predicted roughness with $Kge = 0 = m$; and
T = number of years between measurement date and construction date.

For this study, the pavement sections under evaluation were subdivided into the different Thornthwaite moisture regimes occurring on South African national roads. Since multiple observations existed under each moisture regime, the best estimate for *m* was given by the quotient of the sums of the individual numerators and denominators. The results obtained for the different moisture regimes are summarized in Table 2.

As seen, the calculated environmental roughness calibration factor (*Kge*) in each instance is nearly half of the value recommended by Paterson (*3*) for that moisture regime. Thus, the influence of the environment on the pavement deterioration observed on South African national roads, is only about half of what is predicted by the HDM-III model. Possible contributing factors include the following.

- The generally more balanced deep pavement structures used in South Africa, which result in more support for the surface layer, and, as such, decrease the induced stresses within the upper layers. This results in a longer period before initiation of cracking. This is in contrast to the relatively shallow pavements used during the development of the models.
- The design and quality control during construction in South Africa, which result in a high-quality finish with adequate provision for surface as well as subsurface drainage.
- The maintenance activity employed on South African national roads, which include routine activities such as crack sealing and periodic overlays or reseals, which decrease the environmental influences, thus increasing the life of a pavement.

CRACKING MODEL

Cracking is modeled in two phases: the time before initiation of cracking and the rate of progression of cracking for both all and wide cracking. The cracking model relates the change in cracking to

Incremental cracking area = *Kcp* {*Kci f*(equivalent standard axles, construction quality, structural strength, base type) + *f*(area previous cracking)}

where

Kci = user-defined factor for local calibration of all cracking initiation; and
Kcp = user-defined factor for local calibration of all cracking progression.

The reason for not having calibration factors for wide cracking, is that the initiation of wide cracking was defined as a function of the initiation of all cracking.

Initially the pavement sections were evaluated individually based on the surfacing type, base course type, and climatic area. No noticeable difference in performance between the different surfacing layers or base course layers existed. It is believed that for the surfacing, this is the result of using asphalt layers on South African national roads that are normally thin (30–40 mm) and using Cape seals as surface treatments. The Cape seal has one or more slurry

TABLE 2 Environmental Coefficient (m) for Different Moisture Regimes

Moisture regime	Semi-Arid	Subhumid	Humid
Calculated value for m	0,009	0,014	0,020
Calculated value for Kge	0,392	0,607	0,886
No of observations	20	25	20
Paterson (*3*) value for kge	0,70	1,30	1,74

seals on top of the 19-mm single seal aggregate. This improves the impermeability of the layer and also limits raveling to a large extent, which leads to an improvement in the performance of the seal. For these reasons, similar performance of the surfacing types were found. For the different climatic areas, no noticeable difference existed in the cracking initiation and progression calibration factor values. Thus, the calibration values for cracking initiation and progression were evaluated for all pavement sections regardless of surfacing type, base course type, or climatic area. The only noticeable difference was between original constructed pavements, and those with overlays or reseals.

The cracking initiation calibration values (*Kci*) for original constructed pavements followed a normal distribution with an average value of 1.41 and a standard deviation of 0.59 on 65 roads. This indicates that the period until the initiation of cracking on South African national roads is longer than the period predicted by the HDM-III model for the same volume of traffic. For overlays or reseals, the cracking initiation calibration values also followed a normal distribution, with an average value of 0.63, and a standard deviation of 0.17 (31 roads). This indicates that the expected life until cracking initiation tends to be lower for an overlay than the value predicted by the HDM-III model for the same traffic volume. It is believed that this is the result of the average overlay thickness of 30–40 mm generally used in South Africa being less than the average overlay thickness of 50–125 mm used in the Brazil study, from which the HDM-III cracking models were developed. This thinner layer thickness results in a shorter propagation length for cracks, with a subsequent faster rate of cracking initiation for South African overlays.

The cracking progression calibration values (*Kcp*) for original surfacings also followed a normal distribution, with an average value of 0.21 ($\sigma = 0.08$) indicating that the progression of cracking observed on national roads is lower than the rate of progression predicted by the HDM-III model for the same volume of traffic. The same was applicable for overlays and reseals, with an average value of 0.59 ($\sigma = 0.40$). Possible factors contributing to the aforementioned observations are as follows:

- The routine maintenance program employed ensures that a road is sealed or overlaid within an average of 8 years. This activity severely limits the probability for cracking initiation and progression to the severe rates observed during the Brazil study, as is evident in the low areas of cracking observed.
- In South Africa, the asphalt type used is of a semigap grading, whereas the type generally used in the Brazil study was continuously graded. It is known that a semigap graded asphalt is more resistant to fatigue than the continuously graded, resulting in a longer period before the initiation of cracking and a slower rate of progression once initiated.

The HDM-III model predictions after calibration compare favorably with the observed values, as is evident from Figure 3 with an R-squared value of 0.91 obtained for original surfacings and Figure 4 with an R-squared of 0.94 obtained for overlays and reseals. Since only a limited number of sections with relatively larger areas of cracking were available, it was impossible to evaluate the prediction of area by the HDM-III models for larger areas.

Thus it was concluded that for the low areas of cracking observed on national roads, the HDM-III model predictions after calibration seem reasonable. It is recommended that the ranges in Table 3 be used in the selection of a calibration factor value, if an individual value is not available for the specific section.

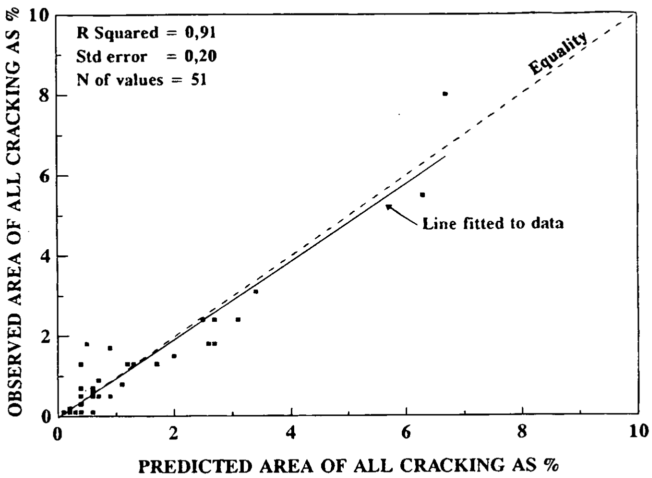

FIGURE 3 Comparison between predicted and observed values for area of all cracking for original surfacings.

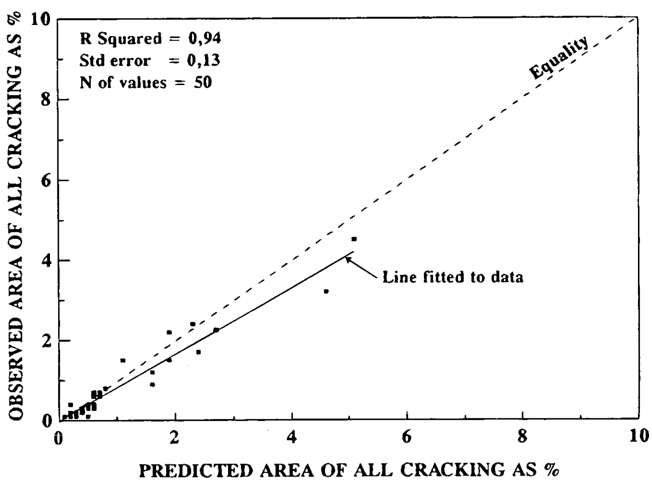

FIGURE 4 Comparison between predicted and observed values for area of all cracking for overlays and reseals.

RAVELING MODEL

Raveling is also modeled in two phases, namely, the time before initiation of raveling, and the rate of progression of raveling. The raveling model relates the change in raveling to

Incremental raveling area = Kvi f(surfacing type, construction quality, traffic) + f(previous area of raveling)

where *Kvi* is the user-defined factor for local calibration of raveling initiation.

Since no accurate method existed for correlating the historic data, no correlation values could be determined for the raveling model of the HDM-III model. It is believed that this would not affect the calibration values of the other models adversely, since the influence of raveling on potholing is of importance only when the area of raveling exceeds 30 percent, which never occurred on the sections eval-

TABLE 3 Recommended Range for Calibration Factor Values of the Cracking Model

Pavement type	Cracking initiation (Kci)	Cracking progression (Kcp)
Original surfacings	1,00-1,50	0,1-0,3
Overlays and reseals	0,4-0,8	0,3-0,7

uated. Thus, it is recommended that for raveling initiation (*Kvi*), a default value of one should be used, until calibration values are determined from a more accurate source of information.

POTHOLING MODEL

The pothole model relates the change in pothole area to

Incremental pothole area = *Kpp f*(wide cracking, raveling, previous pothole area)

where *Kpp* is the user-defined factor for local calibration of pothole progression.

In the HDM-III model, the minimum requirements for the initiation of the pothole models were defined as a minimum area of wide cracking of 20 percent for asphalt surfacings, or a minimum raveled area of 30 percent for surface treatments. As a result of the maintenance activity of patching of all potholes, and the area of wide cracking never exceeding 20 percent for asphalt surfacings or the area of raveling never exceeding 30 percent for surface treatments, the pothole models were never initiated within the HDM-III model. Thus, no method for determining calibration factor values for the pothole models existed, since no predictions were made by the HDM-III model. Thus, it is recommended that for the pothole progression calibration factor (*Kpp*), a default value of one be used until further information becomes available.

RUTTING MODEL

The rutting model consists of the mean rut depth model and the rut depth standard deviation model. The rutting model relates the change in mean rut depth and rut depth standard deviation as follows:

Incremental mean rut depth = *Krp f*(time, equivalent axle load, structural number, compaction, deflection, precipitation)

Incremental standard deviation = *Krp f*(mean rut depth, structural number, compaction, equivalent standard axles)

where *Krp* is the user-defined factor for local calibration of rut depth progression.

The mean rut depth model is not used directly in the HDM-III but is used instead as a means to estimate the variation of rut depth (standard deviation) that contributes directly to the roughness model. For the rutting model, HDM-III allowed for only a user-defined calibration factor for the progression of rutting, *Krp*. Since the use of rut depth measurements at a network level on national roads ceased in 1987, only a limited number of rut depth measurements were available for evaluation.

The calibration values for rut depth progression appear to follow a normal distribution with an average value of 1.57. It is believed that this average value does not necessarily indicate a faster rate of rut depth progression for South African pavements. The reason for this being higher than one is that in South Africa a 2-m straight edge is used compared with a 1.2-m straight edge used in the development of the model. No direct correlation between ruts measured with the different straight edges was found.

In Figure 5, the comparison between predicted values and observed values is illustrated for the mean rut depth, and for rut depth standard deviation in Figure 6. From Figure 5, it is evident that for the limited number of rut depth measurements available on national roads, the predictions given by the HDM-III model after calibration is not that favorable, with an *R*-squared of 0.68 being obtained. From Figure 6, it is evident that the correlation obtained for rut depth standard deviation is even worse, with an *R*-squared value of 0.28 being obtained. The limited data available, as well as the difference in straight edge length, are believed to contribute to the poor correlations. It is recommended that the calibration range in Table 4 be used for the rut depth progression factor (*Krp*).

ROUGHNESS MODEL

This model combines the predictions of all the previously mentioned models into a single value, which forms the basis for deter-

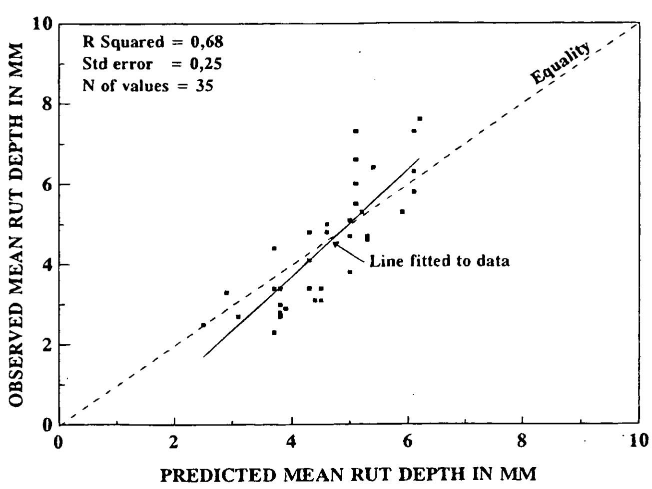

FIGURE 5 Comparison between predicted and observed values for mean rut depth.

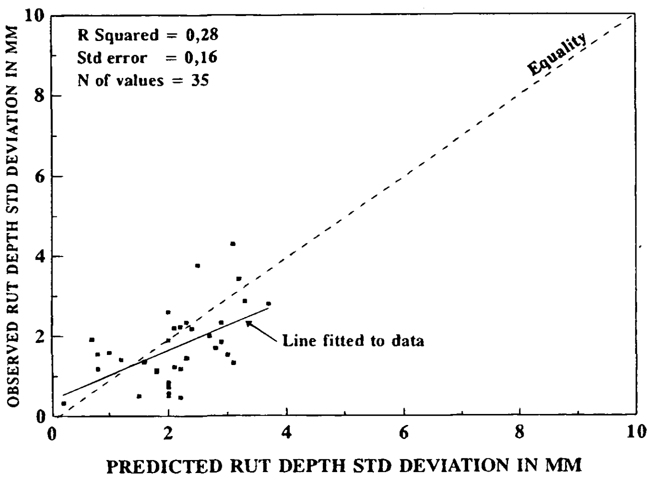

FIGURE 6 Comparison between predicted and observed values for rut depth standard deviation.

mining vehicle operating costs, and economic intervention levels. The incremental roughness model relates the changes in roughness to

Incremental roughness = *Kgp*{*f*(structural number, incremental traffic loadings, extent of cracking, thickness of cracked layer, incremental variation in rut depth) + *f*(changes in cracking, patching, and potholing)} + *Kge*{*f*(pavement environment, time, and roughness)]

where

Kgp = user-specified factor for local calibration of roughness progression, and
Kge = user-specified factor for local calibration of the environment-related annual fractional increase in roughness.

The environment-related calibration factor, *Kge*, is fixed to certain values, defined on the basis of the Thornthwaite moisture index, as discussed previously. Initially, the pavement sections were evaluated individually based on the surfacing type and base course type and whether it was an original constructed surface layer or an overlay or reseal. No noticeable difference in performance existed between the different surfacing layers and base course layers or between original surfacings or overlays and reseals. Thus, only differences in moisture regime were allowed for during the calibration for roughness progression (*Kgp*).

For semiarid areas, the roughness progression calibration factor (*Kgp*) followed a normal distribution with 85 percent of the calibration factor values falling within the range 0.8 to 1.2, with an average of 1.02. The same applied to subhumid areas with 88 percent of the calibration factor values falling within the range 0.6 to 1.4, with an average of 0.95. The aforementioned also applied to humid areas with 70 percent of the calibration factor values falling within the range 0.8 to 1.2, with an average of 0.99. As with the previous two moisture areas, the average value obtained indicates that the observed roughness deterioration on South African national roads is equal to the value predicted by the HDM-III model. Thus, after calibrating the HDM-III model for local environmental conditions, it seems that little or no calibration is needed for the roughness progression model, indicating that the deterioration predicted by the HDM-III for traffic-related distress seems to be similar to the deterioration observed on South African national roads. Furthermore, this indicates that the expected difference in behavior between the different climatic areas is taken into consideration by the environmental roughness calibration factor (*Kge*), which increases or decreases the rate of deterioration as required.

The ability of the HDM-III model to predict the roughness observed on national roads after calibration is illustrated in Figure 7 for all pavement sections evaluated. As seen from the figure, an *R*-squared value of 0.9 was obtained, indicating that after calibration, the HDM-III model is capable of accurately predicting the roughness deterioration observed on South African national roads. Thus, the use of the HDM-III deterioration models for predicting the deterioration observed on South African national roads is highly recommended, as is evident in Figures 8 and 9, in which the observed roughness is compared with the predicted roughness for an individual pavement section evaluated. It is also obvious from these figures that the maintenance activity employed on South African national roads did not allow the evaluation of the exponential nature of the HDM-III models. The reason is that when maintenance is timely, the deterioration of a pavement is kept to more or less a linear progression as seen in Figures 8 and 9.

CONCLUSIONS AND RECOMMENDATIONS

The main conclusion from the comparison of observed values with predicted values, is that the HDM-III models are capable of accurately predicting the observed deterioration on South African national roads, but that for most models calibration is needed for local conditions, especially for the environmental roughness calibration factor (*Kge*).

Despite the favorable correlations obtained for some of the HDM-III models, others could not be calibrated as a result of the lack of suitable South African deterioration data. Thus, for the raveling, potholing, and to certain extent cracking models, additional research should be conducted for determining calibration values for some models, or more accurate calibration values for other

TABLE 4 Recommended Range for Calibration Factor Values of Rut Depth Model

Pavement type	Rut depth progression (Krp)
Original surfacings	1,5-1,75
Overlays and reseals	1,0

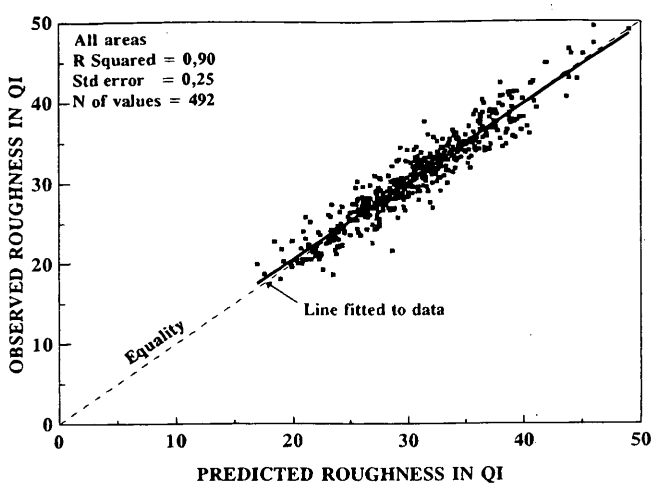

FIGURE 7 Comparison between predicted and observed roughness values for all areas.

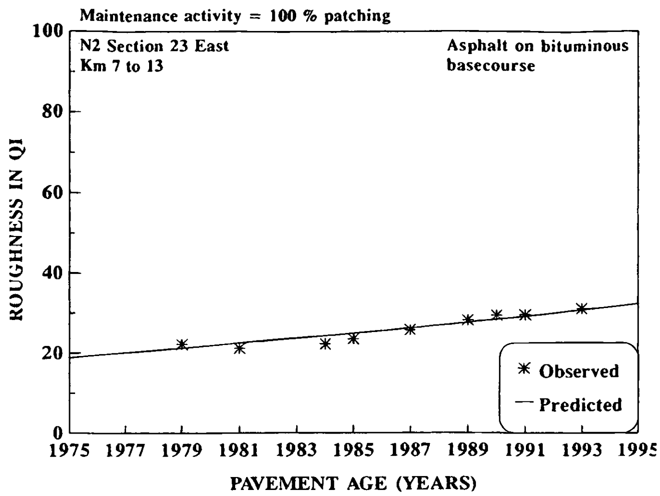

FIGURE 9 Typical illustration of comparison between observed and predicted roughness values for National Route 2, Section 23 East.

models.

Based on the results obtained for the limited number of sections included in the study, it is recommended that the HDM-III models should be considered for incorporation into a balanced expenditure program for the national roads of South Africa. The incorporation of these models would be simple since most of the models only need calibration for them to be applicable to local conditions. The incorporation of these models would allow the prediction of the rate of deterioration of a pavement and the nature of the changes so that the timing, type, and cost of maintenance needs could be estimated.

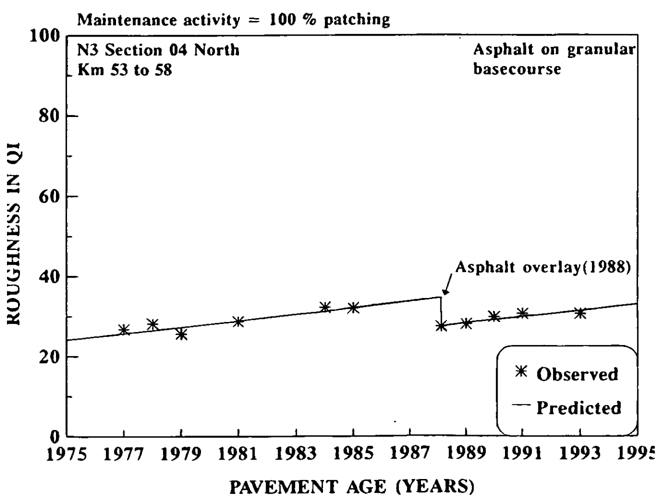

FIGURE 8 Typical illustration of comparison between observed and predicted roughness values for National Route 3, Section 04 North.

ACKNOWLEDGMENT

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REFERENCES

1. *Flexible Pavement Rehabilitation Investigation and Design.* Draft TRH 12 (unpublished). Committee of State Road Authorities Department of Transport, Pretoria, South Africa, 1991.
2. Kannemeyer, L., and A. T. Visser. *The Applicability of Published Pavement Deterioration Models for National Roads.* RDAC PR 93/799. South African Roads Board, Pretoria, 1994.
3. Paterson, W. D. O. *Road Deterioration and Maintenance Effects: Models for Planning and Management.* World Bank, Washington D.C, 1987.
4. Videla, C., G. Echeverria, and R. Gaete. Calibration of Asphalt Pavement Behaviour Models in Chile. Paper 4B/4. *Proc., Annual Transportation Convention*, Pretoria, South Africa, 1991.
5. Visser, A. T. *A Correlation Study of Roughness Measurements With an Index Obtained From a Road Profile Measured With Rod and Level.* Technical Report RC/2/82. National Institute for Transport and Road Research, Pretoria, South Africa, 1982.
6. Hodges, J. W., J. Bolt, and T. E. Jones. *The Kenya Road Transport Cost Study: Research on Road Deterioration.* Laboratory Report 673. Transportation and Road Research Laboratory, England, 1975.
7. Rohde, G. T. Determining a Pavement's Structural Number and the Subgrade Strength From Falling Weight Deflection Testing. Paper 4D/5. *Proc., Annual Transportation Convention*, Pretoria, South Africa, 1994.
8. Irwin, L. H. *Users Guide to MODCOMP 3: Version 3.6.* Local Roads Program, Cornell University, Ithaca, N.Y., 1993.
9. *AASHTO Interim Guide for the Design of Pavement Structures.* AASHTO, Washington D.C., 1986.

The opinions expressed are those of the authors and do not necessarily represent the policy of the South African Roads Board.

Evaluating Pavement Impacts of Truck Weight Limits and Enforcement Levels

EDWARD S. K. FEKPE, ALAN M. CLAYTON, AND RALPH C. G. HAAS

Efforts to compare truck productivity and pavement loading impacts of alternative truck weight limits have met with limited success because of the uncertainty surrounding the important inputs. In addition, the effects of enforcement on the resulting vehicle weights have not been adequately addressed. Parameters for evaluating pavement loading impacts of alternative truck weight limits and enforcement levels are presented. It is indicated that enforcement is a critical factor in assessing pavement impacts of alternative weight limits. For a given weight limit, the effects of enforcement on pavement loading for flexible and rigid pavements differ, with rigid pavements being more sensitive. Parameters measuring total pavement loading and taking into account the amount of payload provide a more objective assessment than the average load per truck alone. In terms of pavement costs resulting solely from axle loads, substantial savings are achievable if strict enforcement schedules are implemented.

Freight movement by trucks has important economic implications in terms of both transport costs and highway infrastructure. The physical and operating characteristics of trucks are governed primarily by the regulations limiting their sizes and weights. Very often governments are confronted with decisions that ultimately require a revision of the regulations governing vehicle weights and dimensions. Reasons for revisions of the regulatory limits include

- Promotion of commerce and economic activity;
- Improvement of operating efficiency in the trucking industry; and
- Achievement of technical harmony and promotion of trade in a geographic region (e.g., provinces of Canada, U.S. states, the countries of the North American Free Trade Agreement, European Community countries).

With increasing need for efficiency in the management of transport infrastructure and objectivity in evaluating the consequences of alternative regulations, reliable methods to forecast traffic information for direct input into pavement design, evaluation of management policies, alternative maintenance and rehabilitation strategies, and pavement performance modeling are important. Pavement impact analyses in recent studies have relied on educated estimates of pavement loadings for given regulatory and enforcement regimes (*1,2*). Uncertainties surrounding these estimates place limitations on the results. Reliable prediction procedures to assist in the management of infrastructure facilities are required.

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The purpose of this paper is to describe a method for estimating pavement loading impacts to assist in evaluating alternative weight limits. The procedure is based on a new methodology that predicts gross vehicle weight (GVW) distributions of a given truck as a function of the weight limit and the intensity or level of enforcement. The paper is also directed to the importance and quantification of the enforcement factor in pavement loading and, indeed, the overall pavement impact analysis of regulatory changes. Enforcement effect as distinct from compliance is addressed. Finally, economic consequences of different enforcement schedules are discussed.

ENFORCEMENT FACTOR

Enforcement of vehicle weight and dimension (VWD) regulations is intended to protect the highway infrastructure from premature deterioration by keeping overweight trucks off the highway system. Illegally overweight trucks rob the system of its life without reimbursing the public and compete unfairly with other trucks. The VWD regulations are meaningless unless they are enforced. The effects of nonenforcement can give rise to potentially important effects respecting public safety, fairness and equity in operations, and efficient use of public funds. These are reflected in increased pavement maintenance and rehabilitation costs due to increased pavement damage. Strict enforcement of the regulations is a step toward reducing violations, heavy-truck accidents, and highway maintenance and rehabilitation expenditures.

A truck weight study in the United States observed that estimating the effects of illegally overweight trucks on pavement costs is difficult because reliable estimates of the magnitude and frequency of illegal overloads are not available (*1*). Although the benefits of enforcing the regulations in terms of reduced pavement damage and subsequent maintenance and rehabilitation expenditures are identified, they have not been expressed objectively and quantitatively. In particular, the relationship between level of enforcement and pavement implications is not well established. Studies have shown that a high level of enforcement is associated with a high probability of noncompliance detection perceived by truckers, and consequent high compliance rates; and that truck weight distributions can be related to and expressed in terms of the weight limit and level of enforcement of the weight regulations (*3*). In addressing pertinent technical and policy issues regarding highway infrastructure management, it is therefore important to account for the enforcement effect in loading impact analyses. The level of enforcement is defined as the inspection rate or inspection capacity, that is, the number of trucks inspected as a percentage of all trucks using a highway facility.

TRUCK WEIGHT PREDICTION

In evaluating the impacts of alternative regulatory scenarios, knowledge of the probable weight distributions is required. Whereas it is easy to obtain truck weight data under the prevailing limits, weight prediction models are required to forecast the probable weights that are expected under proposed limits. New GVW distribution prediction models have been developed as a function of the GVW limit and the level of enforcement (*3*). The procedure overcomes major shortcomings of previous methods by recognizing and accounting for the effects of enforcement, establishing stability (transferability) of the models in time and space, and converting GVW distributions to axle weight distributions in an objective manner on the basis of truck weight split characteristics. The procedure and formulation of the models have been detailed by Fekpe and Clayton (*3,4*).

In this procedure, two distinctive GVW distribution "families" are identified and each represented by the most common or dominant configuration that reflects the characteristics of that family. The first family comprises configurations that are used for hauling "all-commodity" freight, where no one commodity or small number of commodities dominates. Trucks in this family are used to transport the full range of commodities (volume based and weight based), in both truckload and less-than-truckload quantities. This family is termed the "all-commodity" family and is typified by the tractor-semitrailer and straight trucks. The second family comprises truck configurations that are operated at GVWs very close to the weight limit. The probability density distributions of such configurations have a strong positive skew. This family is characterized by the double-trailer configurations. These trucks are generally used for hauling dense products (i.e., heavy weight-based commodities) in truckload quantities. This family is termed the "weight-based" family. The five-axle tractor semitrailer truck (3-S2) and the eight-axle tractor-semitrailer-semitrailer truck (eight-axle B-train, i.e., three-axle tractor plus tridem-axle semitrailer plus a second tandem-axle semitrailer) are considered the reference configurations for the "all-commodity" and "weight-based" families, respectively.

Essentially, the GVW predictive models are cumulative functions that determine the probability of the number of trucks operating at a given GVW in terms of the governing limit and the intensity of enforcement of the weight limit. The predictive models are developed for "steady state" conditions for loaded trucks, expressing the weight distributions that could be expected under particular weight limits. The steady state condition represents the situation that would exist if any change in the limits had been in effect long enough for the trucking industry to have fully adjusted the fleet to optimize operation under the new limits. For a given stable demand situation, fixed weight and dimension limits, and consistent enforcement, a "steady state" hauling situation emerges, exhibiting regularity in truck weight distributions for each given truck type (*4*). In reality due to system dynamics, a full steady state condition can be approached only in the limit.

Predicted GVW distributions of the reference trucks are translated into those of other truck types in the same GVW family based on a concept of truck substitution ratios (*3,5*). The rationale behind the development of the substitution ratio is that the GVW distributions for different vehicles in the same family are very similarly distinguished, primarily by the differences in the legal GVW limits. These ratios are factors that convert the GVW distribution of the reference truck to that of the target configuration in the same family. It is calculated as ratio of the effective GVW limit of the target truck to the effective GVW limit of the reference truck. The effective GVW limit is defined as the lesser of (a) the legislated GVW limit or (b) the sum of the axle weight limits.

Model Formulation

The parametric form of the GVW predictive model is given in Equation 1 where, for a given GVW limit and level of enforcement, the GVW distribution can be predicted. The level of enforcement is measured by the violation rate (i.e., number of trucks in violation as a percentage of all trucks inspected). This paper presents the model for the five-axle tractor-semitrailer truck (3-S2) representing the "all-commodity" GVW distribution family. This truck is the most common type in Canada and the United States, accounting for about 70 percent of all trucks. The model is given in Equation 2 as obtained from nonlinear regression analysis on truck weight data using the modified Gauss-Newton numerical search method in the SAS statistical package (*6*). The coefficient of correlation is 0.995 with a mean squared error of 0.00223. The *t*-test statistic was used to assess the goodness of fit, which indicated that at the 95 percent confidence limit, the quadratic function is sufficiently accurate in relating the variables. The model was validated with new independent data not used in its development and found to be accurate at the 95 percent confidence limit. Statistical tests used in assessing the predictive capability of the fitted model include the nonparametric two-sided Kolmogorov-Smirnov test statistic (*3,4*).

$$F(x) = \left[\frac{1}{1+f(z)}\right] P_r(x) \tag{1}$$

$$\mathrm{F}(x) = \left[\frac{1}{100 + f(z)}\right]\left[23 - 1.43x + 0.022x^2\right] \text{ for } x > 35 \tag{2}$$

where

$F(x)$ = proportion of trucks operating at GVW less than or equal to x;
$P_r(x)$ = proportion of trucks operating at GVW less than or equal to x under complete compliance condition;
x = operating GVW as a percentage of GVW limit (35 percent being the average tare weight as a percentage of the GVW limit for 3-S2);
$f(z)$ = violation rate (i.e., percentage of trucks inspected that are in violation) = f (inspection rate); and
$1 + f(z)$ = violation factor.

The relationship between level of enforcement and violation rate (VR) is described elsewhere (*7*). The VR is a reflection of the level of enforcement and depends on the method of enforcement (e.g., permanent weigh scale or mobile inspection teams). Since a given VR corresponds to different levels of enforcement for different methods of enforcement, VR is used as a proxy of the level of enforcement. It should also be noted that the definition of what constitutes violation varies among jurisdictions (e.g., whereas a charge laid against an operator may be considered a violation in one jurisdiction, only a successful prosecution of an operator is counted as a violation in the other).

In applying the models for different levels of enforcement, except for the complete compliance condition, a 20 percent maximum degree of overweight (amount by which weight limits are exceeded) is assumed. This accounts for (a) tolerances above the weight limit exercised by enforcement personnel, (b) extra loading from overweight trucks operating under special permits, and (c) wide variability in the degree of overweight as evidenced in available data.

PAVEMENT IMPACT ANALYSIS

This section illustrates the application of the model in pavement loading impact analysis of alternative GVW limits for the 3-S2 truck. In the following sections, parameters that can be used in comparing pavement loading of a given truck (e.g., 3-S2 truck) operated under alternative weight limits are discussed. The weight limits considered are the current U.S. federal weight limit with the grandfather clause and the Canadian interprovincial weight limit. It is assumed that pavement design, construction, and maintenance standards are identical in all cases.

Equivalent Pavement Loading

GVW distributions are first predicted under the two weight limits and converted into axle load distributions on the basis of the weight split characteristics on the axle units of this truck type. Equivalent standard axle loads (ESALs) are then calculated using the AASHTO load equivalency factors (*8*). Truck load factors (TLFs), or average ESALs per truck, are obtained as the weighted sum of the ESAL factors.

Flexible and rigid pavements are treated separately, but a terminal serviceability index, p_t, of 2.5 is used for each type. Flexible pavements with a structural number (SN) of 5 and rigid pavements with a slab thickness of 10 in. are used as representative structures. For each weight limit and pavement type, four levels of enforcement reflected in the VR are considered, namely 0 percent ("complete compliance"), 5 percent, 10 percent, and 15 percent. These values reflect typical VRs experienced at permanent weigh scales and by mobile inspection teams and are used to illustrate the effect of level of enforcement. Typical maximum VR for permanent weigh scales is in the region of 5 percent corresponding to an inspection rate of 3 percent or less; the corresponding value for mobile inspection teams is about 15 percent corresponding to an inspection rate of about 10 percent.

Truck Load Factor (Average ESAL per Truck)

Figure 1 indicates the percentage increase in TLF above the complete compliance situation as a function of the level of enforcement measured by the VR. The figure is derived from models that are demonstrated to be accurate in predicting average pavement loading at the 95 percent confidence level; therefore, these values are also deemed to have the same level of accuracy. The figure indicates that TLF generally increases with VR but at rates that depend on pavement type and the weight limit. For a given truck type, the consequences of nonenforcement of the regulations are more pronounced at higher weight limits. For example, under U.S. limits, a 1 percent increase in VR is accompanied by an approximately 2 percent increase in TLF on average for both flexible and rigid pavements. For the Canadian limit, which is about 18 percent higher than the U.S. limit, the corresponding increases are 2.7 percent for flexible pavements and 4.3 percent for rigid pavements.

Table 1 contains the relative changes in TLF at different levels of enforcement for two pavement types when the Canadian limit is compared to the U.S. federal limit. For the truck under consideration, 3-S2, TLFs under the Canadian limits are at least 32 percent greater than the U.S. equivalent, suggesting that load-associated pavement deterioration will be increased. The increase is likely to be minimized by exercising tight weight control strategies.

Figure 2 depicts the general relation between TLF, GVW limit, and level of enforcement for the 3-S2 truck. This relationship is developed from an ESAL calculated assuming the "fourth power" rule, with an exponent of 3.8 and no distinction between pavement types. The figure illustrates the effect of enforcement on the equivalent pavement loading for different GVW limits for the same truck.

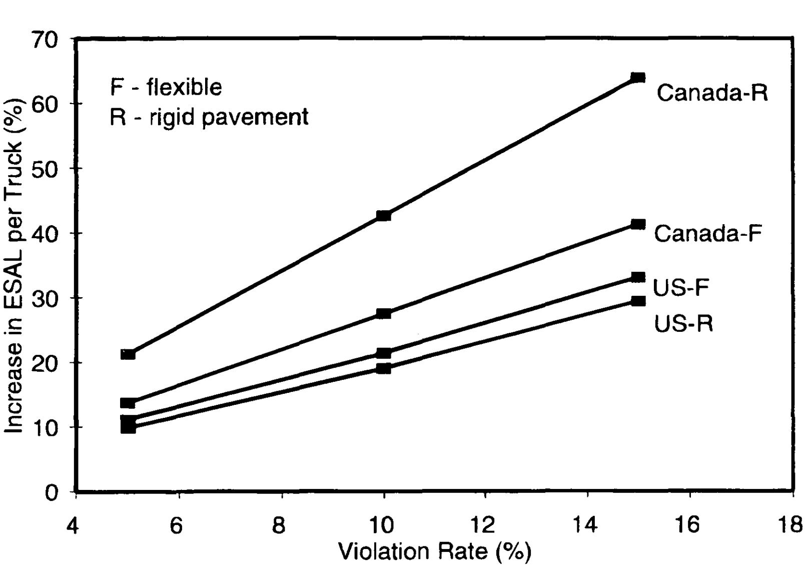

FIGURE 1 Enforcement effect on ESAL per truck (3-S2).

TABLE 1 Summary of Evaluation Parameters

Violation Rate (%)	Flexible Pavement[a]			Rigid Pavement[b]		
	US	Canada	Δ(%)[c]	US	Canada	Δ(%)[c]
			(a) Truck Load Factor			
0	1.102	1.495	+35.6	0.731	0.965	+32.1
5	1.256	1.700	+38.7	0.803	1.170	+45.7
10	1.338	1.905	+42.4	0.869	1.376	+58.2
15	1.466	2.111	+43.9	0.945	1.581	+67.3
			(b) ESAL per Payload			
0	0.081	0.098	+20.5	0.054	0.063	+17.4
5	0.087	0.107	+23.4	0.057	0.074	+29.6
10	0.091	0.115	+27.0	0.059	0.084	+41.1
15	0.096	0.124	+28.7	0.062	0.093	+49.6
			(c) ESAL-km			
0	153.7	131.0	-14.8	101.8	84.6	-17.0
5	170.9	149.0	-12.8	112.0	102.6	-8.4
10	186.5	167.0	-10.4	121.2	120.6	-0.5
15	204.5	185.0	-9.5	131.8	138.6	+5.2
			(d) ESAL-km per Payload			
0	11.33	8.58	-24.2	7.51	8.54	-26.2
5	12.10	9.39	-22.4	7.93	6.46	-18.5
10	12.72	10.16	-20.1	8.27	7.33	-11.3
15	13.45	10.89	-19.1	8.67	8.15	-5.9

[a] - SN = 5.0; ρ_t = 2.5. [b] - D = 10"; ρ_t = 2.5.
[c] - changes relative to the US equivalent.
GVW Limit (tons): US = 36.3; Canada = 39.5
VMT (billions, 1995): US = 139.42 km; RTAC = 87.66 km Source: TRB, 1990a.

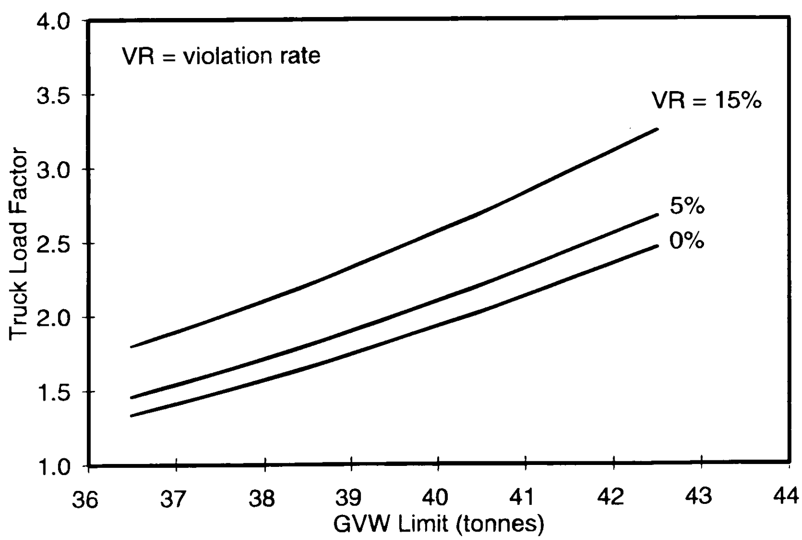

FIGURE 2 TLF-GVW limit relationship (3-S2).

ESAL per Payload

It is normal practice to estimate the damaging effect of a given vehicle on pavements from the ESAL. However, comparison of vehicles in terms of ESAL does not account for the fact that vehicles with higher weights require fewer trips to transport the same amount of freight, thereby offsetting part of the additional pavement wear caused by increased weight. To circumvent this problem, vehicles can be compared in terms of the ESALs per unit freight carried (*1*). Furthermore, in evaluating alternative weight limits for different vehicles, it is useful to consider how the relationship between the actual ESALs and the actual average payload associated with different weight limits changes as a function of the weight limit. It has been indicated that a unit change in the GVW limit is accompanied by a change in the ESAL per payload on the order of between 2.3 and 3, and that there is no optimum GVW limit at which the ESAL per payload is a minimum (*9*).

ESAL per payload under the two weight limits are compared in Table 1. Changes in the ESAL per payload at different levels of enforcement indicate that, generally, introducing a higher weight limit (e.g., Canadian limit) results in higher ESALs per payload—20–28 percent on flexible pavements and 17–49 percent on rigid pavements—compared to the U.S. limits. Again, the effect of level of enforcement is very noticeable, emphasizing its importance in evaluating the impacts of alternative weight limits.

ESAL-Kilometer

Even though ESAL per payload takes into account the amount of freight moved, it does not consider the number of repetitions of the loading on the pavement, that is, total loading. Changes in the weight limit are accompanied by changes in the total distance traveled per unit period for the same amount of freight. ESAL total distance traveled can be used as an indication of the total load repetitions imposed. It is noted, however, that highway cost allocation and road user charges or taxes are usually based on the ESAL-km moved. Relative changes in the ESAL-km are therefore studied using a base case forecast of 1995 vehicle miles traveled (VMT) for this truck under the two weight limits (*1*).

Table 1 also shows the percentage changes in ESAL-km. Introducing the Canadian weight limit reduces the ESAL-km of this truck operating on flexible pavements by 9 to 15 percent, relative to the U.S. limit, depending on the level of enforcement. On rigid pavements, the relative change varies from −17 percent at complete compliance, to +5.2 percent at 15 percent VR. The reduction decreases as the level of enforcement is relaxed. The results indicate that, for comparable levels of enforcement and same pavement type, the total pavement loading imposed by the 3-S2 truck, moving the same amount of payload operating under the Canadian limit, is less than under the U.S. limit. It is interesting to note that ESAL-km and ESAL per payload comparisons indicate opposing changes (i.e., the equivalent pavement loading per unit freight moved by this truck type will be substantially increased but the total loading over the given time period will be reduced).

ESAL-km per Payload

Considering that total imposed loading (magnitude and frequency) is determined by the quantity of freight, it is worthwhile to examine how the ESAL-km per payload varies under the alternative weight limits. This parameter relaxes the constant freight condition and allows comparison of the total loading per unit weight of freight moved by each truck type under the alternative weight limits.

Table 1 shows that substantial reductions in ESAL-km per unit payload may result when this truck is operated under the Canadian limit compared to the U.S. limit. The reduction is between 19 percent and 24 percent in ESAL-km per payload on flexible pavements and 6 percent to 26 percent on rigid pavements, depending on the level of enforcement. These results indicate that total pavement loading per unit payload moved under the Canadian limit is less than under the U.S. weight limit for a given level of enforcement. To realize the benefits of reduced ESAL-km per payload indicated by adopting a higher weight limit, it is imperative, therefore, to exercise tighter weight controls on truck operations. This is more critical for rigid pavements than for flexible pavements. However, it should be noted that these comparisons assume that the pavements are designed, constructed, and maintained to identical standards.

Discussion

In general, for the levels of enforcement considered, the range of variation of the relative changes in the parameters examined for rigid pavements is about four times that for flexible pavements. The values also indicate that load-associated damaging potential for rigid pavements is more sensitive to the level of enforcement than for flexible pavements. The analyses demonstrate the scope of the models and, in particular, highlight the importance of enforcement in the evaluating alternative pavement loading scenarios.

These comparisons are based purely on pavement loading. The cost of enforcement, cost of upgrading the existing infrastructure to withstand the increased loading (TLF) resulting from a higher weight limit, the maintenance and rehabilitation costs associated with the higher loading per unit payload, and so forth, need to be considered in the total evaluation process. It is worthwhile to note that in situations where pavement deterioration is attributed more to environmental effects than to traffic loading, these parameters may not be very useful from the pavement performance standpoint. However, these parameters may be of value in highway cost allocation and taxation mechanisms since they are based primarily on the pavement loading.

It is observed that the ESAL-km per payload is a more objective and flexible parameter because it is not constrained by the fixed amount of payload under alternative scenarios and takes into account the total amount of pavement loading.

PAVEMENT COST

From the standpoint of highway cost allocation, it is relevant to express the enforcement factor quantitatively in the pavement loading analysis. A study in Canada (*10*) suggested that environmental factors account for most pavement deterioration in Canada. From the perspective of highway cost allocation, this implies that most pavement costs can be treated as a common cost (i.e., costs that cannot be traced to one user—truckers—versus another). There is, however, a broad range of costs attached to ESAL-km, depending on pavement type, truck type, and costing mechanism. To illustrate the pavement cost implications of alternative weight limits and the enforcement levels, estimates by Rilett et al. (*10*) representing typical conditions in Ontario, Canada, are used, that is, 0.6 cents per ESAL-km (high-volume highway) and 2.2 cents per ESAL-km (low-volume high-

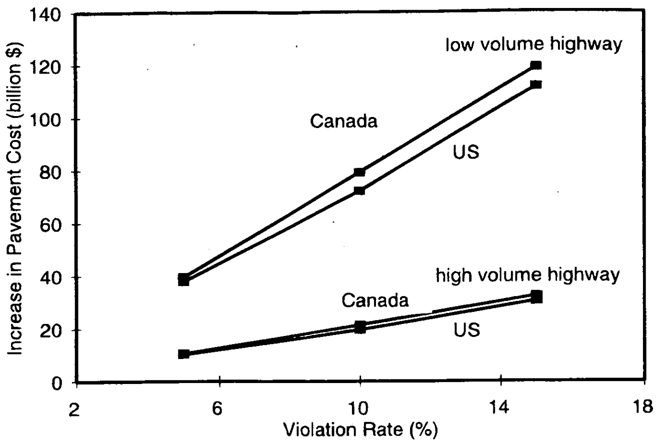

FIGURE 3 Enforcement effect on pavement cost (3-S2).

way). These are pavement costs arising from axle loads only (i.e., pavement costs due to environmental factors are excluded).

Figure 3 illustrates the incremental changes in the pavement cost as a function of the level of enforcement and highway usage. It indicates how the effect of enforcement reflected in the pavement loading translates into pavement costs. It is noted that the values for the higher Canadian weight limit are marginally higher than those for the U.S. limit for the same highway usage and level of enforcement. This suggests that for a given highway type, pavement cost is more dependent on the level of enforcement than weight limit. In other words, pavement costs can be minimized by adopting strict weight enforcement measures. Large differences between the rates of increase on low- and high-volume highways may be partly attributed to the assumption that the VMT on both highway types are the same. This may not necessarily be the case in reality.

CONCLUSION

Parameters for evaluating pavement loading of alternative GVW limits and their enforcement are presented. It is observed that enforcement is a critical factor in pavement loading analysis of alternative weight limits. Equivalent pavement loading on flexible and rigid pavements respond to enforcement levels differently, with rigid pavements being more sensitive to the enforcement level. Consideration of the payload and/or distance traveled together with the ESAL per truck under alternative weight limits provides a more objective assessment of pavement loading impacts than the truck load factor alone. In terms of pavement costs resulting solely from pavement loading, the order of magnitude of savings resulting from implementing tight enforcement schedules is attractive.

REFERENCES

1. *Special Report 225: Truck Weight Limits: Issues and Options.* TRB, National Research Council, Washington, D.C., 1990.
2. *Special Report 227: New Trucks for Greater Productivity and Less Road Wear: An Evaluation of the Turner Proposal.* TRB, National Research Council, Washington, D.C., 1990.
3. Fekpe, E. S. K. *Characteristics of Heavy Vehicle Weight Distributions and Pavement Loading.* Ph.D. dissertation. University of Manitoba, Winnipeg, Canada, 1993.
4. Fekpe, E. S. K., and A. Clayton. Predicting Heavy Vehicle Weight Distributions. *ASCE Transportation Engineering Journal*, Vol. 121, No. 2, March–April 1995, pp. 158–168.
5. Fekpe, E. S. K., and A. Clayton. A Rational Approach to Predicting Pavement Loading. *Proc. Annual Conference of Canadian Society for Civil Engineering*, Fredericton, New Brunswick, Canada, Vol. 3, 1993, pp. 597–606.
6. *SAS User's Guide*, Statistics Version 5 Edition. SAS Institute Inc., Cary, N.C., 1985.
7. Fekpe, E. S. K., and A. Clayton. Quantitative Assessment of Effect of Enforcement Intensity on Violations of Vehicle Weight and Dimension Regulations. *Transportation Planning and Technology Journal*, Vol. 18, No. 2, 1994, pp. 143–153.
8. *ASSHTO Guide for Design of Pavement Structures.* Washington, D.C., 1993.
9. Fekpe, E. S. K., and A. Clayton. Model for Trucking Productivity Analysis of Alternative Weight Limits. *Proc. IRF Regional Meeting*, Calgary, Alberta, Canada, Vol. 5, 1994, pp. C15–25.
10. Rilett, R., B. G., Hutchinson, and R. C. G. Haas. Cost Allocation Implications of Flexible Pavement Deterioration. Presented at 68th Annual Meeting of the Transportation Research Board, Washington, D.C., 1989.

Role of Spatial Dimension in Infrastructure Condition Assessment and Deterioration Modeling

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The treatment of the spatial dimension in assessing infrastructure condition, modeling its deterioration, and, consequently, maintenance decision making are achieved through the development of a spatial distress model that provides the necessary structure for such a treatment. The relationship of the spatial model developed to the temporal deterioration modeling literature is presented. The spatial model developed recognizes two deterioration mechanisms: the *environmental* mechanism, which describes deterioration as a consequence of causal factors and exhibits both macroscopic and microscopic scales, and the *interactive* mechanism, which describes deterioration as a result of distress at a location influencing the deterioration of neighboring locations and exhibits a microscopic scale. The results of the application of the model for the identification of uniformly behaving regions appropriate for condition assessment emphasize the importance of the explicit recognition of the spatial dimension within the infrastructure management process.

Information on facility condition is essential to infrastructure management. Infrastructure facilities are spatially extensive in nature. This paper is concerned with the treatment of the spatial dimension in assessing infrastructure condition, modeling its deterioration, and, consequently, facilitating maintenance decision making.

The Infrastructure management process can be characterized by the following three components (*1*):

- Data collection and analysis;
- Condition assessment and forecasting;
- Strategy selection for inspection and maintenance.

The first component involves gathering and analyzing the relevant data for decision making. This includes data on use, such as current traffic volume and mix, and data on the surrounding environment, such as soil condition, temperature fluctuations, and precipitation. Both use and the surrounding environment play a major role in the degradation process. Distress data representing this degradation are also important to collect. They include information on the location and magnitude of the different distress types exhibited. The most common types of distress for which data are collected are those appearing on the surface of facilities.

The second component of the management process entails assessing the current condition and forecasting the future condition using the data collected in the context of the first component along with deterioration models. Many studies have been conducted in the area of deterioration modeling—see Ramaswamy (*2*).

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Both the current assessment and the prediction of condition provide the necessary inputs to the third component of the management process, namely, selecting the strategy for inspection and maintenance over time. The function of this component is to select maintenance activities and inspection strategies that minimize total user and maintenance costs. Many studies have been conducted in the area of strategy selection for infrastructure management—see Madanat (*3*).

As the presentation above indicates, the literature has focused predominantly on the temporal aspects of the management process. The importance of the temporal dimension should not be underestimated. However, there is a clear lack of explicit treatment of the spatial dimension. This paper defines the role of the spatial dimension within the management process and presents a new understanding of the spatial behavior of infrastructure distress. A methodology for identifying this behavior is, in turn, developed. Finally, the importance of the new spatial understanding in achieving effective maintenance decisions is indicated through an application.

ROLE OF SPATIAL DATA WITHIN THE MANAGEMENT PROCESS

The spatial dimension plays a role in the following functions (relating to all three components of the infrastructure management process):

- Condition assessment and temporal behavior modeling;
- Inspection strategy selection with regard to spatial coverage; and
- Identification of systematic type measurement errors resulting from unexpected exogenous factors.

Temporal deterioration at each location may be strongly related to the deterioration of its neighboring locations. Hence, measures of condition are, in general, associated with a specific region. Typical measures include percentage of the area cracked and average variation in deformation per unit length. Such measures can only be quantified in the context of well-defined regions. Therefore, to use the available data for infrastructure management purposes, the behavior of distress over space must be understood. This understanding is also necessary for condition assessment and temporal behavior modeling. The critical consideration is to quantify condition based on regions that will behave uniformly over time. Otherwise, the prediction of the behavior of such stand-alone entities ceases to have any meaning. Therefore, the purpose of modeling behavior over space is to provide the necessary structure based on which current condition can be quantified and deterioration modeled.

The role of the spatial dimension is also relevant at the strategy selection level. Because inspection is conducted over the length of the facility, it is necessary to specify not only *when* to inspect but also *where*. Understanding the spatial behavior allows such spatial decisions. Finally, the role of the spatial dimension is also significant for the data collection and analysis component. This role relates to measurement errors. Understanding the spatial behavior can potentially enhance the identification and quantification of systematic type errors resulting from unexpected exogenous factors.

LITERATURE REVIEW ON SPATIAL AGGREGATION

The literature on the partitioning of infrastructure space into regions within which distress data can be aggregated for condition assessment is very limited. The Agency Method is based on the argument that since deterioration is a function of causal variables, space is partitioned into the largest possible regions such that each of the causal variables are "constant" within each region (*4*). Common causal variables include

- Daily traffic volume and mix (percentage of trucks);
- Structural design;
- Quality of construction;
- Environmental factors;
- Maintenance history.

The Agency Method identifies the locations of "change" in any of the causal variables. These locations are the boundaries of the regions used for condition assessment. Each causal variable is assumed to have a "constant" magnitude within each region. In reality, however, none of the causal variables have a *constant* magnitude even within short lengths along the facility. Therefore, given that many of the important variables are continuously changing along the facility, the critical requirement for the method to produce good results is the appropriate definition of "change" and, consequently, its detection. Unfortunately, the data on causal variables are unavailable at the desirable level of detail primarily because of the difficulties in collecting such data despite recent advances in remote sensing technologies (*5*). Therefore, agencies rely on historic data, which are spatially aggregate. In the very nature of such aggregate causal data is the presence of locations of change. Therefore, the definition of regions along the facility reduces to superpositioning the locations of change in the aggregate data.

Furthermore, the aggregate historic data on causal variables are unreliable due to the lack of knowledge about the original circumstances surrounding their construction and maintenance. The variation of the causal variables within regions thought to be uniform, therefore, could be significant. The thickness of the different pavement layers, for example, have been found to vary significantly compared with data obtained from original design plans and maintenance records (*6*). According to AASHTO (*4*), one of the most difficult variables to assess with regard to "changes" along the facility is the quality of the subgrade or foundation as characterized by the soil type.

In response to the uncertainty in the use of causal variables, AASHTO has proposed a method for delineating "homogenous" regions using either distress measurements or condition indexes (*4*). The method is based on a response function that represents either a measure of distress or the value of a distress index (which is a composite of measures of several distress types) along the highway. It is assumed that the function is piecewise constant. The locations where the response function intersects its mean are adopted as the boundaries of uniform regions. However, it is not necessary that the mean intersects the response function at *all* locations of abrupt change. Only when there is a single abrupt change, does this method guarantee the desired solution. Moreover, as AASHTO has indicated (*4*), the response function is not piecewise constant in nature. Therefore, under realistic situations of high small-scale variability, the method has the potential to identify many unnecessary small regions, resulting in misleading condition assessments. For a more detailed discussion on this method see Mishalani (*7*).

Another study that addressed the spatial dimension relates to the determination of maintenance activity regions that take into account maintenance implementation constraints (*8*). Such considerations require as input uniformly behaving regions.

METHODOLOGY

In this section, pertinent deterioration models are first reviewed. Based on the review, a spatial distress model is developed. Subsequently, a methodology for identifying regions of uniform behavior using the developed spatial model is presented.

Mechanisms of Deterioration

Once an infrastructure facility is constructed, it is subjected to both traffic loading and weathering. As a result, the facility undergoes a natural degradation over time. In addition to these exogenous factors, the durability of the facility (as captured by the design of the facility and the quality of its construction) and the material on which it is founded (as captured by soil conditions) play a significant role in its behavior over time. All these factors influencing the deterioration process are referred to as environmental variables in this paper. The variables associated with the design, construction, and original soil properties are static in nature, while traffic and weather vary over time with a cumulative effect on the facility. Given that facilities extend through a vast amount of space, one expects *both* the static and dynamic exogenous variables discussed above to *vary* over space. Hence, understanding the mechanisms of deterioration from a spatial perspective is fundamentally important in understanding the temporal behavior of a facility.

The focus of most studies on the mechanisms of deterioration is the temporal dimension. Despite time and space being closely related, very little effort has been directed toward investigating the spatial dimension. Therefore, the following review of the deterioration literature focuses primarily on exploring the spatial knowledge implicitly or explicitly assumed in temporal modeling.

Theoretical models (as opposed to empirical) capture the effects of traffic loading and weathering on the mechanical characteristics of the material of which the facility consists and, consequently, determine the distress that would result from that effect. Moavenzadeh and Brademeyer (*9*) and Markow and Brademeyer (*10*) developed model systems that use basic mechanical principles to explain the behavior of infrastructure facilities over time.

From a spatial perspective, Moavenzadeh and Brademeyer (*9*) explain surface deformation as a consequence of *microscopic* (or small-scale) variation in the material properties over space. The use of a spatial correlation function is proposed for capturing such variation. This is an important component of the overall spatial variation of interest in this study and, therefore, should be taken into account in building the spatial model that describes the behavior of distress along extensive facilities.

Markow and Brademeyer (*10*) adopt a dynamic structure where the change in condition is modeled. It is assumed that the condition of a facility at time $t + \Delta t$, denoted by $C(t + \Delta t)$, is a function of both the condition at time t, $C(t)$, and explanatory variables such as design and traffic. Measures of condition are, in general, associated with a specific region. Therefore, the dynamic nature of the model assumes that the condition of a region at a particular point in time is a fundamental determinant of the condition of that same region at a subsequent point in time. This implies that the space within the region considered evolves in the same manner over time. This is expected to be the case when the magnitude of the causal variables are "fairly" constant within the region.

Due to the complexity of the mechanisms involved and the high degree of variability in the factors affecting the deterioration process, it is difficult to develop a realistic mechanistic model that accurately explains the behavior of distress over time. Nevertheless, such models provide the necessary foundations for building empirical models. Empirical models have proven to be more successful and are widely used by facility management agencies. They are founded on direct observations of surface distress, and correlations of such observations with the explanatory variables and maintenance actions of interest.

Paterson (*11*) adopted such an approach. Condition is represented in a disaggregate manner by a vector whose elements are associated with the different distress types. The behavior of each element is modeled separately over time. The dynamic dimension is microscopic compared to Markow and Brademeyer's model. For the distress types that occur discretely in space—such as cracking, raveling, and potholes—the temporal model is characterized by two distinct phases: initiation and progression. The initiation model predicts the failure time, which is the time at which the first distress appears. The progression model is conditional on initiation having taken place and measures the change in condition from one point in time to the next.

The representation of the dynamic dimension at this microscopic level implicitly assumes that the distress at a location influences the deterioration of neighboring locations. Therefore, the mechanism of deterioration captured by these models relates to distress *interaction* in space. Once a crack occurs in a particular region, the temporal behavior *switches* from one of initiation to one of progression, implying that the *first* crack induces initiation and progression of other cracks in that region. Such regions are referred to as sections, and are defined as "nominally homogeneous." It is suggested that a "convenient" section has the width of a lane and a length of 320 m. The use of the width of the lane is appropriate due to the general confinement of traffic to lanes. The length of the section should capture the spatial extent within which distress interaction takes place. Although the scale associated with such interaction is expected to be *microscopic*, the section length of 320 m is not quantified as such but rather chosen based on engineering judgment.

In summary, three types of spatial behaviors are implicitly or explicitly assumed by the deterioration models examined above:

- Macroscopic environmental behavior;
- Microscopic environmental behavior; and
- Microscopic interactive behavior.

Spatial Model

The process that results in the occurrence of surface distress can be broken down into two mechanistic processes that occur simultaneously. The environmental process and the interactive process. The *environmental* process describes deterioration as a consequence of a multitude of environmental factors: subsurface conditions (such as soil conditions, design standards, and construction quality); and external conditions (such as traffic, weather, and drainage). These factors are not uniform over space, and different combinations will result in different deterioration propensities. The realized distress will mirror the environmental variability. For example, regions of both poor soil condition and high traffic volumes have a much higher likelihood of exhibiting high distress levels than regions of good soil conditions and low traffic volumes. It is this process that motivates the use of causal variables by agencies managing infrastructure facilities. In relation to the model developed by Paterson, it is the environmental factors that influence the time at which initiation takes place.

The *interactive* process describes deterioration as a result of distress at a location influencing the deterioration of neighboring locations. For example, the likelihood that additional cracks will initiate near other cracks is greater than of their doing so in a region exhibiting no cracks. Moreover, the likelihood that two neighboring cracks will propagate and connect is very strong. This phenomenon is known as crack coalescence in the material science literature (*12*). In terms of Paterson's initiation and progression model, it is both the interactive process and the environmental process that are responsible for the progression stage.

The environmental process exhibits a spatially extensive scale where the level of distress is "fairly" constant for long stretches along facilities. This is a direct consequence of the nature of the underlying causal variables. The variables are not expected to vary substantially within relatively long stretches and when a change occurs, it is expected to be in the form of an abrupt shift. For example, traffic volumes are expected to be "fairly" constant. Changes in volume occur at exits and entrances; therefore, any significant change in volume will most likely be abrupt in nature. Since design standards are usually based on traffic volumes, a similar pattern in the design is expected. Moreover, since construction quality is a function of the source of materials and the contractor, this variable will also be constant for relatively long stretches, and any changes will most likely be abrupt. Therefore, the scale of the environmental process is expected to be macroscopic with an order of magnitude of several kilometers. In terms of the model developed by Markow and Brademeyer, the uniformity of the deterioration of a region over time is consistent with this environmental process.

Although the causal variables in most cases remain relatively constant for some "long" stretches, they still exhibit *small-scale variations* as captured by the model developed by Moavenzadeh and Brademeyer. Moreover, the interactive process is expected to exhibit a spatially "local" scale compared to the macroscopic scale associated with the environmental process. Since interaction takes place primarily as a result of the weakness distress induces on its surroundings, from a mechanistic perspective the interactive process is governed by the environmental mechanism in the sense that the environment affects the magnitude of the weakness a distress induces and, consequently, the strength of the interaction. For example, in situations of high design standards, the strength of the interaction between two cracks is lower than in the case of poor design standards. Therefore, the interactive process contributes to the variability within the regions of "fairly" constant environmental variables.

The hypothesis that emerges from this discussion is a spatial pattern with regions of well-defined boundaries. [Within each region, distress fluctuates around a region-specific constant level.] Such regions are referred to as *fields* in this study. The developed spatial hypothesis is represented by a stochastic process. For the sake of simplicity, the case of a single distress type is presented. Let X_s be the measure of distress at location s of the facility. The magnitude of the distress at location s consists of two components:

- A systematic component, constant within each field, which captures the macroscopic behavior; and
- A stochastic component, with zero mean, which captures the inherent variability of the microscopic behavior resulting from both the interactive process and the small-scale environmental variations.

The stochastic component also captures the random nature of measurement errors. It is assumed that the systematic nature of measurement errors (i.e., the bias) has been corrected—see Humplick (13).

Mathematically, this model is represented as follows:

$$X_s = \mu_s + \varepsilon_s \text{ and } \mu_s = \begin{cases} \mu_1 & \text{if } s \in F_1 \\ \bullet & \bullet \\ \mu_m & \text{if } s \in F_m \\ \bullet & \bullet \\ \mu_M & \text{if } s \in F_M \end{cases} \tag{1}$$

where

X_s = distress at location s;
s = locations along the facility;
m = index representing the fields ($m = 1, \ldots, M$);
F_m = set of locations contained within field m;
μ_s = systematic mean distress;
μ_m = systematic mean distress within field m; and
ε_s = random variable (of zero mean) representing the deviation of the actual observed distress from the mean μ_s.

Since the deterioration at the microscopic level occurs within the context of the systematic deterioration at the macroscopic level, the fields are expected to behave uniformly over time (with respect to both the mean distress level and its rate of change). The microscopic environmental process and the interactive process will result in spatial variation in distress within each field. That is, since the underlying dominant force governing deterioration is the environmental process, such local variations will take place conditional on the environment within which they occur. Since the local variation within such fields is a variation around a constant level of deterioration, any aggregation within a region fully contained within such a field is an estimate of the constant level of deterioration. Therefore, the larger the region, the better the estimate (i.e., the lower the variance of the estimate) as long as the region is contained within the field. On the other hand, if the region size exceeds the size of the field within which it was originally defined, more than one level of deterioration will be introduced within the same region, and, therefore, the estimate would not be an accurate representation of either level (i.e., the estimate is biased). Consequently, this results in an erroneous condition assessment.

Therefore, the best aggregation scheme is a configuration in which each macroscopic field is a region. The problem at this point lies in the lack of knowledge on the locations of the boundaries defining the fields of interest. Moreover, since the environmental variables at the level of detail of interest are either unavailable or unreliable from a measurement point of view, the most useful indicators of the boundaries defining the fields are the distress measures over space. From a methodological point of view, the problem reduces to the identification of the locations along the facility where the *mean* of the stochastic process undergoes an abrupt change.

Field Identification

Due to the insignificance of the lane width with respect to the longitudinal scale of interest, the two-dimensional nature of a highway lane is approximated as one-dimensional. Furthermore, since, in general, the longitudinal extent of discrete distress types such as cracking is small [averaging between 2 and 10 m in the cases examined—see Mishalani (*7*)] with respect to the scale of interest, for purposes of this analysis distress types are characterized by:

- Point events in continuous one-dimensional space (cracking, potholes); and
- Continuous variables in continuous one-dimensional space (rutting).

The point representation naturally lends itself to an analysis using spatial point processes. Point processes are a type of stochastic processes in which the events of interest are points occurring randomly in continuous space. Using point processes provides a convenient means for representing point data by a distress intensity function in space. The intensity function, $x(s)$, is the number of events per unit length for each location in continuous space and, therefore, represents the propensity of a particular location to exhibit a point event. Using the original point events, $x(s)$ is estimated using a nonparametric kernel-based estimator (*14*) that minimizes the mean square error. The estimator is given by:

$$\hat{x}(s) = \sum_{j=1}^{C} \frac{1}{w}\,\delta\!\left(\frac{S - S_j}{w}\right) \tag{2}$$

where

$\delta(\cdot)$ = kernel function that specifies the relative strength by which the existence of an event at s_j contributes to the estimate of the intensity at s;
s_j = location of event j;
C = total number of events; and
w = a parameter representing half the window width within which events contribute to the estimation of $x(s)$.

The statistical properties of the estimator $\hat{x}(s)$ are dependent mostly on the parameter w. If w is large, more events are used in the estimation but the role of each as an indicator is less significant. On the other hand, if w is small, the more significant events are used but there are fewer of them. Therefore, the effect of w on the estimates relates to efficiency and bias. Large values of w are associated with both high efficiency (i.e., low variance) and large bias, whereas small values of w are associated with low efficiency and small bias.

The functional form of the kernel is not important from a statistical point of view. Hence, for mathematical convenience, the uniform kernel is used:

$$\delta(y) = \begin{cases} 1/2 & \text{for } |y| \le 1 \\ 0 & \text{otherwise} \end{cases} \tag{3}$$

Using the function above, an optimal parameter w^*—which minimizes the mean square error of the estimator $\hat{x}(s)$—can be determined. The determination of the optimal window width, $2w^*$, also allows for conveniently representing rutting over space. See Mishalani (*7*) and Mishalani and Koutsopoulos (*15*) for more detail on representing distress over space.

Once distress functions (crack and rut intensity functions) are quantified, the identification of the boundaries defining the fields can proceed. The probability density function of X_s is unknown. Therefore, a nonparametric solution approach for the detection of the boundaries is adopted. The field identification problem is formulated within a cluster analytic framework with the additional constraint that all observations belonging to the same cluster should be spatially contiguous. The boundaries of the clusters represent locations where the systematic mean function, μ_s, undergoes an abrupt shift in value.

Formulating the optimal clustering problem in a manner where the number of fields and their boundaries are determined simultaneously is mathematically intractable. Therefore, the following heuristic is proposed:

Step 1: Set the number of fields, M, to 2.

Step 2: Determine the locations of the optimal boundaries of the M fields.

Step 3: Examine the stopping criterion (this step is applicable for $M \ge 3$). If the criterion is satisfied, the optimal solution with $(M - 2)$ fields is the final solution. Otherwise, set M to $M + 1$ and go to Step 2.

The main idea behind this iterative approach is to reduce the complexity of the problem by making a tentative assumption on the number of fields. This allows for an optimal determination of the location of the boundaries. This process is repeated in an iterative manner until the stopping criterion is satisfied. The purpose of the stopping criterion is to determine which of the various sets of solutions best captures the spatial behavior.

The decision variables of the optimization problem of Step 2 are the locations of the boundaries. The objective is to minimize the *total within cluster variation*. Therefore, the boundaries will optimize a measure of similarity within the fields (i.e., observations within each cluster are as close as possible to their cluster mean). The additive structure of the objective function allows for the use of a dynamic programming algorithm (*16*) that guarantees a globally optimal solution.

The stopping criterion should indicate the number of fields that best characterize the spatial behavior. It consists of examining the incremental contribution of every additional boundary to the reduction in the overall variation throughout the facility. Let $Z^*(M)$ be the value of the objective function when all the observations over space are optimally partitioned into M clusters. $Z^*(M)$ is monotonically decreasing with M. $Z^*(1)$ captures the total variation of the intensity function with respect to the overall mean. The incremental contribution of the Mth additional field to explaining the total variation (or equivalently to the reduction in the objective function) is measured by the following ratio:

$$r(M) = \frac{Z^*(M-1) - Z^*(M)}{Z^*(1)} \tag{4}$$

The stopping criterion is satisfied when the change in the incremental contribution becomes insignificant.

APPLICATION AND EMPIRICAL FINDINGS

The developed methodology has been applied to three independent flexible pavement highway facilities, each of which is approximately 15 km long. All three applications reveal similar results. One of the three applications is presented in this paper. The facility is an undivided two-lane (one lane in each direction) highway in Mississippi. Its Federal Functional Classification is a rural major collector. It was constructed between 1947 and 1948 and was resurfaced once in 1984–1985. The current average daily traffic is 1,040 vehicles, 10 percent of which is truck traffic. The original design has a total pavement thickness of 17.8 cm to 43.2 cm. The resurfacing resulted in an additional 2.5 cm. Detailed surface distress data were collected by PaveTech (Oklahoma) on one of the two lanes during the summer of 1991 using a van-mounted state-of-the-art video camera. The video images were interpreted by experts but the distress measurements, such as crack lengths and areas, were quantified automatically using image-processing techniques.

The mean length (in the longitudinal direction) of the cracks is 2.53 m. Moreover, almost 99 percent of the cracks have lengths less than or equal to 30 m. Therefore, in the context of the expected macroscopic nature of the environmental process (where fields are expected to be several kilometers long), the assumed point representation of cracks is, in general, realistic. The optimal window width, $2w^*$, is 50 m. In this particular application, the crack intensity function reveals all the fields. Therefore, results focusing on cracking are presented. See Mishalani and Koutsopoulos (*15*) for a presentation of the results relating to rutting.

Figure 1 depicts the contribution ratio as a function of the number of fields M. It is clear that for $M \ge 7$, the incremental contribution ratio is consistently low suggesting that all fields beyond the first six overfit the spatial model. Therefore, there are six significant fields that explain the spatial behavior. This conclusion is further confirmed by the examination of the location of the boundaries associated with the six fields. The optimal boundaries along with the crack intensity function are indicated in Figure 2. The horizontal lines below the plot indicate the solutions where a vertical bar represents the location of a field boundary. Two solutions are given: the optimal solution and the solution of the Agency Method. Notice that the optimal boundaries (first line in Figure 2) reveal the piecewise constant nature of the mean of the crack intensity function. With six fields, the average field length is 2.5 km. This is consistent with the engineering expectation that the macroscopic environmental process (which is captured by the fields) exhibits such a scale.

Another important conclusion that can be drawn relates to the spatial extent of the interactive mechanism along with the microscopic environmental mechanism. This is achieved by examining the spatial correlation structure within each field m. The spatial correlation function is defined by $\rho_{mh} = \mathrm{Cov}[X_{ms}, X_{m(s+h)}]/\mathrm{Var}[X_{ms}]$. The sample estimate of the correlation function, $\hat{\rho}_{mh}$, indicates the structure of the linear dependence exhibited by the data. The positive spatial correlation exhibited within each field confirms the presence of the microscopic interactive and environmental mechanisms. The extent of this correlation is captured by the largest separation distance at which the spatial correlation within each field significantly

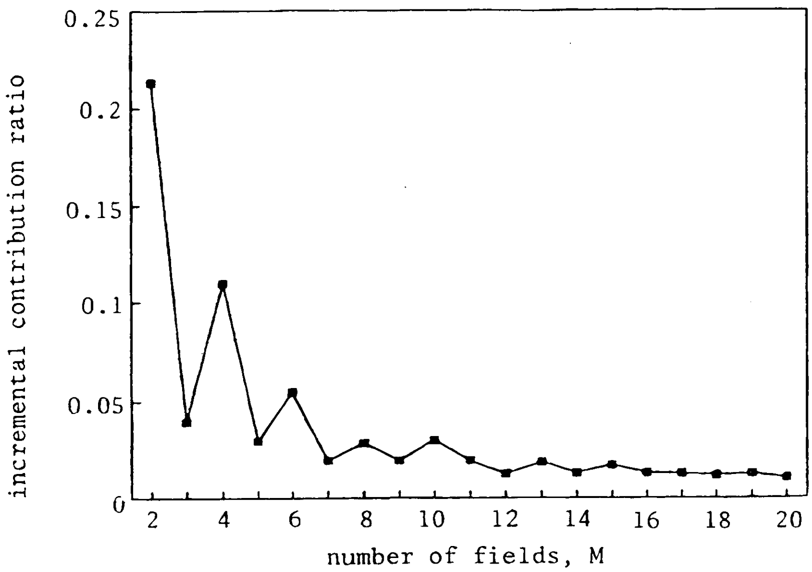

FIGURE 1 Incremental contribution ratio function.

differs from zero. That distance for all the fields of all three facilities examined varies between 20 m and 180 m considering both cracking and rutting. Figure 3 presents the estimate within Field 6 (i.e., the region from 11.53 km to 14.48 km). Notice that the correlation becomes insignificant at a separation greater than 50 m. Recall the range of interaction assumed by Paterson (*11*) at 320 m based on engineering judgment. The range of interaction identified based on this empirical study is of the same order of magnitude. However, the empirical analysis reveals a high degree of variability in the exhibited ranges and, in general, smaller magnitudes.

Having demonstrated empirically the validity of both the developed spatial model and the corresponding field identification methodology, it is worth examining the performance of the state-of-the-art methods in light of the findings of this study. Figure 2 indi-

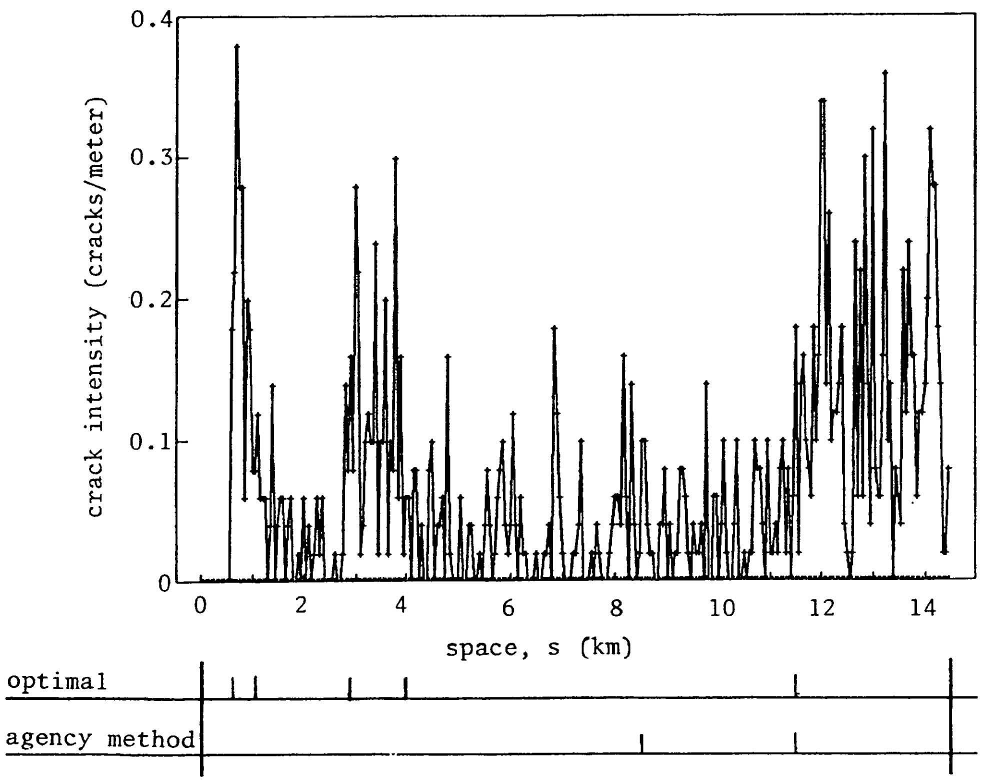

FIGURE 2 Field boundary solutions.

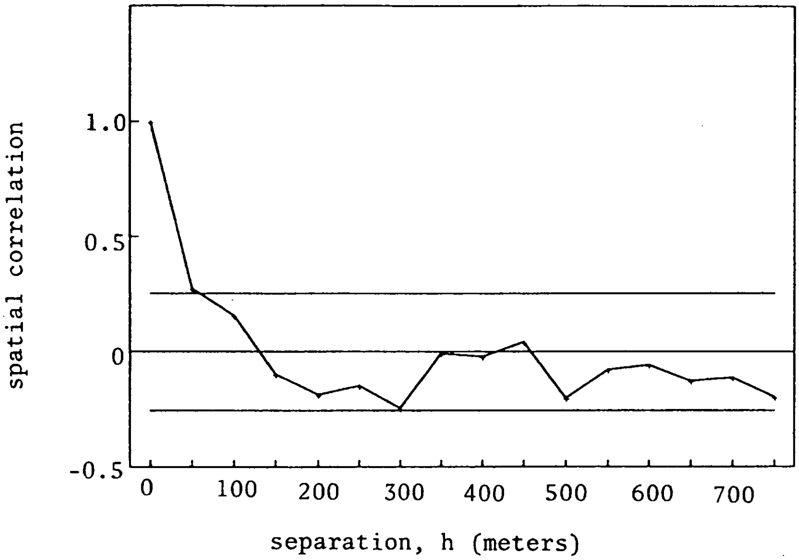

FIGURE 3 Estimate of Spatial Correlation Function Within Field 6 and 95 percent confidence region.

cates the solution of the Agency Method (on the second horizontal line below the plot). The boundary at 8.43 km is due to differences in the original design and the boundary at 11.79 km is due to a 1-year difference in the timing of the resurfacing activity. The Agency Method was able to identify only one of the boundaries detected by the developed field identification methodology. In addition, however, the Agency Method has identified a boundary at 8.43 km that the field identification methodology did not capture. One potential explanation follows. The resurfacing that was applied only a year before the data were collected is concealing the change in the distress manifestation that one expects to see. Even though the boundary was not evident at the time the distress data were collected, once further deterioration is allowed to take place the boundary is expected to become apparent.

Since one of the fundamental objectives behind this study is to provide the appropriate inputs for condition assessment, deterioration modeling, and subsequently decision making, one of the more important such inputs is computed using both the optimal fields and the Agency Method regions. The percentage area cracked is indicated in Table 1 using both sets of boundaries. The percentage difference between the Agency Method estimates and the optimal estimates (using the fields as basis) is also shown. One can see that the Agency Method appreciably underestimates the percentage area cracked in the region of Fields 2 and 4 and appreciably overestimates it in Fields 1 and 5. The Agency Method regions result in a contamination of the observations from one field with observations from other fields of different mean levels. The Agency Method overestimates or underestimates the percentage area cracked by at least 20 percent in regions comprising 65 percent of the total length. Such differences could potentially result in not maintaining regions that do need maintenance, thus resulting in a rapid deterioration that would require more costly maintenance in the future. It could also result in overspending on unnecessary maintenance work. In either case, the life cycle costs associated with the facilities would be suboptimal.

As for the AASHTO method, it is evident that under the highly stochastic nature of the spatial distress process revealed in this paper, it would result in too many small regions that would substantially over- or underestimate the mean distress level. Moreover, assuming for the sake of this argument that the small-scale vari-

TABLE 1 Comparing Percentage Area Cracked of Optimal Fields with Those of Agency Method

Optimal field number	1	2	3	4	5		6
From (km)	0	0.625	1.025	2.825	3.975		11.525
To (km)	0.625	1.025	2.825	3.975	11.525		14.48
% area cracking	0.69	26.02	5.57	7.94	2.83		11.83
% difference[a]	581.16	-81.94	-15.62	-40.81	66.08	34.63	0
Agency region	1					2	3
From (km)	0					8.43	11.525
To (km)	8.43					11.525	14.48
% area cracking	4.70					3.81	11.83

[a]Using the field measures as a reference.

ability is insignificant with respect to the piecewise constant mean function (which is definitely not the case in reality), as already discussed the AASHTO method does not guarantee the identification of all the boundaries of abrupt change. Hence, the usefulness of the AASHTO method is limited.

CONCLUSION

In this study, a model that describes the spatial behavior of infrastructure distress is established. The model captures both the macroscopic and microscopic scales of behavior. The macroscopic scale is associated with the environmental deterioration mechanism, and the microscopic scale is associated with both the environmental and interactive deterioration mechanisms. Based on the spatial distress model, a methodology that identifies the uniformly behaving spatial fields is developed. The spatial model developed is validated using detailed data along 15-km-long facilities (one of which is presented in this paper). Such an empirical analysis is the first of its nature in the context of infrastructure research.

The spatial model developed plays an important role not only in condition assessment and deterioration modeling but also in addressing a host of other issues within the infrastructure management process. For example, the past decade has witnessed the adoption of automated technologies by infrastructure agencies resulting in significant productivity improvements in relation to manual surface distress data collection processes. This, in turn, provides the opportunity to collect detailed data across the vast lengths of the facilities. This large amount of data poses a potential problem to agencies since their level of detail is not compatible with the scale of interest for maintenance application. The spatial model developed provides the necessary structure for aggregating the detailed distress data in a meaningful manner without any loss of information. Moreover, the spatial model allows for determining the optimal sampling schemes. This results in a more representative data collection by the new technologies. Finally, the spatial model can be used to identify systematic measurement errors resulting from unexpected exogenous factors. However, further research is required to fully understand the use of the spatial model in the two latter applications.

ACKNOWLEDGMENT

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REFERENCES

1. Ben-Akiva, M., F. Humplick, S. Madanat, and R. Ramaswamy. Infrastructure Management under Uncertainty: Latent Performance Approach. *Journal of Transportation Engineering*, Vol. 119, No. 1, 1993, pp. 43–58.
2. Ramaswamy, R. *Estimation of Latent Pavement Performance from Damage Measurements*. Ph.D. dissertation, Massachusetts Institute of Technology, Cambridge, 1989.
3. Madanat, S. Optimal Infrastructure Management Decisions Under Uncertainty. *Transportation Research C*, Vol. 1C, No. 1, 1993, pp. 77–88.
4. *AASHTO Guide for Design of Pavement Structures*. American Association of State Highway and Transportation Officials, Washington D.C., 1986.
5. Maser, K., B. Brademeyer, and R. Littlefield. Pavement Condition Diagnosis Based on Multisensor Data. In *Transportation Research Record 1196*, TRB, National Research Council, Washington D.C., 1988, pp. 62–71.
6. Roddis, W. M. K., K. Maser, and A. Gist. Radar Pavement Thickness Evaluations for Varying Base Conditions. In *Transportation Research Record 1355*, TRB, National Research Council, Washington D.C., 1992, pp. 90–98.
7. Mishalani, R. G. *Extracting Spatial Information on Infrastructure Distress Using a Stochastic Modeling Approach: Application to Highway Maintenance Decision Making*. Ph.D. dissertation, Massachusetts Institute of Technology, Cambridge, 1993.
8. Alfelor, R., and S. McNeil. Heuristic Algorithms for Aggregating Rail Surface Defect Data. *Journal of Transportation Engineering*, Vol. 120, No. 2, 1994, pp. 295–311.
9. Moavenzadeh, F., and B. Brademeyer. A Stochastic Model for Pavement Performance and Management. *4th International Conference on Structural Design of Asphalt Pavement*, Ann Arbor, Mich., *Proc.*, Vol. 1, 1977, p. 876.
10. Markow, M., and B. Brademeyer. *Modification of the System EAROMAR: Final Technical Report*. FWHA-RD-82-086, CTM Inc., 1984.
11. Paterson, W. D. O. *Road Deterioration and Maintenance Effects: Models for Planning and Management*. Johns Hopkins University Press, Baltimore, Md., 1987.
12. Herrmann, H. J., and S. Roux. *Statistical Models for the Fracture of Disordered Media*. Elsevier Science, Amsterdam, The Netherlands, 1990.
13. Humplick, F. Highway Pavement Distress Evaluation: Modeling Measurement Error. *Transportation Research B*, Vol. 26B, No. 2, 1992, pp. 135–154.
14. Diggle, P. A Kernel Method for Smoothing Point Process Data. *Applied Statistics*, Vol. 34, No. 2, 1985, pp. 138–147.
15. Mishalani, R. G., and H. N. Koutsopoulos. Infrastructure Condition Assessment: Definition and Identification of Uniformly Behaving Spatial Fields. *Journal of Infrastructure Systems*, Vol. 1, No. 1, 1995, pp. 44–55.
16. Hartigan, J. A. *Clustering Algorithms*. John Wiley and Sons, Inc., New York, 1975.

Statistical Analysis of Pavement Structure Data in PMS Data Base

M. SEKIGUCHI, M. HOSODA, M. INAGAKI, H. TOMITA, AND T. MCGREGOR

The Tokyo Metropolitan Government Road Authority is currently performing pavement structure surveys using a system that integrates ground penetrating radar and a proprietary borehole camera. The road authority's intention is to increase the efficiency and usefulness of its pavement management system (PMS) data base related to pavement structure. A large amount of data was acquired during 1992 and 1993. By means of statistical methods, these structure data could be manipulated to illustrate the nature of several segments of pavement structure in the Tokyo metropolitan area. The results were successful, thus reassuring the road authority of the usefulness of this relatively new system of PMS data base input. Statistical methods determined that the segment lengths are relatively short (an average of 500 m per segment). The accuracy of the field data is good enough to use as structure thickness for the PMS data base, but it must be noted that normalized thickness is achieved as a result of averaging the actual thickness that includes some irregularity due to the irregularity of the layer. In most of the areas surveyed, designed structures corresponded to measured structures with a few exceptions. These exceptions were detected by the field survey system. Though there is no practical difference between normalized and actual segment thicknesses when they are used to perform falling weight deflectometer inverse analysis, further field data and a statistical data correlation study are needed to develop this method to a more practical level.

Knowledge of pavement structure plays an important role in failure curve determination by providing indispensable information to a pavement management system (PMS) data base. A pavement structure data base must include initially designed data and subsequent maintenance records. A large number of underground utilities (water, sewer, electrical, steam, etc.) in a city area repeatedly require road maintenance work that is likely to overwrite historical structure information. Often no information is available for roads constructed prior to the establishment of modern record keeping. As a result, it is inevitable that those areas of pavement structure with unknown thicknesses within a given road authority jurisdiction will constitute a significant portion of the total road miles represented in the data base.

The survey system integrated with ground penetrating radar (GPR) and the borehole camera (BHC) discussed here was developed specifically for acquiring real structure data to upgrade existing records, and to create records where none exist. Using this system, data have been collected for pavement structure data base input for the Tokyo metropolitan PMS from 1992 to 1993, and this effort is ongoing. Further, the focus here is to describe the use of statistical methods when large amounts of structure data are available.

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DESCRIPTION OF SURVEYS

The pavement structure survey under discussion consists of two individual surveys, the preliminary survey and the precise survey (*1*). The preliminary survey is carried out by vehicle-mounted GPR followed by a segmentation analysis. The central frequency of the radar pulse is 750 MHz. The precise survey is carried out using the BHC followed by a section profiling analysis. Figure 1 contains the working flow diagram.

Preliminary Survey

The preliminary survey is carried out by driving a GPR-mounted survey vehicle at a speed of 30 to 40 kmph. The electromagnetic wave that radiates from the GPR antenna enters the ground and reflects off of each boundary. The return wave forms a continuous wave that is charted in variable density. A study of the chart gives a rough idea of the pavement structure. The initial purpose here is to divide the road into segments by determining which lengths have uniform or homogeneous structure. The structure might change transversely so it is necessary to survey additional lanes to develop a more complete image of the structure, but the current study included single lane data only.

Pavement segments in Tokyo are specifically designed, section by section, with an eye to traffic load and bearing capacity of subgrade using indexes such as the California bearing ratio (CBR). Repeatedly performed maintenance works in an area, however, easily overwrite and alter "as-designed" information.

Segment boundaries cannot be observed from the surface of a lane segment. GPR enables the pavement engineer to determine boundary information rationally by analysis of the trace output.

The purpose of the preliminary survey is not to gain a precise determination of pavement section, but just a segmentation. The GPR survey supplies lists and maps of segmentation, including number of segments, segment length, and start and end coordinates measured with block ending-beginning linkage. The coordinates are input directly into the PMS data base, and the map is used to determine the boring points where simulated video core samples have been taken in the precise survey.

Precise Survey

The velocity of an electromagnetic wave depends upon the material of the medium through which it travels. Direct reading of layer thickness from GPR records does not give an actual thickness. Additionally, the material composition of each layer cannot be determined from a GPR record. Only layer continuity can be deter-

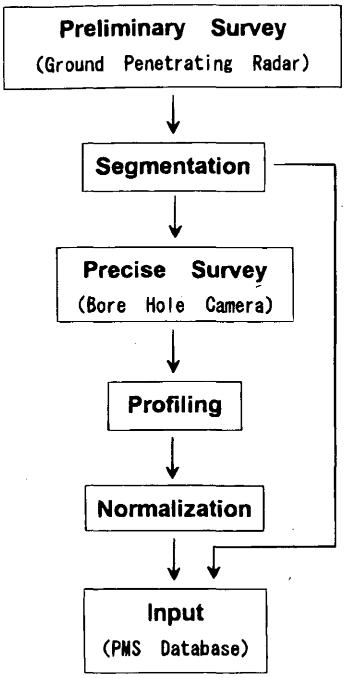

FIGURE 1 Work flow.

mined. Calibration of GPR data by BHC ground truth data is required to determine actual thickness.

For that purpose, a BHC survey, which is preceded by a small diameter boring, is performed. If CBR data are required, a visual observation survey, which must be preceded by a large diameter boring, is performed instead of the BHC survey.

Given a known material composition and actual thickness, a specific dielectric constant is determined by comparing the GPR and BHC data. The scaling accuracy of the BHC image is 0.5 percent. The dielectric constant is used for determining the calibration coefficient. Calibration must be performed not by an automatic comparison, but by a deliberate and rational consideration of dielectric constant values aimed at minimizing errors.

The profile is made by preparing a plot every 10 m. It is too precise to treat as PMS input data since the normal data base segment is 500 m. Consequently, normalized values per segment are prepared as input data by averaging profile values. As the PMS data base evolves over time, it will be possible to treat the precise data themselves as input data because the survey interval of the falling weight deflectometer (FWD) is currently 20 m, and FWD is becoming more popular in Japan, hence the justification for a smaller segment length.

ANALYSIS

Segmentation

Segmentation determination is not automatic; it is conducted with rational consideration. There are some key points for judgment. The five main points to consider are as follows:

- Break of segment should be indicated at the point where the overall appearance of the GPR record changes. But careful attention must be paid to avoid misinterpreting a break at railway crossings, bridges, and local repair locations where a false break could be read.
- Reflection at the bottom of the asphalt mixture layer is usually very strong because of a high contrast in the dielectric constant. The break should be indicated at the location where thicknesses on the GPR record obviously change despite the thicknesses being precalibrated because a calibration error originated from dielectric constants is usually very small and is nullified by the actual thickness.
- A noted difference in the number of layers is itself a decisive factor in the determination of a segmentation break.
- Even though the thicknesses of the asphalt mixture are the same, a break should be noted where the thicknesses of the crushed stone layer are different. Because crushed stone layers are likely to be more irregular than asphalt mixtures, global consideration is needed to avoid misjudgment at locally changed layer thickness points.
- There are some cases where reflection intensities are different while thicknesses are the same. Though it is caused by the difference of dielectric constant contrasts, it is ambiguous whether the anomaly is subgrade oriented or subbase oriented. Most of the cases in the Tokyo area showed that the difference comes from loam as subgrade, which has a very high dielectric constant with water content. Therefore, the break is basically not noted in that case as it is better under such circumstances to select the bottom of the asphalt mixture as the break criteria.

Profiling

Calibration is carried out before profiling. Figure 2 is a schematic explanation of the difference between actual thickness and thickness on the record. The data are calibrated by means of the following formula:

$$Tactual = M \cdot \frac{Vi}{Vo} \cdot Trecord = \frac{M}{\sqrt{\varepsilon i}} \cdot Trecord$$

where

εi = dielectric constant of ith layer,
Vo = electromagnetic wave velocity in a vacuum,
Vi = electromagnetic wave velocity in ith layer's material,
M = constant related to output machine,
$Tactual$ = actual thickness, and
$Trecord$ = thickness on the record.

Materials used for pavement have standard values of dielectric constants. If the value calculated with GPR and BHC data comparison deviates highly from the standard, it will not provide proper calibration and should be excluded.

The dielectric constant of mixed material is calculated from the component materials. In the case of crushed stone, it is calculated as follows:

$$\sqrt{\varepsilon} = \eta a \sqrt{\varepsilon a} + \eta w \sqrt{\varepsilon w} + \eta s \sqrt{\varepsilon s}$$

$$\eta w = \frac{2.5\,\gamma\,(1-\phi)}{1-\gamma}$$

$$\eta a = \phi - \eta w$$

$$\eta s = 1 - \phi$$

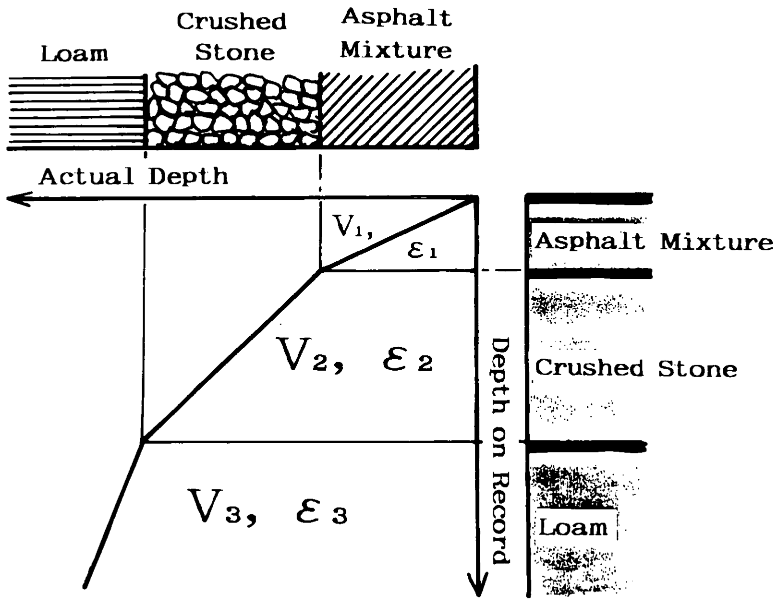

FIGURE 2 Schematic view of dielectric constant effect.

where

- ε = dielectric constant of crushed stone,
- εa = dielectric constant of air (1),
- εw = dielectric constant of water (81),
- εs = dielectric constant of soil particle,
- ηa = volume fraction of air,
- ηw = volume fraction of water,
- ηs = volume fraction of soil particle,
- γ = water content rate, and
- ϕ = porosity.

It has been demonstrated that the dielectric constant of soil particles is assumed to be approximately four (2). The calculated dielectric constant is shown in Figure 3. Besides the theoretical approach, statistical values collected through past calibration works in Figure 4 for crushed stone and in Figure 5 for asphalt mixture are indicated. The standard values are estimated at from four to six for asphalt mixtures and from six to nine for crushed stone.

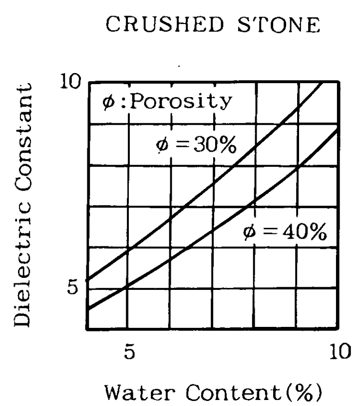

FIGURE 3 Calculated dielectric constant of crushed stone.

Though only one calibrating ratio is determined per segment, that value should not be considered representative of the segment. The tendency of all the values in the route should be looked over first. If the tendency is normal, the average can be taken. If some values are abnormal, the average can be taken after eliminating them. If it is a specific value in the segment, the specific value may be taken for the segment. By these deliberate considerations, the error cancellation will be realized despite each value containing some error-oriented variation.

FEATURE OF SEGMENT LENGTH

Using GPR data acquired in 1992 and 1993, the distribution of segment lengths has been quantified. Figure 6 indicates the distribution, including all the data. Figure 7 includes data for both light traffic and heavy traffic.

Segments with lengths distributed from 100 to 200 m occupy the largest portion of all the segments at over 20 percent. While the longest segment exceeds 2 km, the average segment length is 472 m.

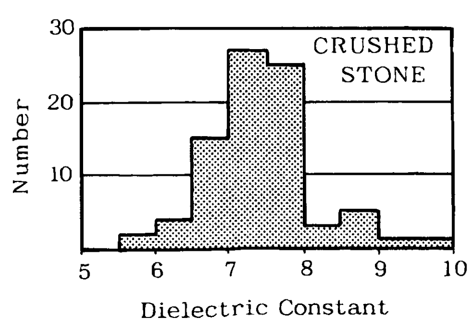

FIGURE 4 Observed dielectric constant of crushed stone.

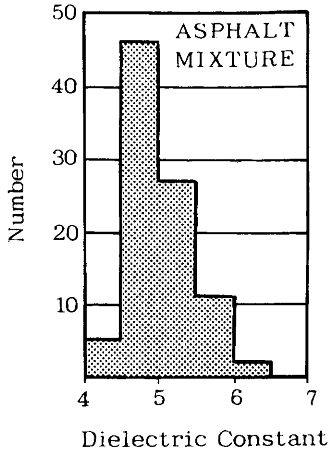

FIGURE 5 Observed dielectric constant of asphalt mixture.

No remarkable difference has been observed in the distribution of segment length correlated with traffic flow. As previously noted, almost all the pavement segments in the Tokyo metropolitan area are designed as an average 500-m-long segment.

The average restriction length for 2,586 cases of road maintenance operations that were performed from 1979 through 1988 is 395 m. In 1992, this figure was approximately 200 m. It is much shorter compared with the previous period because of increasingly severe restrictions in working time on the road (lane closure period) and in working space. The distribution of road working segments in 1992 is depicted in Figure 8.

ACCURACY

The accuracy of layer thicknesses measured by this system depends upon two criteria.

- *Nonuniformity of dielectric constant.* A pavement material has a specific value of dielectric constant but even though it is the same material, the dielectric constant is influenced by nonuniformity, which essentially exists in combined materials such as asphalt mixtures. For example, variation in the relative amounts of aggregate in asphalt mixtures, water content ratio, and porosity in the crushed stone as subbase all bring about a small deviation in dielectric constants. As a result, an error from the small deviation appears in plotted sections after calibration. The error is small in asphalt mixture layers, and larger (±2 cm) in subbase materials.
- *Vehicle speed and irregularity in layer thickness.* The speed of the survey vehicle is about 30 kmph. In the precise survey, a boring is carried out at a representative location within the segment. The boring point cannot be expected to be the exact same point as that on the GPR record. It has already been proven that probability of discrepancy in location increases in proportion to vehicle speed (*3*). This results in some thickness error in plotted sections. This error occurs in linkage with the original application irregularity of layer thicknesses. The layer irregularity is expected to be within an allowable standard by design. Irregularities may become considerably larger as a result of repeated maintenance work, such as cutting and filling and overlay.

In the actual survey, the error occurs as a result of the two aforementioned factors and diminishes thickness accuracy. A test survey (*5*) performed by the Tokyo metropolitan government showed that 84 percent of the asphalt mixture layer data and 78 percent of the crushed stone layer data were within allowable values, which were determined to be ±2.5 cm for asphalt mixture and ±5 cm for crushed stone, considering a vehicle speed of 30 to 40 kmph. Asphalt mixture thickness is designed with a 5-cm tolerance in Tokyo; therefore, an accuracy of ±2.5 cm is required. There is, however, no theoretical reason for crushed stone to use a tolerance of ±5 cm; this figure is determined as a rule of thumb based on the lesser need for accuracy with crushed stone than with asphalt mixtures.

A statistical approach has been performed using 1992 data, which include 300 locations of boring data. Correlations between GPR-oriented data and boring-oriented data are shown in Figure 9 (asphalt mixture) and in Figure 10 (crushed stone). It has been reconfirmed that layer thicknesses can be measured with the same degree of accuracy as with a test survey. Eighty percent of asphalt

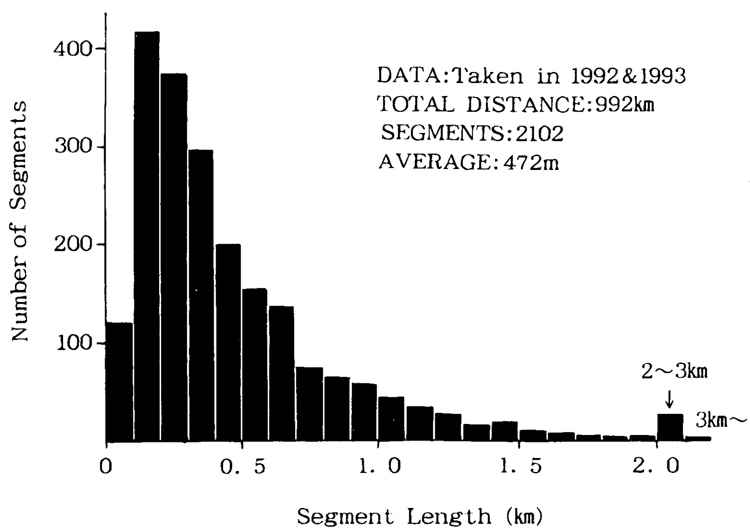

FIGURE 6 Distribution of segment lengths (all data).

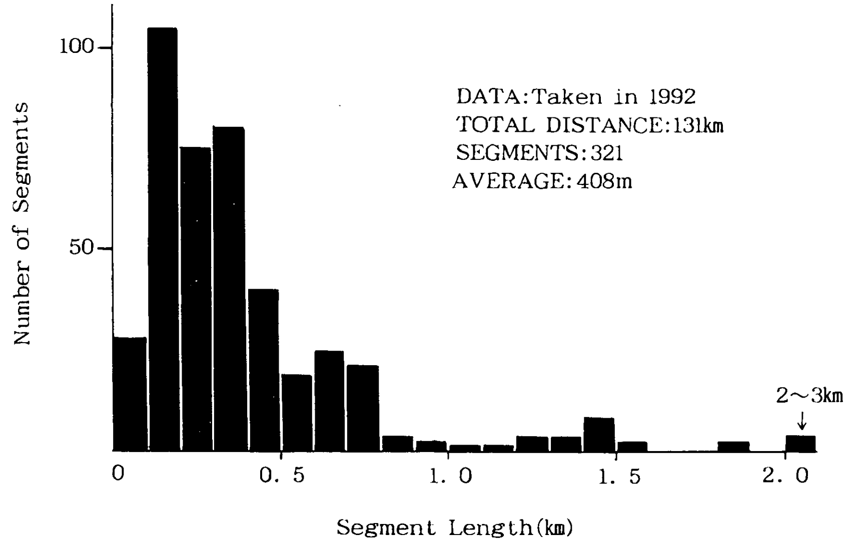

FIGURE 7 Distribution of segment lengths (based on traffic variation).

mixture and 70 percent of crushed stone have been measured within allowable values.

Comparison between GPR and boring data in a static condition (vehicle speed is 0 kmph) indicated that 80 percent of asphalt mixtures were measured within ±1 cm (*5*). This indicates that the main reason for the error originated in the linkage effect of vehicle speed and pavement layer irregularity.

Assuming the data shown in Figures 9 and 10 represent normal distribution, it was determined that a 95 percent reliable range of the data are ±5.1 cm for asphalt mixture and ±10.2 cm for crushed stone. This means the allowable values of ±2.5 cm and ±5 cm are both considered a severe condition. Despite the severity, the allowable values are considered to be reasonable in terms of accuracy needed for pavement structure information, which is stored in the referenced PMS data base.

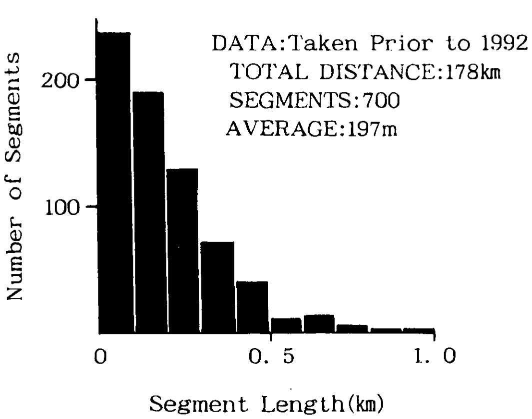

FIGURE 8 Distribution of road maintenance operation segments.

IRREGULARITY OF LAYER THICKNESS

Thickness data stored in the PMS data base are a normalized value. They represent the average and normalized thickness of segments. Basically, normalization is performed by averaging every 10 meters of plotted thickness values in a profile. The concern here is how large an irregularity of layer thickness might be concentrated to only one normalized value. For that purpose, the degree of irregularity was expressed as a distribution of standard deviations. The standard deviations are calculated from layer thicknesses to normalized

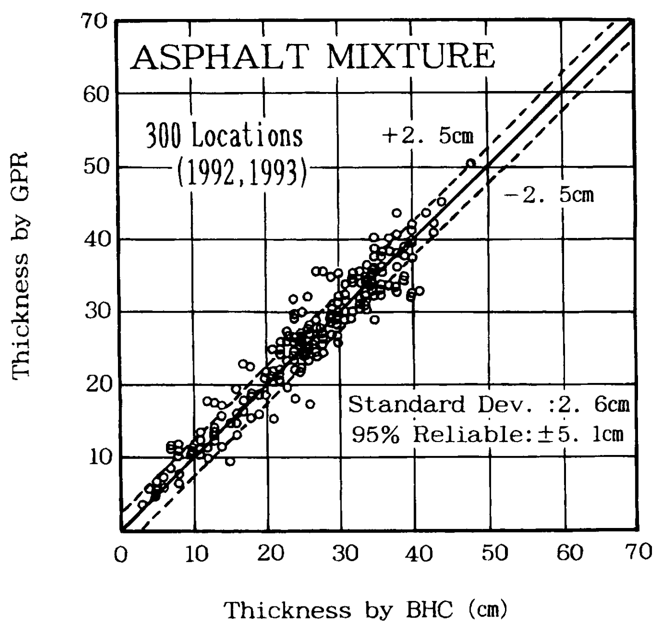

FIGURE 9 Accuracy in asphalt mixture.

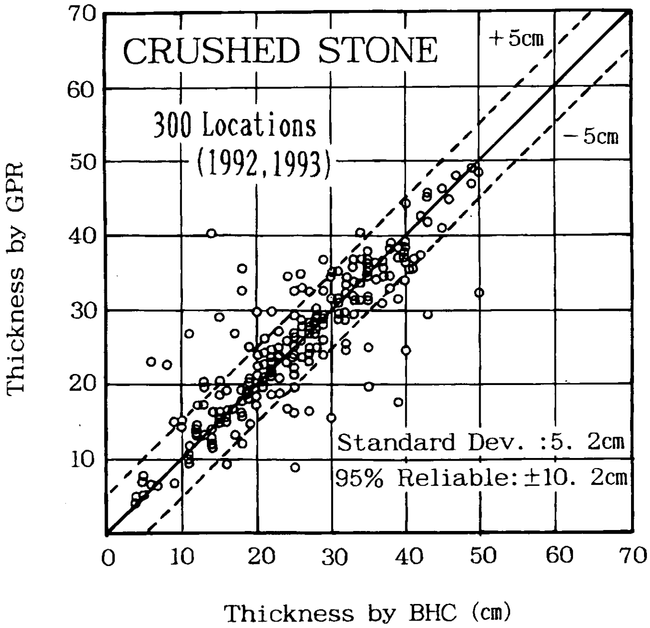

FIGURE 10 Accuracy in crushed stone.

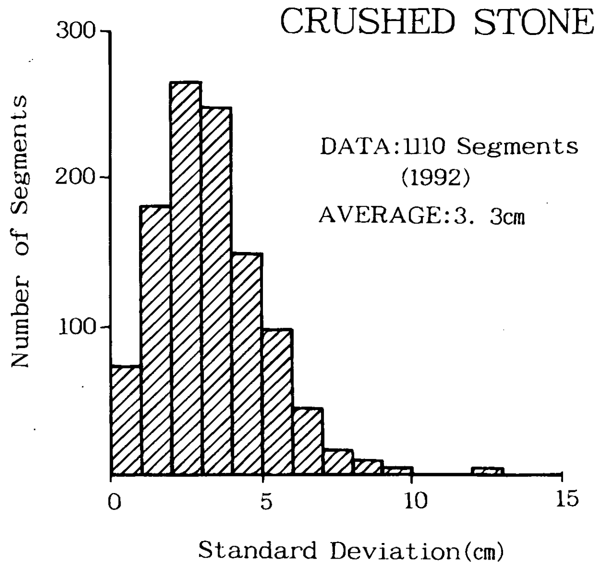

FIGURE 12 Distribution of standard deviation for crushed stone.

value. The results are shown in Figure 11 for asphalt mixtures and Figure 12 for crushed stone. The average standard deviation for asphalt mixture is 2.3 cm and 3.3 cm for crushed stone. The irregularity of asphalt mixture is smaller than that of crushed stone as might be expected.

An example of a GPR record is shown in Figure 13. Surface reflection is not smooth, because the antenna moves up and down on the *Y* axis while the survey vehicle runs on the *X* axis. When a profile is made, an absolute value is taken between the surface and the bottom of asphalt mixture layer. Therefore, surface irregularity does not affect that calculation of layer thickness. What should be kept in mind is that every layer still has an original irregularity even though the surface reflection is shifted to form a smooth line. Nonuniformity of the dielectric constant and vehicle speed influence do not explain the irregularity sufficiently. The irregularity in the record reflects an actual irregularity that originally existed in the pavement structure.

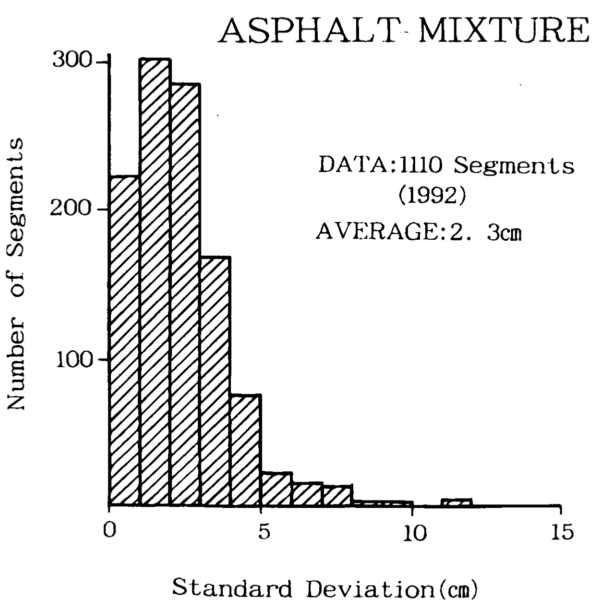

FIGURE 11 Distribution of standard deviation for asphalt mixture.

Ta COMPARISON BETWEEN NORMALIZED AND DESIGNED LAYER THICKNESS

To view the extent of difference between normalized and initially designed thicknesses, the difference was calculated with a Ta structure index comparison. The Ta index, widely used in Japan, can be converted to an AASHTO structure number by dividing the Ta number by 5.68. The result is shown in Figure 14. Data used in calculations are the values of 92 segments that were wholly reconstructed in 1992.

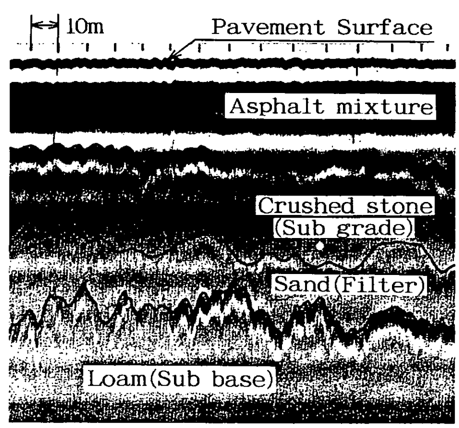

FIGURE 13 Example of GPR record.

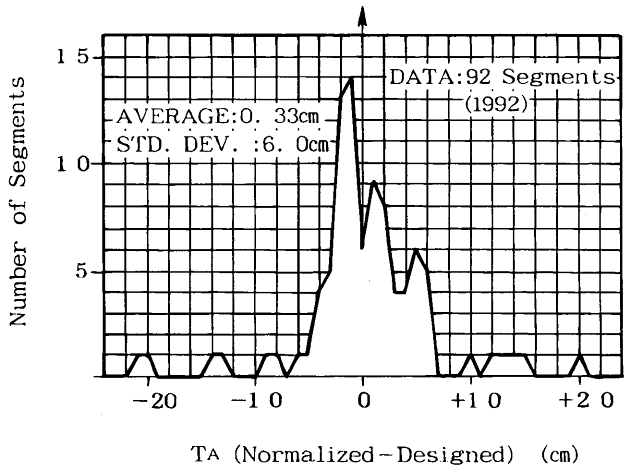

FIGURE 14 Ta comparison between normalized and designed thickness.

Coefficients of relative strength to calculate Ta are 1 for asphalt mixture and 0.55 for cement treatment. Because crushed stone (subbase) is difficult to recognize as a different grade of composition, 0.3 has been adopted as an average value coefficient. The magnitude of error due to adoption of the average value of the coefficient to subbase is estimated to be maximum ± 2 cm, while the subbase thickness is assumed to be 40 cm.

All the data are classified into five types according to asphalt mixture thickness, 45 cm, 36 cm, 35 cm, 32 cm, and 27 cm. The average of subtracted Ta (normalized less designed) is 0.33 cm, which means the normalized value closely corresponds to the designed value. The shape of the distribution looks like a normal distribution with standard deviation and has been calculated to be 6 cm.

Most of the large differences are the result of the difference in asphalt mixture layer thickness. It is considered that repair work such as cutting and overlaying after whole reconstruction resulted in the large differences. Extremely large differences are probably caused by input mistakes of maintenance records or by incorrect determination of sample location. To avoid such a misjudgment, the data base of maintenance history should be updated as soon as possible. The GPR survey is useful to pick up the portions in existing pavement where the design information is uncertain.

CONCLUSION

The data base of pavement structure gives us valuable information that contributes to the more accurate evaluation of pavement design and repair strategy. Pavement structure cannot be observed from the surface of the road, and the accuracy of structure information is easily skewed by ongoing road maintenance work. These factors contribute to the difficulty in managing an accurate road structure data base.

The system integrated with GPR and BHC proved its practicability through actual surveys and statistical analysis using large amounts of data.

The nature of segment length distribution in the routes managed by the Tokyo metropolitan government, the accuracy of measurement, and the relationship between actual thicknesses and normalized thicknesses have been quantitatively clarified. This system can be used to find abnormal locations where designed and actual structures are significantly different.

Change of design conditions due to such changes as traffic load increases requires a change of pavement structure. Design change conditions occur as often as maintenance operations. It is important to make a single structure's segment a quantifiable unit, and to prepare a format that allows the value in the data base to be easily updated.

REFERENCES

1. Inagaki, M., H. Tada, A. Kasahara, H. Tomita, and T. McGregor. Subsurface Pavement Structure Inventory Using Ground Penetrating Radar and a Bore Hole Camera. *ASCE, AATT*, Aug. 1991, pp. 453–457.
2. Nakamura, T., H. Kubota, and M. Inagaki. Utilization of Waterway Tunnel Survey System Using GPR. *Tunneling and Ground Conditions*, April 1994, pp. 407–412.
3. Sekiguchi, M., E. Fujita, M. Inagaki, and T. McGregor. Objective-Oriented Approach to Fulfilling the Need for PMS Pavement Structure Survey Data in Japan. In *Transportation Research Record 1397*, TRB, National Research Council, Washington D.C., 1993.

Effect of Sample Unit Size and Number of Surveyed Distress Types on Pavement Condition Index for Asphalt-Surfaced Roads

M. Y. SHAHIN, CHAD STOCK, MERCEDES CROVETTI, AND LISA BECKBERGER

A study was conducted to quantify the effects of altering the sample unit size for performing a distress survey according to the Pavement Condition Index (PCI) method for asphalt surfaced roads. The effect of consolidating distresses into fewer distress types during condition surveys was investigated. The effect of reducing sample unit size was investigated using surface photographs of 24 asphalt-surfaced pavement sections located in Urbana-Champaign, Illinois. Continuous 35-mm strip photographs of each pavement surface were obtained using the PASCO system. The continuous photographs were subdivided into image units, each 3 m (10 ft) long by one lane wide. Standard sample units, each 60 m (200 ft) long by one lane wide, were developed by grouping 20 contiguous images. The PCI of each sample unit was calculated based on observable distresses using Auto PAVER. Pavement image groups were developed by combining varying numbers of contiguous pavement images. The PCI was then calculated for each group using standard deduct curves and PCI calculation methodology. The effect of reducing the number of recorded distresses was investigated using distress data contained in Micro PAVER data bases from several military installations and cities. Comparisons were made between PCI values calculated using standard PCI procedures (19-distress types) and PCI values calculated using modified distress identification procedures developed by the Metropolitan Transportation Commission (7 distress types).

This paper presents the results of a study to quantify the effects of altering the sample unit size for performing a distress survey according to the pavement condition index (PCI) method, for asphalt-surfaced roads. This paper also investigates the effect of consolidating distresses into fewer distress types during condition surveys.

A primary requirement for effective pavement management is the accurate assessment of present and future pavement condition. As such, a pavement distress survey is an important component of any pavement management system. The information collected from these distress surveys is used to document existing pavement condition, to chart past performance history, and to predict future pavement performance. This information is used in determining appropriate maintenance and repair alternatives and their optimal timing.

Methods have been devised by various agencies to standardize distress classifications. The PCI distress identification and survey procedures developed by the U.S. Army Corps of Engineers have been widely used by many highway and airport agencies (*1,2*). PCI is a repeatable index from 0 to 100, with 100 being excellent, that is used to quantify pavement condition based on distress information.

To increase the efficiency of the rating process, various forms of automation have been introduced for the recording, reduction, processing, and/or storage of data. For example, small handheld computers have been used to speed up recording and transfer of data from the field to the office computer. Vehicles that obtain photographs or other visual images of the pavement surface have been developed to accelerate the field data collection time and to provide a permanent visual record of the pavement condition. However, in most applications, human interpretation of the surface condition, either in the field or in an office environment, is necessary to fully quantify all existing distress (*3*).

To increase the efficiency of distress measurements significantly, methods are needed to accelerate data collection and to reduce the time required for data entry. Advancements are continually being made in the development of specially equipped vehicles for pavement distress survey. The direction of current development activities is the use of video imaging to photograph a portion of pavement and, through pattern recognition technology, classify and quantify pavement distress directly without subjective evaluation by human raters.

Auto PAVER is one such method that simplifies the workload of measuring pavement distresses from digitized images and enters the data into the Micro PAVER system (*4*). Pavement sections are photographed and logged into an image-processing system. Auto PAVER employs sophisticated algorithms to fully process the user-defined distresses, including all necessary data entry tasks into Micro PAVER.

EFFECT OF CHANGING SAMPLE UNIT SIZE ON PCI FOR ASPHALT ROADS

For pavement management, a pavement network is divided into uniform sections based on use, pavement structure, construction history, traffic, and other factors. Each pavement section is further divided into inspection or sample units by which each existing distress is identified and quantified. The recorded distress data are used to calculate the PCI of each sample unit inspected; the PCI of the section is determined by averaging all sample unit PCI values.

The PCI procedures for asphalt-surfaced roads are based on an assumed sample unit size of 230 m^2 (2,500 ft^2). The sample unit size was selected for convenience by the developers of the PCI. For example, the 230 m^2 for asphalt roads is two highway lanes wide (8 m) by 30 m (100 ft) long. Occasionally, it is inconvenient or impossible to obtain a sample unit of that size. For example, the section length is not always divisible into 30-m (100-ft) units.

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The effect of altering sample unit size has never been quantified. The rule of thumb has been that a sample unit size should be within ±40 percent of the recommended size, 230 m^2 ± 90 m^2 (2,500 ft^2 ±1,000 ft^2), for which there was no proven basis. As such, one objective of this study was to determine the effect of varying sample unit size on the PCI value for roads and streets.

This study was limited to asphalt-surfaced pavements. Twenty-four different pavement sections located in Urbana-Champaign, Illinois, including conventional flexible pavement and composite pavement construction, were used. The test sections were surveyed using the PASCO (*5*) photographic system. A continuous set of photographic prints was produced for each lane. Digitized images of one-lane width by 3 m in length (approximately 10 m^2) were developed from the prints. Therefore, sample units of different sizes could be produced by grouping the distress information from several images. For example, a recommended sample unit size of approximately 230 m^2 (2,500 ft^2) is produced by grouping 20 consecutive images, while a sample unit half this size is produced by grouping 10 images.

Distress identification was performed on each digitized image using Auto PAVER V1.0 (*4,6*). Auto PAVER is a mouse-driven computer system that automates distress quantity calculation and creates an image distress file. An additional software program was written to perform PCI calculation on different groups of images. The groups comprised 1, 2, 4, 5, 10, 20, and 40 images. Therefore, the sample unit sizes created ranged from 5 percent to 200 percent of the recommended sample unit size, which would consist of 20 images.

The results of the PCI calculations are provided in Tables 1 and 2. The PCI values shown are outlined as follows:

TABLE 1 Calculated PCI Values for Inspected Sample Units

Pavement ID	Street Name	Number of Sample Units	Sample Unit PCI Value	Rating per Sample Unit
Champ/00002/02E	Newmark Drive	3	47 73 76	Fair Very Good Very Good
Champ/00002/02W		3	60 72 49	Good Very Good Fair
Champ/0005/05N	Curtis Road	3	31 33 12	Poor Poor Very Poor
Champ/00005/05S		3	13 34 30	Very Poor Poor Poor
Champ/00006/06E	First Street	3	21 20 16	Very Poor Very Poor Very Poor
Champ/00006/06W		3	25 34 42	Very Poor Poor Fair
Champ/00008/08N	Logan Road	1	50	Fair
Champ/00008/08S		1	56	Good
Champ/00009/09E	Fourth Street	1	35	Poor
Champ/00009/09W		1	40	Poor
Champ/00010/10N	Chalmers Street	2	34 59	Poor Good
Champ/00010/10S		2	39 51	Poor Fair

(*continued on next page*)

1. A sample unit consisting of 20 consecutive images was first defined. The distress data from the 20 images were added and the regular sample unit PCI calculated (Column 1).
2. The PCI for each of the 20 individual images was calculated, and the average of the 20 image PCI values was reported as the 5 percent sample unit PCI (Column 2).
3. The distress data for each pair of two consecutive images were combined and the PCI of each pair calculated. The average PCI of the 10 image pairs was reported as the 10 percent sample unit PCI (Column 3).
4. Steps 1 through 3 were repeated for different image groups to obtain the PCI values in the remaining columns.

Figure 1 provides a plot of the results for the 10 percent sample unit size. A constrained least square technique was used to fit a fourth-degree polynomial through the data. Similar analyses were completed for the remaining sample unit sizes with the results illustrated in Figure 2. The average change in PCI for each of the sample unit sizes investigated is plotted in Figure 3. As indicated in Tables 3 and 4, the change in PCI is less than a few PCI points for sample unit sizes of ±40 percent of the recommended size.

EFFECT OF REDUCING NUMBER OF DISTRESS TYPES ON PCI FOR ASPHALT ROADS

The PCI is used for pavement evaluation and determination of maintenance and rehabilitation requirements. The PCI is also used to project pavement performance and to establish maintenance and rehabilitation strategies. Therefore, it is imperative that the PCI be repeatable with a reasonable degree of accuracy.

The PCI procedure uses 19 distress types for asphalt-surfaced roads and streets to provide adequate information in all geographical areas. Some users have expressed interest in reducing the num-

TABLE 1 *(continued)*

Champ/00011/11E	Broadway Road	2	50 33	Fair Poor
Champ/00011/11W		2	38 29	Poor Poor
Champ/00012/12E	Broadway Road	3	37 44 42	Poor Fair Fair
Champ/00012/12W		3	42 43 44	Fair Fair Fair
Champ/00013/13E	Lincoln Avenue	5	20 36 33	Very Poor Poor Poor
Champ/00013/13W		5	18 20 24	Very Poor Poor Poor
Champ/00014/14N	Pennsylvania Avenue	2	46 47	Fair Fair
Champ/00014/14S		2	41 52	Fair Fair
Champ/00015/15E	Mattis Avenue	3	56 41 31	Good Fair Poor
Champ/00015/15W		3	26 26 47	Poor Poor Fair
Champ/00016/16N	Bloomington Road	3	33 16 26	Poor Very Poor Poor
Champ/00016/16S		2	36 22	Poor Very Poor

TABLE 2 PCI Values and Standard Deviations per Group

Group Condition Index (GCI)										
Pvmt. ID	Group Size 1 mean	s	Group Size 2 mean	s	Group Size 4 mean	s	Group Size 5 mean	Group Size 10 mean	Group Size 20 (Normal Sample Unit)	Group Size 40 mean
02E	68.98	14.83	64.80	16.18	60.40	18.76	59.25	54.50	47	
	78.4	10.58	75.4	5.80	74.2	3.96	73.75	74.0	73	54
	78.95	8.06	77.9	6.05	76.6	2.41	76.75	76.5	76	
02W	73.65	12.36	70.30	12.98	67.4	12.20	66.5	64.5	60	
	78.35	9.49	76.9	7.43	75.8	6.76	75.25	74.5	72	60
	70.1	19.13	68.6	18.88	63.2	16.07	64	56.5	49	
	49.6	11.67	45.7	7.79	39.8	6.42	38.75	33.5	31	
05N	53.9	17.65	49.7	18.04	41.4	15.18	40	37.0	33	34
	29.15	16.82	26.7	17.12	16.2	8.76	12.5	13.5	12	
05S	33.85	14.49	29.8	14.60	22.0	10.0	21.25	17.5	13	
	48.5	13.86	43.7	12.18	36.8	8.93	37	34.0	34	19
	42.45	11.08	38.3	10.25	34.0	11.66	31.25	30	30	
06E	50.35	18.49	43.8	21.36	33.0	18.26	28.00	26.5	21	
	36.8	12.54	34.8	10.22	28.6	11.72	30.75	27.0	20	20
	41.15	13.70	34.5	13.63	30.4	15.61	27.25	21.0	16	
06W	49.1	15.50	43.7	17.97	38.6	15.47	36.25	33.5	25	
	48.45	8.38	44.9	7.82	39.8	8.58	39.75	37.5	34	29
	57.1	14.09	52.9	8.72	51	8.37	49.75	46.0	42	
08N	65.4	12.68	62.3	12.76	58.8	8.02	56.25	53.0	50	53
08S	72.05	18.58	66.6	15.81	63.0	14.63	63.5	58.5	56	
09E	56.05	16.97	53.3	16.64	47.0	17.0	42.75	39.5	35	38
09W	55.25	10.43	51.4	10.69	47.0	10.51	46.0	43.5	40	
10N	62.3	17.74	56.7	17.22	48.4	13.87	49.75	41.0	34	
	72.15	6.28	70.2	6.48	68.2	6.53	68.25	65.0	59	42
10S	57.1	15.97	53.9	16.49	48.8	18.57	47.0	42.5	39	
	62.3	9.02	59	8.45	57.0	7.04	58.0	56.0	51	40
11E	72.9	16.69	69.9	16.19	64.8	12.81	64.0	57.5	50	
	52.45	10.20	47.7	11.68	40.2	6.11	38.25	34.5	33	39

(*continued on next page*)

TABLE 2 (*continued*)

Group Condition Index (GCI)											
Pvmt. ID	Group Size 1		Group Size 2		Group Size 4		Group Size 5	Group Size 10	Group Size 20 (Normal Sample Unit)	Group Size 40
	mean	s	mean	s	mean	s	mean	mean		mean
11W	66.8	22.04	64.4	24.87	60.4	26.73	59.25	56.5	38	
	51.8	12.48	45.8	12.06	40.4	12.92	39.25	33.5	29	35
12E	58.3	22.38	52.5	18.40	47.4	13.30	46.0	39.0	37	
	64.45	17.64	59.6	17.00	55.6	17.27	54.25	49.0	44	39
	60.75	21.39	56.5	15.18	54	7.87	53.75	49.0	42	
12W	61.0	17.29	55.8	14.56	52.4	17.11	49.0	46.5	42	
	55.65	24.11	51.2	14.78	47.0	12.94	45.25	43.5	43	39
	57.75	20.23	53.5	15.39	49.6	12.94	48.0	47.0	44	
13E	43.9	19.41	38.5	16.30	30.6	13.96	29..5	24.0	20	
	56.55	6.63	53.5	6.26	47.8	5.22	47.0	41.5	36	29
	50.75	16.42	45.9	13.12	42.2	10.06	41.75	35.5	33	
	49.95	11.18	45	13.67	40.4	12.26	39.75	37.0	35	34
	52.45	8.42	50.1	12.14	44.6	15.37	43.0	40.0	37	
13W	53.25	24.37	48.6	26.81	26.2	18.27	24.5	18.0	18	
	47.1	19.21	38.4	18.95	25.6	9.76	26.25	23.5	20	14
	43.55	15.51	37.7	14.46	37.7	13.67	29.5	24.5	24	
	47.9	10.78	42.2	12.44	35.2	5.45	35.75	38.0	29	28
	54.75	12.41	52.5	13.68	45.2	11.34	44.0	40.0	35	
14N	68.75	17.71	63.8	16.67	61.0	18.56	59.0	55.0	46	
	68.55	13.56	62.9	9.63	58.8	7.33	58.0	54.0	47	42
14S	53.8	10.44	51.2	8.77	48.2	10.96	47.5	45.0	41	
	62.75	9.96	61.8	8.24	60.6	7.06	59.5	59.0	52	39
15E	68.1	20.0	64.6	18.48	64.2	15.90	59.75	59.5	56	
	61.2	21.56	56.1	18.83	49.4	7.73	50.0	46.0	41	43
	57	24.70	50.0	22.52	41.6	19.65	40.75	30.0	31	
15W	43.95	23.41	38.8	22.91	31.2	18.75	29.5	27.0	26	
	39.6	19.70	33.6	16.47	33.0	15.84	32.25	28.0	26	26.0
	64.7	21.92	57.1	13.86	50.8	7.01	51.75	48.0	47	
16N	59.6	26.83	56.2	27.47	51.2	26.06	48.5	45.0	33	
	44.75	20.54	37.7	17.36	31.2	22.59	30	19.5	16	17
	49.55	17.79	40.7	11.67	31.8	6.76	30.75	28.0	26	
16S	52.2	14.86	49.9	14.60	44.4	13.90	43.5	42.5	36	
	42.5	14.24	39.9	14.86	34.0	10.05	32	26.5	22	29

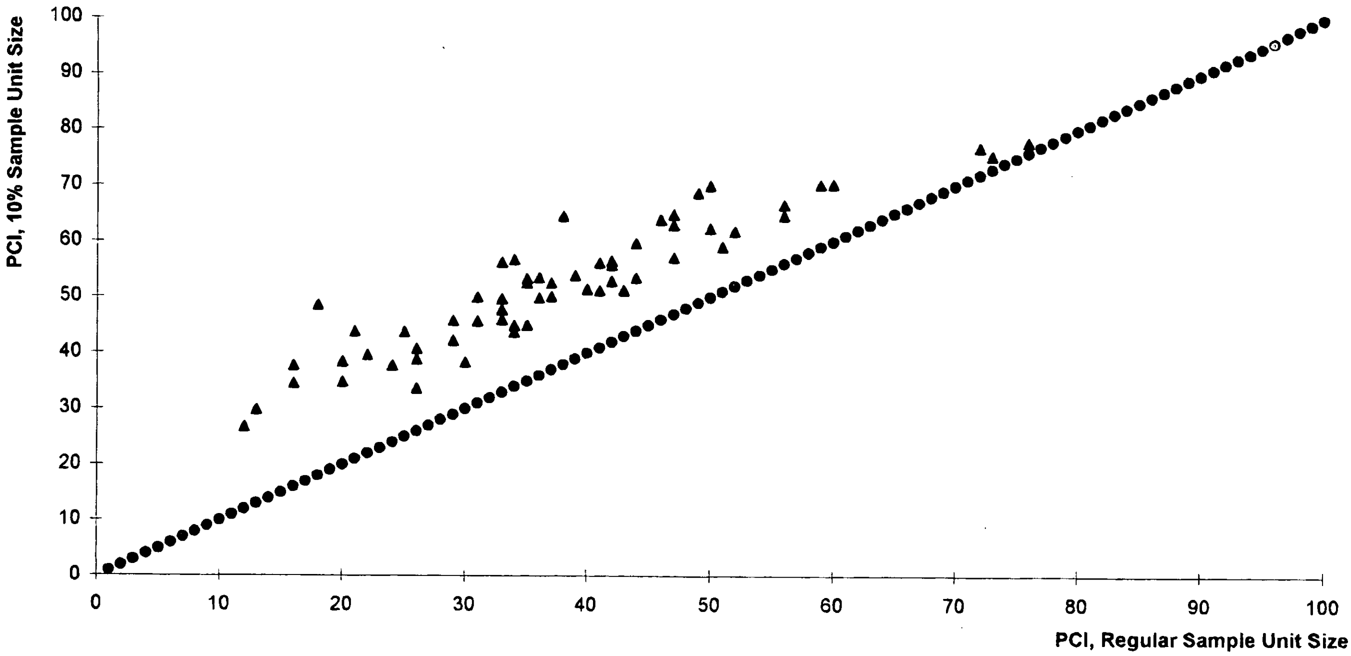

FIGURE 1 **PCI of regular sample unit size versus 10 percent sample unit size.**

ber of distresses used in the PCI procedure to expedite field inspection. This section presents an analysis of the effect of reducing the number of distresses on the PCI values. This study was limited to comparison of the standard PCI method to a modified PCI method used by the Metropolitan Transportation Commission (MTC), Oakland, California, in its pavement management system implementation. The MTC is the transportation planning agency for the 103 cities and counties in the San Francisco Bay Area.

Development of MTC-Modified PCI Procedure

The major objectives of MTC were to expedite the pavement condition survey process and minimize the time required for training the agency staff who will do the survey, while providing adequate information to make reasonable maintenance and rehabilitation decisions. These objectives are addressed by Smith (*7*):

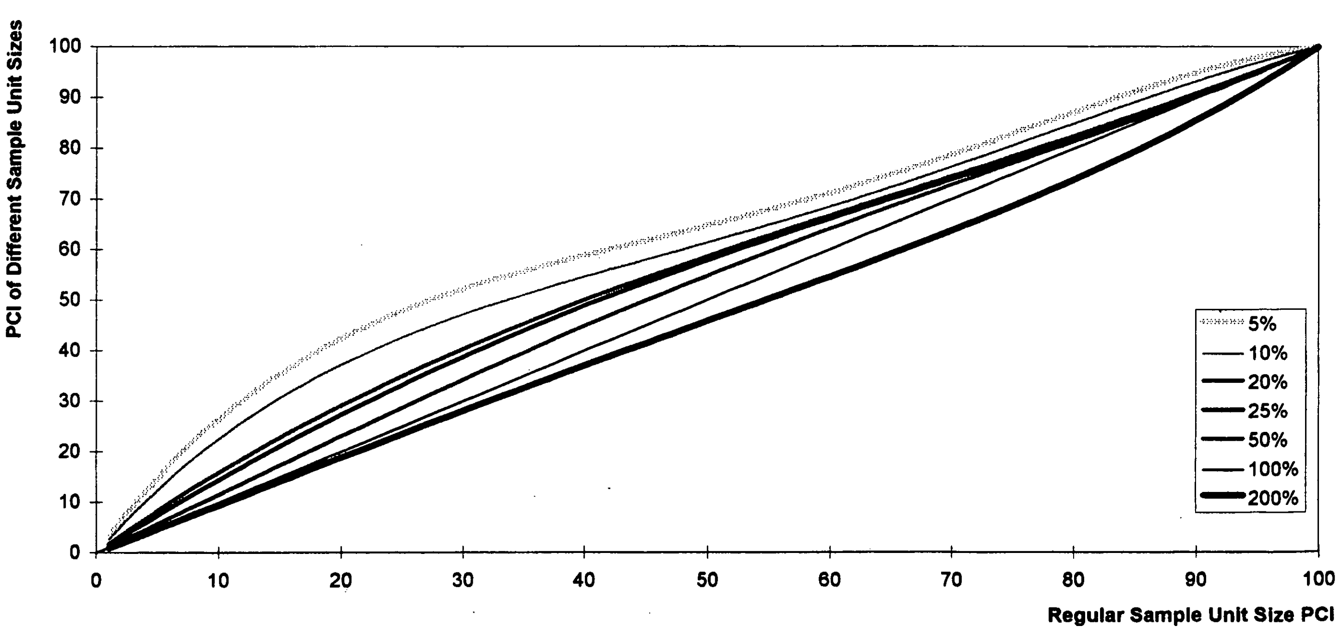

FIGURE 2 **Effect of sample unit size on PCI.**

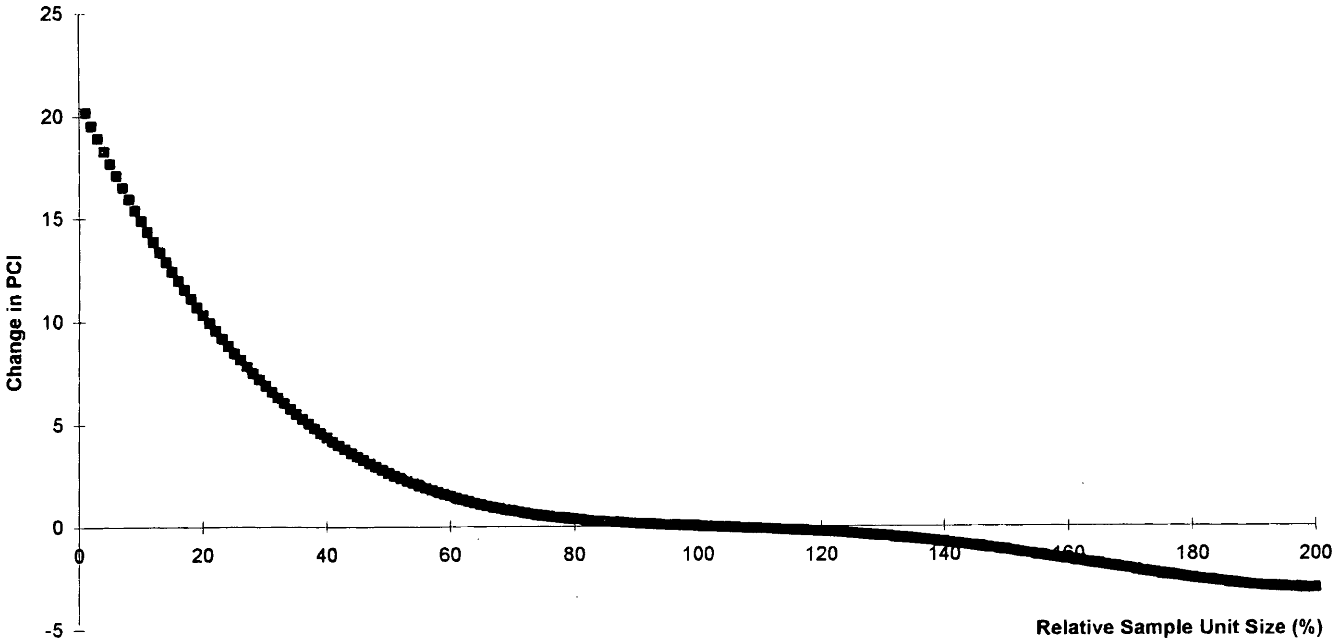

FIGURE 3 Change in PCI versus percent sample unit size.

1. Limiting the distress types in the condition survey procedure to only those usually found in the area of study or implementation site;
2. Limiting the distress types in the condition survey procedure to only those used to identify maintenance and rehabilitation needs in the area of study or implementation site;
3. Combining less common distress types based on distress causation and maintenance requirements; and
4. Developing a sampling technique to expedite the inspection process.

Since more than 95 percent of pavements maintained by cities and counties in the MTC area have bituminous surfaces (*8*), only flexible surfaced pavements were initially included in the system. An analysis of the prevailing distress types occurring in the MTC cities resulted in the compilation of seven key distresses to be used for PCI calculation. These distress types were identified as those that are useful in determining maintenance and rehabilitation needs at the network and project levels. The MTC-modified PCI procedure preserves the rating scale of 0 to 100 and the distress deduct curves used in the conventional PCI procedure. Table 5 presents a list of distress conversions and deduct curves used.

Comparing Modified MTC PCI and Standard PCI Procedure

A software program was developed to recalculate each PCI in a given Micro PAVER data base according to the MTC procedure. The distress types in each sample unit were converted to one of the seven MTC distresses. The program was used with several Micro PAVER data bases from military installations and cities. Table 6 shows summary statistics for several of the data bases. Figures 4 and 5 illustrate the PCI differences for sample units and for entire sections, respectively.

CONCLUSIONS AND RECOMMENDATIONS

The two main objectives of this study were (a) to determine the effect of sample unit size on the PCI value and (b) to determine the impact of PCI value when distress types are consolidated.

Previously completed pavement condition surveys (limited to asphalt-surfaced roads), using pavement surface photography analyzed for distress with the Auto PAVER image-processing software, were used in this study. The PCI was calculated for 61 sample units inspected (approximately 1,220 images). Figure 3 depicts the average effect of sample unit size on the PCI value; however, it should be noted that the effect of sample unit size on the PCI is also a function of the PCI value of the pavement as indicated in Figure 2. Currently, the guidance is to use a sample unit size equal to 230 m^2 (2,500 ft^2) $\pm$ 40 percent. This guidance is acceptable and will provide a PCI value that is $\pm$ two points of the recommended sample unit PCI. It is important to recognize that the comparison was based on digitized images. These results may be different if the comparison was based on traditional visual surveys.

The effect of consolidating distress types on the PCI is summarized in Table 6 for several data bases. The difference in PCI is very dependent on the data base and the types of distresses that exist in any specific site or region. It is evident from Table 6 that there is deviation from the true PCI when reducing the number of distresses. Each agency will have to assess the benefit of reducing the number of distresses versus the deviation from the true PCI.

TABLE 3 PCI for Different Sample Unit Sizes

	Group Size 1	Group Size 2	Group Size 4	Group Size 5	Group Size 10	Group Size 20	Group Size 40
Regular Sample Unit PCI	5%	10%	20%	25%	50%	100%	200%
12	29.15	26.7	16.2	12.5	13.5	12	
13	33.85	29.8	22	21.25	17.5	13	
14						14	14
16	44.75	37.7	31.2	30	19.5	16	
16	41.15	34.5	30.4	27.25	21.0	16	
17						17	
18	53.25	48.6	26.2	24.5	18.0	18	
19						19	14
20	47.1	38.4	25.6	26.25	23.5	20	
20	43.9	38.5	30.6	29.5	24.0	20	
20	36.8	34.8	28.6	30.75	27.0	20	
21	50.35	43.8	33	28	26.5	21	20
22	42.5	39.6	34	32	26.5	22	
24	43.55	37.7	37.7	29.5	24.5	24	19
25	49.1	43.7	38.6	36.25	33.5	25	17
26	43.95	38.8	31.2	29.5	27.0	26	24
26	49.55	40.7	31.8	30.75	28.0	26	
26	39.6	33.6	33	32.25	28.0	26	
27						27	28
28						28	29
29						29	25
29	51.8	45.8	40.4	39.25	33.5	29	
29	47.9	42.2	35.2	35.75	31.0	29	29
30						30	30
30	42.45	38.3	34	31.25	30.0	30	28
31	49.6	45.7	39.8	38.75	33.5	31	
31	57	50	41.6	40.75	30.0	31	
32						32	34
33	53.9	49.7	41.4	40	37.0	33	
33	59.6	56.2	51.2	48.5	45.0	33	
33	52.45	47.7	40.6	38.25	34.5	33	
33	50.75	45.9	42.2	41.75	35.5	33	
34	48.5	43.7	36.8	37	34.0	34	33
34	48.45	44.9	39.8	39.75	37.5	34	
34	62.3	56.7	48.4	49.75	41.0	34	35
35	56.05	53.3	47	42.75	39.5	35	
35	54.75	52.5	45.2	44	38.0	35	
35	49.95	45	40.4	39.75	37.0	35	
36	52.2	49.9	44.4	43.5	42.5	36	
36	56.55	53.5	47.8	47	41.5	36	
37	58.3	52.5	47.4	46	39.0	37	
37	52.45	50.1	44.6	43	40.0	37	38
38	66.8	64.4	60.4	59.25	56.5	38	
39	57.1	53.9	48.8	47	42.5	39	36
40	55.25	51.4	47	46	43.5	40	39
41	53.8	51.2	48.2	47.5	45.0	41	40
41	61.2	56.1	49.4	50	46.0	41	39
42	57.1	52.9	51	49.75	46.0	42	
42	60.75	56.5	54	53.75	49.0	42	40
42	61	55.8	52.4	49	46.5	42	
43						43	38
43	55.65	51.2	47	45.25	43.5	43	39
44	57.75	53.5	49.6	48	47.0	44	
44	64.45	59.6	55.6	54.25	49.0	44	
45						45	40
46	68.75	63.8	61	59	55.0	46	42
47	68.55	62.9	58.8	58	54.0	47	42
47	68.95	64.8	60.4	59.25	54.5	47	
47	64.7	57.1	50.8	51.75	48.0	47	39
49	70.1	68.6	63.2	64	56.5	49	43
50	72.9	69.9	64.8	64	57.5	50	
50	65.4	62.3	58.8	56.25	53.0	50	
51	62.3	59	57	58	56.0	51	
52	62.75	61.8	60.6	59.5	59.0	52	
53						53	43
56	72.05	66.6	63	63.5	58.5	56	53
56	68.1	64.6	64.2	59.75	59.5	56	
59	72.15	70.2	68.2	68.25	65.0	59	
60	73.65	70.3	67.4	66.5	64.5	60	54
66						66	60
70						70	67
72	78.35	76.9	75.8	75.25	74.5	72	
73	78.4	75.4	74.2	73.75	74	73	
76	78.95	77.9	76.6	76.75	76.5	76	

TABLE 4 Change in PCI for Different Sample Unit Sizes

	Group Size 1	Group Size 2	Group Size 4	Group Size 5	Group Size 10	Group Size 20	Group Size 40
Regular Sample Unit PCI	**5%**	**10%**	**20%**	**25%**	**50%**	**100%**	**200%**
12	17.15	14.7	4.2	0.5	1.5	0	
13	20.85	16.8	9	8.25	4.5	0	
14						0	0
16	28.75	21.7	15.2	14	3.5	0	
16	25.15	18.5	14.4	11.25	5	0	
17						0	
18	35.25	30.6	8.2	6.5	0	0	
19						0	-5
20	27.1	18.4	5.6	6.25	3.5	0	
20	23.9	18.5	10.6	9.5	4	0	
20	16.8	14.8	8.6	10.75	7	0	
21	29.35	22.8	12	7	5.5	0	-1
22	20.5	17.6	12	10	4.5	0	
24	19.55	13.7	13.7	5.5	0.5	0	-5
25	24.1	18.7	13.6	11.25	8.5	0	-8
26	17.95	12.8	5.2	3.5	1	0	-2
26	23.55	14.7	5.8	4.75	2	0	
26	13.6	7.6	7	6.25	2	0	
27						0	1
28						0	1
29						0	-4
29	22.8	16.8	11.4	10.25	4.5	0	
29	18.9	13.2	6.2	6.75	2	0	0
30						0	0
30	12.45	8.3	4	1.25	0	0	-2
31	18.6	14.7	8.8	7.75	2.5	0	
31	26	19	10.6	9.75	-1	0	
32						0	2
33	20.9	16.7	8.4	7	4	0	
33	26.6	23.2	18.2	15.5	12	0	
33	19.45	14.7	7.6	5.25	1.5	0	
33	17.75	12.9	9.2	8.75	2.5	0	
34	14.5	9.7	2.8	3	0	0	-1
34	14.45	10.9	5.8	5.75	3.5	0	
34	28.3	22.7	14.4	15.75	7	0	1
35	21.05	18.3	12	7.75	4.5	0	
35	19.75	17.5	10.2	9	3	0	
35	14.95	10	5.4	4.75	2	0	
36	16.2	13.9	8.4	7.5	6.5	0	
36	20.55	17.5	11.8	11	5.5	0	
37	21.3	15.5	10.4	9	2	0	
37	15.45	13.1	7.6	6	3	0	1
38	28.8	26.4	22.4	21.25	18.5	0	
39	18.1	14.9	9.8	8	3.5	0	-3
40	15.25	11.4	7	6	3.5	0	-1
41	12.8	10.2	7.2	6.5	4	0	-1
41	20.2	15.1	8.4	9	5	0	-2
42	15.1	10.9	9	7.75	4	0	
42	18.75	14.5	12	11.75	7	0	-2
42	19	13.8	10.4	7	4.5	0	
43						0	-5
43	12.65	8.2	4	2.25	0.5	0	-4
44	13.75	9.5	5.6	4	3	0	
44	20.45	15.6	11.6	10.25	5	0	
45						0	-5
46	22.75	17.8	15	13	9	0	-4
47	21.55	15.9	11.8	11	7	0	-5
47	21.95	17.8	13.4	12.25	7.5	0	
47	17.7	10.1	3.8	4.75	1	0	-8
49	21.1	19.6	14.2	15	7.5	0	-6
50	22.9	19.9	14.8	14	7.5	0	
50	15.4	12.3	8.8	6.25	3	0	
51	11.3	8	6	7	5	0	
52	10.75	9.8	8.6	7.5	7	0	
53						0	-10
56	16.05	10.6	7	7.5	2.5	0	-3
56	12.1	8.6	8.2	3.75	3.5	0	
59	13.15	11.2	9.2	9.25	6	0	
60	13.65	10.3	7.4	6.5	4.5	0	-6
66						0	-6
70						0	-3
72	6.35	4.9	3.8	3.25	2.5	0	
73	5.4	2.4	1.2	0.75	1	0	
76	2.95	1.9	0.6	0.75	0.5	0	
Average	**18.74**	**14.46**	**9.17**	**7.92**	**4.12**	**0.00**	**-2.91**

TABLE 5 Distress Conversion

Reduced List	*Micro PAVER Distresses*	*Deduct Curves Used*
Alligator Cracking	Alligator Cracking Potholes Slippage Cracking Edge Cracking (High Severity)	Alligator Cracking
Block Cracking	Block Cracking	Block Cracking
Distortions	Corrugations Bumps and Sags Shoving Swell	Corrugations
Longitudinal & Transverse Cracking	Longitudinal & Transverse Cracking Edge Cracking (Low and Med. Severity) Joint Reflection Cracking	Longitudinal & Transverse Cracking
Patching (& Utility Cut Patching)	Patching (& Utility Cut Patching)	Patching
Rutting & Depressions	Rutting & Depressions	Rutting
Weathering & Raveling	Weathering & Raveling	Weathering & Raveling
Not Counted	Bleeding Lane/Shoulder Drop-Off Polished Aggregate Railroad Crossing	

TABLE 6 Summary of Statistics Between Standard PCI and MTC PCI (Consolidated Distress PCI)

Database	Difference, Absolute Mean	Difference, Arithmetic Mean	Difference, Standard Deviation
Fort Lee, VA	3.97	0.61	6.27
Oakdale	4.23	2.56	4.98
USACERL	4.67	3.98	4.63
Fort Leonard Wood, MO	6.78	7.17	6.28
Pinellas Park, FL	1.00	0.23	3.19
Rockland, NY	2.79	0.46	4.86

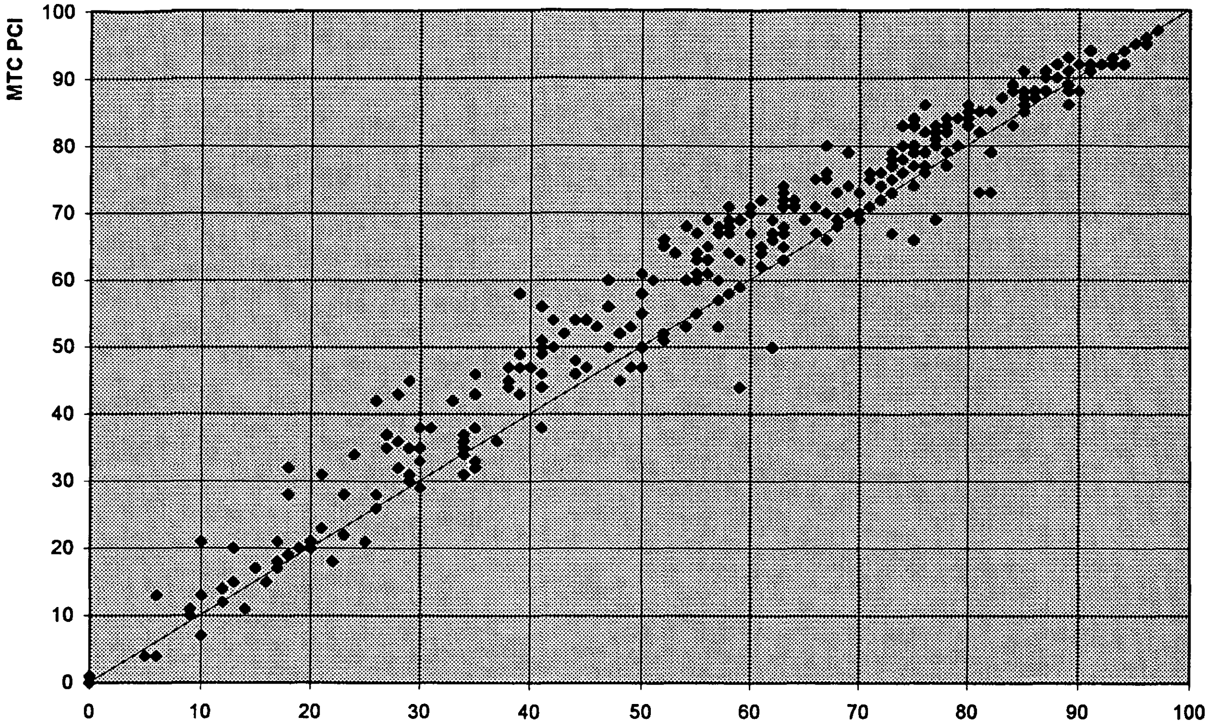

FIGURE 4 Consolidated distress PCI (sample unit).

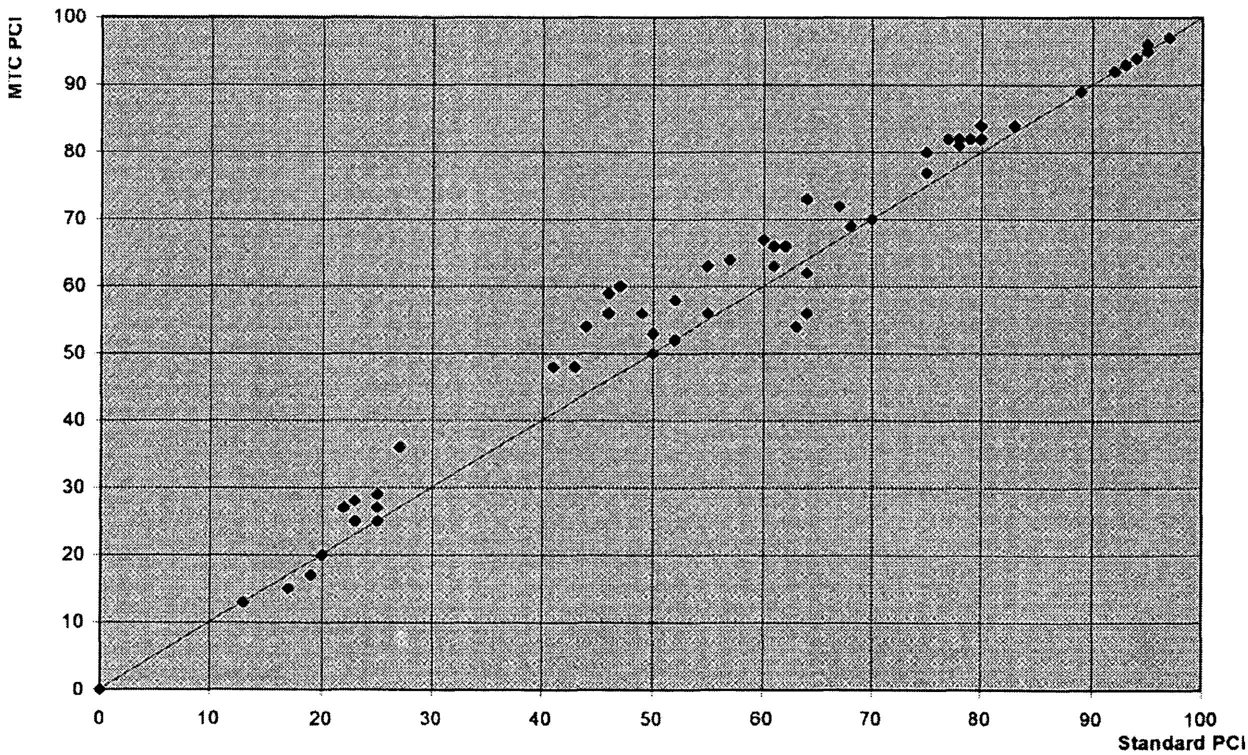

FIGURE 5 Consolidated distress PCI (section).

REFERENCES

1. Shahin, M. Y. *Pavement Management for Airports, Roads, and Parking Lots*, Chapman and Hall, New York, 1994.
2. Shahin, M. Y., and J. A. Walther. *Pavement Maintenance Management for Roads and Streets Using the PAVER System.* USACERL, M-90/05, U.S. Department of the Army, 1990.
3. Hudson W. R., et al. *Improved Methods and Equipment to Conduct Pavement Distress Surveys.* Report No. FHWA-TS-87-213, Final Report. FHWA, April 1987.
4. Ginsberg, M. D., M. Y. Shahin, and J. A. Walther. *Auto PAVER: A Software Package for Automated Pavement Evaluation.* USACERL Technical Report M-90/15. U.S. Department of the Army, 1990.
5. PASCO Road System. PASCO Corporation, c/o Mitsubishi International Corporation, Project and Development Division, New York, 1987–1988.
6. Shahin, M. Y., and S. D. Kohn. *APWA—PAVER Pavement Condition Index Field Manual.* American Public Works Association, Chicago, Ill., 1984.
7. Smith R. E. *Pavement Maintenance Management Study in the San Francisco Bay Area.* Final Report. ERES Consultants, Champaign, Ill., 1985.
8. Smith R. E. *Structuring a Microcomputer Based Pavement Management System for Local Agencies.* Ph.D. dissertation. University of Illinois, Urbana-Champaign, 1986.

The views of the authors do not purport to reflect the position of the Department of the Army or the Department of Defense.

Metropolitan Planning Organizations and Pavement Management: The Massachusetts Experience

CORNELIUS W. ANDRES AND MATTHEW TURO

Management system mandates contained in the Intermodal Surface Transportation Efficiency Act of 1991 (ISTEA) quadrupled the roadway mileage that must be included under a pavement management system (PMS) in Massachusetts. Most of the additional mileage is local jurisdiction roadways that are eligible for Surface Transportation Program funds under ISTEA. To handle this additional responsibility, the Massachusetts Highway Department is working cooperatively with the commonwealth's metropolitan planning organizations (MPOs). This partnership is in keeping with the spirit of ISTEA and takes advantage of Massachusetts MPOs' experience in pavement management as well as local agency investments in pavement management systems. An appropriate interagency PMS is being developed to address planning, programming, budgeting, design, and maintenance requirements of a roadway network composed of various classes of pavement under the control of many jurisdictions. This is being accomplished by harmonizing (making the individual systems work together) rather than by standardizing a single PMS. The PMS is being integrated into a geographic information system data base shared by other management systems.

Several years ago, the Massachusetts Highway Department began the process of developing a statewide pavement management system (PMS). This effort was being accomplished in cooperation with regional and local agencies. Since that time, federal legislation has mandated that each state have a pavement management system for all roadways eligible for federal aid, regardless of jurisdiction. This includes all public roadways except for rural minor collectors and local roadways. This legislation, the Intermodal Surface Transportation Efficiency Act (ISTEA), has acted as a catalyst to accelerate the development of the statewide pavement management system. In two years, it will have quadrupled the number of miles that must be included in the state's PMS.

This paper describes how Massachusetts is unifying state, regional, and local pavement management efforts so that they provide consistent data for the statewide PMS. Consistency is required to determine overall network condition and to assess the priorities of projects from all regions of the state.

The state, however, did not want to achieve this consistency at the cost of compromising the individuality of the existing PMSs, which use various software packages. These software packages have been refined over time and rely on specific condition survey procedures. The procedures range from automated data collection at the state level to windshield surveys at the local level. The various software packages trigger actions (candidate projects) based on specific criteria. If condition data are not appropriate, the ability of the PMSs to accurately predict actions and budgets may be severely diminished.

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The state, therefore, chose to harmonize the individual pavement management systems (make the systems work together) rather than mandate a standardized system because there is no single pavement management system that is appropriate for all agencies. Additionally, standardization is politically difficult. Local and regional agencies have a great deal invested in their individual systems. Standardization would also be financially devastating to many private consultants who have developed PMS software.

BACKGROUND

The Commonwealth of Massachusetts comprises 351 cities and towns. These are contiguous jurisdictions with no unincorporated land between them. Each has responsibility for the local public roadways within its jurisdiction. There are 14 counties in the state. These counties, with a few exceptions, are an archaic level of government with few responsibilities. They are not responsible for roadway maintenance. The state is also divided into 13 regional planning areas that, in most cases, do not follow county bounds. There is a regional planning agency (RPA) for each of these areas.

All the regional planning areas, except for three, include urbanized areas with populations of over 50,000 people and are thus mandated to have metropolitan planning organizations (MPOs). In Massachusetts, the MPOs are generally composed of the RPA, the regional transit authority, the Massachusetts Highway Department, and the Executive Office of Transportation (*1*). The three regional planning areas that are not MPOs have been organized as informal MPOs and will hereafter be referred to as MPOs. The transportation staffs of the RPAs are the recipients of transportation planning funds provided to the state by the FHWA. The RPAs provide technical assistance to the local communities and serve as the transportation planning staff of the MPO. ISTEA also recognized the usefulness of this regional approach and mandated MPO involvement with pavement management.

There are five district offices of the Massachusetts Highway Department (MHD). Their bounds are not common with those of the counties or RPAs.

Available Resources

State

The Massachusetts Highway Department Pavement Management Section was established in November 1986. Its main purpose is to coordinate the pavement-related activities involved in planning, design, construction, maintenance, research, and rehabilitation. It is staffed by six full-time engineers. The Pavement Management Section has an automatic road analyzer (ARAN), skid testing unit, and a falling weight deflectometer. Organizationally, the MHD has a fully equipped materials laboratory and a pavement design and engineering section.

MPO

The transportation staffs of the MPOs are primarily planning staffs. Given that several jurisdictions control the roadways in any region, it is reasonable that the MPOs could serve a necessary and coordinating role in network-level pavement management. This role has been described as "ranging from that of an 'initiator' or 'facilitator' to that of a 'coordinator' or 'doer' " (*2*). The MPOs, however, are not organized to handle the detailed engineering requirements of pavement management. They typically turn to the state highway agency (SHA), technology transfer center, local engineering departments, or private consultants for this type of assistance.

In the early 1980s, the MPOs in Massachusetts started to assist local communities with implementing PMSs. Typically, the MPO would provide training, analysis, reports, and presentations to local officials. The local community would collect data. Some MPOs pooled these local efforts to estimate regional needs (*3*). The recent ISTEA pavement management mandate has focused MPO resources toward examining all federal-aid roadways in their jurisdictions rather than all the roadways under the control of specific municipalities. This new direction, however, takes advantage of the previous pavement management efforts, as well as the MPOs' familiarity with independent local pavement management efforts in their regions.

Local

There is tremendous variation in the resources and abilities of the local highway agencies. They range from cities with engineering and maintenance staffs to small maintenance departments run by working foremen. Massachusetts has encouraged pavement management at these local agencies through the regional efforts described above, as well as through its pavement management policy (*4*). This policy, which was established in 1989, allows local highway agencies to use state-aid funds to establish PMSs.

Coordination

The state PMS works cooperatively, through the MPOs, with the more advanced local agencies, some of which have established PMSs. This avoids duplication of effort. In the smaller communities, which may have only a few federal-aid roadways within their jurisdictions, the MPOs coordinate data collection, which is performed by MPO staff, summer engineering interns, or contracts with private consultants. MPO coordination ensures consistent data collection.

ISTEA Requirements

This section describes parts of the Intermodal Surface Transportation Efficiency Act that are relevant to the development of Massachusetts's statewide PMS.

Section 1024. Metropolitan Planning

23 USC 134(f) contains factors to be considered (in the development of long range plans):

(1) Preservation of existing transportation facilities;

(9) The transportation needs identified through the use of the management systems; and

(12) The use of life-cycle costs in the design and engineering of bridges, tunnels, and pavements.

Section 1025. Statewide Planning

23 USC 135(b) addresses coordination with metropolitan planning, the state implementation plan. In carrying out planning under this section, a state shall coordinate such planning with the transportation planning activities carried out under Section 134 of this title for the metropolitan areas of the state.

23 USC 135(c) covers the state planning process. Each state shall undertake a continuous transportation planning process that shall, at a minimum, consider the following:

(1) The results of the management systems required pursuant to Subsection (b) (see above);

(5) The transportation needs of the nonmetropolitan areas through a process that includes consultation with local elected officials with jurisdiction over transportation;

(15) The transportation needs identified through use of the management systems required by Section 303 of this title;

(18) Long-range needs of the state transportation system; and

(20) The use of life-cycle costs in the design and engineering of bridges, tunnels, and pavements.

Section 1034. Management Systems

(a) The states shall develop pavement management systems. In metropolitan areas, the management systems shall be developed and implemented in cooperation with the MPOs; 500.107 (From Proposed Rule Making for Management Systems):

(a) Each state shall have procedures, within the state's organization, for coordination of the development, establishment, and implementation of the management systems. The procedures must include an oversight process to ensure that adequate resources are available for implementation and that target dates of the systems are complementary so that the outputs of all the systems can be given timely consideration in the development of metropolitan and statewide transportation plans and programs.

(d) Each state shall be responsible for overseeing and coordinating such activities.

STATEWIDE PMS OVERVIEW

The procedure with which Massachusetts will meet the requirements set forth in ISTEA incorporates the evaluation and inventory of the entire federal-aid eligible highway system in the state—regardless of jurisdiction. This statewide PMS also includes linkages with a central computerized geographic information system (GIS) data base, the development of economic models and budgets, procedures to assess the priorities of pavement maintenance and rehabilitation projects, and an institutional framework for the statewide PMS. Figure 1 presents the activities at both the state and MPO levels that are required to proceed from a network evaluation to the development of rehabilitation projects and needs.

As indicated in Figure 1, the responsibilities for evaluating and analyzing the federal-aid roadway system have been divided between the state highway agency and the MPOs. The state will survey all roadways that can be tested with the department's ARAN. These roadways include the entire national highway system (NHS) and other roadways that are eligible for Surface Transportation Program (STP) funds. The MPOs will be responsible for the coordination of data collection for the remainder of the federal-aid system. As previously stated, this will include gathering data from the more advanced local agencies that already have acceptable pavement management systems, as well as obtaining data for the roadways in communities that do not yet have an acceptable PMS.

Because each PMS has distinctive data requirements for triggering treatment selection, all data will be analyzed using the PMS for which they were collected. Analyzing the condition data within the respective system keeps the individual integrity of each PMS intact. Correlating condition data to a standardized index before analysis would severely diminish the strengths of each individual system. These strengths include features such as triggering actions based on the type of distress, drainage conditions, or curb reveal (insufficient curb reveal can prohibit certain actions in urban areas). During this phase of network-level analysis (conducted by the state for national highway system roadways and the MPOs for Surface Transportation Program roadways), potential treatments for candidate projects and estimates of overall budget needs are developed. Treatment selection will, of course, be based on costs and pavement performance for typical pavements in the region.

Based on this analysis, the state will forward candidate projects and cost estimates for local roadways evaluated with its ARAN to the MPOs for inclusion in estimates of their regional needs. The candidate projects will then be refined through project-level analysis conducted by qualified personnel (town engineers, state-aid engineers, MPO staff, consultants, etc). The MPOS will, in turn, forward the network-level project list for state roadways to the state for inclusion in estimates of state highway needs. Project-level analysis will then be carried out at the district level of the Massachusetts Highway Department.

Until this point, emphasis has been placed on developing a list of candidate projects and determining network-level budget needs. However, to observe existing and projected statewide conditions, and to assess priorities across the state, a uniform measure of pavement condition must be developed. This is the point at which the individual systems will be harmonized. This will be accomplished through a correlation of condition surveys. In Massachusetts, this task requires the correlation of the condition ratings of the three most common PMSs to the SHA's PMS. The harmonized condition data will be used by the SHA to assess network conditions and to develop a ranking of all NHS and STP projects to determine regional funding requirements.

Eventually, through the statewide and metropolitan planning processes (which consider the results of the other management systems), projects will be programmed for construction. After construction, the PMS data base will be updated with "as-built" data. The SHA will be responsible for maintaining historical records for its pavements. The MPOs will serve as regional repositories for the historical roadway records of all other federal-aid roadways within their jurisdictions. This regional approach offers the advantage of

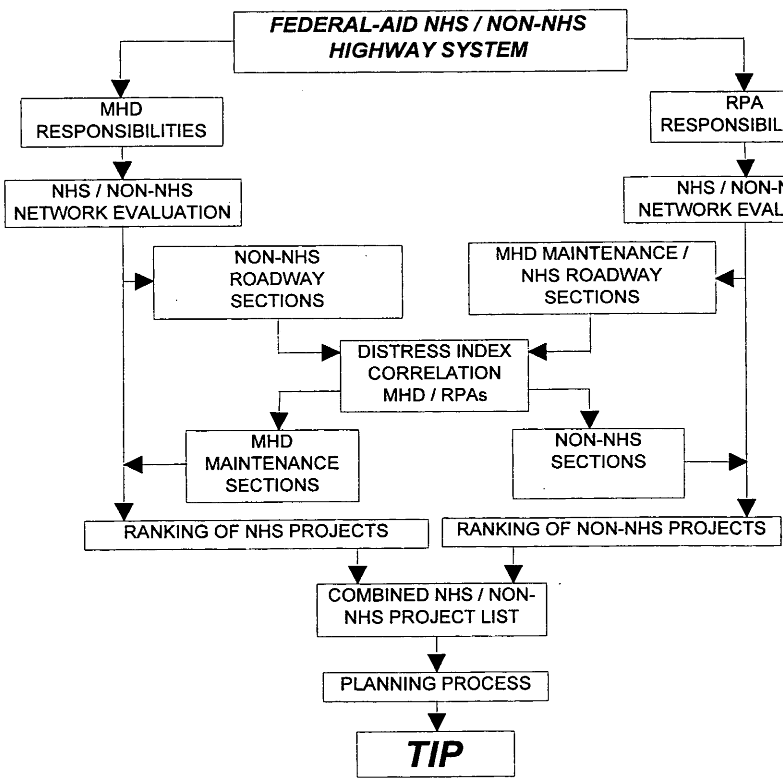

FIGURE 1 Massachusetts pavement management activities.

allowing the pooling of regional deterioration data so that accurate multiyear projections can be made quickly. Obviously, feedback will be crucial and necessary to ensure the credibility and reliability of the overall PMS process.

COORDINATION WITH OTHER MANAGEMENT SYSTEMS

Coordination with other management systems is being accomplished through the use of a shared GIS platform. This approach is a natural outgrowth of previous work efforts mandated by ISTEA. It also takes advantage of the latest technology available for transportation planning.

Massachusetts began the coordination of the management systems by accomplishing the revision of the urban-rural boundaries and the functional classification update with the GIS system. The completion of these steps determined the federal-aid roadway network that the statewide PMS had to address. Existing state inventory numbers were attached to the roadway segments in the GIS so that existing attribute data, such as lane width, pavement type, and jurisdiction, could be attached. The functionally classified network has also been used for transportation modeling purposes. Traffic monitoring, safety, bridge, public transportation, and intermodal facilities data also share a common GIS platform.

The results of all the management systems will be examined through the planning process. It is anticipated that coordination of condition, capacity, safety, and mobility factors identified through the respective management systems will provide valuable information to decision makers.

CONCLUSIONS

- PMSs must fit into institutional systems. In Massachusetts, the PMS fits into the existing MPO structured regional planning arrangements. This approach is practical and fully consistent with ISTEA.
- Massachusetts chose to harmonize the individual pavement management systems (make the systems work together) rather than mandate a standardized system because no single pavement management system is appropriate for all agencies. Various pavement management software packages are used to develop candidate projects and cost estimates. The distress indexes of the individual PMS software packages will then be correlated to the state condition index. This allows comparison of the condition of different roadway segments without compromising the ability of the individual network-level PMS software packages to predict potential treatment.
- PMSs can share a common data base with other management systems. The roadway inventory portion of the statewide PMS development was based on the urban-rural boundary revision and functional classification update requirements of ISTEA. These efforts resulted in a GIS data base that is shared with the other management systems.
- Communication between agencies is essential. In Massachusetts, communication was facilitated by establishing user groups (pavement management and transportation modeling) for regional agencies. These organizations have fostered communication between the state and the regions and helped to reduce institutional barriers.

REFERENCES

1. Luce, D. *Regional Planning Agency (RPA) and Metropolitan Planning Organization (MPO) Activities: The 3C Transportation Planning Process in Massachusetts.* Massachusetts Highway Department, Boston, 1993.
2. Ofori-Darko, E. *The Role of Metropolitan Planning Organizations in Pavement Management.* Master's thesis, University of Massachusetts, Amherst 1991.
3. *Funding the Rehabilitation and Maintenance of Locally Maintained Roads in the MAPC Region.* Metropolitan Area Planning Council, Boston, Mass., 1987.
4. Johnson, R. H. *Policy Directive No. 89-21.* Massachusetts Department of Public Works, Boston, 1989.

Network-Level Analysis of Staged Pavement Rehabilitation and Reconstruction

VERA MIJUSKOVIC, DRAGAN BANJEVIC, AND GORAN MLADENOVIC

An analysis of the influence of the staged construction on the overall network condition was performed. It investigates the relations between strategies concerning the sequence of rehabilitation works for one-step and staged construction. The road network pavement deterioration and repair were described by means of a controlled nonhomogeneous Markov process. The influence of initial network condition on final condition and users' costs for both one-step and staged construction was studied. It was concluded that the basic relations between strategies for one-step construction remained unchanged in the case of staged construction. Also, there was no difference between the effects considered (backlog, extra users' costs and extra routine maintenance costs) of two types of construction in the first 15 years of a 20-year design period. No reason has been found to improve a greater part of network by measures of shorter service life instead of improving a minor part of network by measures of longer service life.

Quality improvements of a road network in very poor condition are usually limited by budgetary constraints. Although project-level analysis, based on life-cycle costs, has indicated that staged construction is not profitable, highway officials are often forced into repairs with shorter service lives and postponement of achieving excellent condition. Other studies have proved that "what is the best for the section must not be the best for the whole network" (*1,2*). Thus, the basic aim of investigations described hereafter was to compare the network-level effects of two ways of producing a high-quality pavement: one-step and staged construction.

The network-level pavement performance prediction model, used to describe the interdependence between pavement quality and preservation strategy for one-step construction, has already been presented in previous studies (*3,4*). Only the basic characteristics needed to understand the adaptations made for staged construction simulation will be presented herein. Since the model deals with strategies defined as principles, it is as simple as possible. For practical use, it has to be widened and calibrated.

MODEL DESCRIPTION FOR ONE-STEP CONSTRUCTION

Road networks are classified according to pavement type, pavement width, design period, and traffic volume. Pavement condition on the part of a network of the same type, same width, and in *k*th class of traffic volume in a year *i* is described by *state vector* α_k^i:

$$\alpha_k^i = \left[\alpha_{10,k}^i\ \alpha_{11,k}^i\ \alpha_{12,k}^i\ \alpha_{20,k}^i\ \alpha_{21,k}^i\ \alpha_{3,k}^i\ \alpha_{4,k}^i\right] \qquad (1)$$

where

$\alpha_{10,k}^i$ = the contribution of new, strengthened, and reconstructed road sections in excellent state;
$\alpha_{11,k}^i$ = the contribution of new, strengthened, and reconstructed road sections in excellent state after one treatment with a thin layer;
$\alpha_{12,k}^i$ = the contribution of new, strengthened, and reconstructed road sections in excellent state after two treatments with thin layers;
$\alpha_{20,k}^i$ = the contribution of new, strengthened, and reconstructed road sections without surface treatment in good state;
$\alpha_{21,k}^i$ = the contribution of new, strengthened, and reconstructed road sections with one thin layer in good state;
$\alpha_{3,k}^i$ = the contribution of roads in fair state; and
$\alpha_{4,k}^i$ = the contribution of roads in poor state.

Pavement condition classes are delimited by values of any index or group of indicators that serve as standards for particular types of interventions.

Another group of *s* vectors describes the percentile distribution of road length, in particular age classes with an increment of 1 year, separately for originally constructed pavement and separately for every type of improvement (Figure 1).

There are three types of interventions aimed at bringing the pavement into excellent condition:

- Improvement from good to excellent condition by applying surface treatment or thin layers (whose minimum and maximum depths depend on traffic load class or constructibility);
- Improvement from fair to excellent condition by rehabilitation; and
- Improvement from poor to excellent condition by reconstruction.

Markov processes are used to forecast pavement deterioration on the entire network. As there are only four pavement condition categories for which excellent condition comprised a long period on the rating plot, inhomogeneous chains were chosen. To treat deterioration and repair as parts of a unique process, controlled chains

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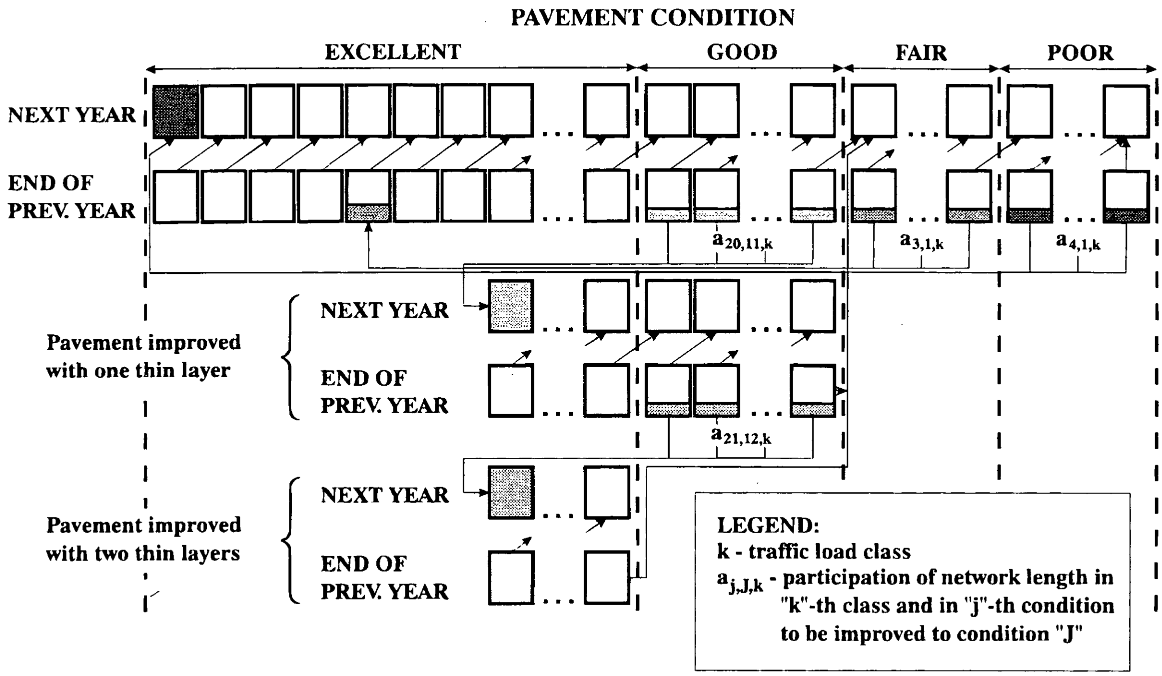

FIGURE 1 Calculation of transition probabilities based on age vectors.

were adopted. The probabilities needed for the transition matrix are calculated for every year by summing the corresponding classes of the age vectors. There is no obstacle to introducing semi-Markov processes if the required data are available. Six strategies concerning priorities were considered. Four of them were with fixed priorities, and two were with optimal choice according to investor's and users' points of view. The quantity of repair work was determined in the frame of the given equal or unequal amounts of annual budget. Six strategies were considered:

1. *Best-first*—sections in good condition are improved first followed by sections in fair and poor condition;
2. *Proportional*—the length of repaired roads is in proportion to their contribution in particular condition classes;
3. *Involving standards*—the part of the road length under the allowed quality standards is improved first, the remaining budget is spent according to the best-first strategy;
4. *Worst-first*—sections in poor condition are improved to excellent condition, followed by sections in fair condition, and finally sections in good condition;
5. *Optimal investor's*—the priority sequence is defined by optimization according to the "minimal backlog" criterion, where backlog represents the total needed to bring the entire network into excellent condition in 1 year;
6. *Optimal users'*—the priority sequence is defined by optimization according to the criterion of minimal extra users' costs caused by imperfect pavement condition.

The second and the third strategy have only practical meaning, so only the results of the first, fourth, fifth, and sixth are presented in the paper.

Three different indicators of effects for every strategy mentioned above have been considered.

Backlog

Backlog represents the total needed to improve the entire network to an excellent condition. The general idea of backlog as a measure of effects was taken from Bates's pavement management forecasting (PMF) model (*5*). Compared with the other criteria used in the network-level optimization systems, backlog has some advantages that must be respected:

- It is easily understandable to decision makers who are not experts in highway engineering;
- Savings in backlog can be discounted during the analysis period and added to other types of savings (i.e., the profitability indicators may be calculated); and
- It is very convenient for the optimization process.

Extra Users' Costs

Extra users' costs are additional costs caused by nonperfect condition of pavement surface. The basic target of any traffic network improvement is to lower transportation costs. We also must be aware that these costs are dispersed on millions of users and automatically incorporated in the price of every article, while it is extremely difficult for the road manager to generate the money from different sources. This is probably why vehicle operating costs are not included in many pavement management systems. They exist mostly in the systems developed by the World Bank as well as several European countries, such as Norway and Finland (*6*). In 1987, the Organization for Economic Cooperation and Development (OECD) Scientific Expert Group for Pavement Management (*7*) "recognize[d] that user costs are an important factor in economic analysis, [but] it is common practice to exclude all or part of these costs from decision-making models for the following reasons:

—Precise quantitative data [are] not presently available
—Extremely high relative level of these costs may lead to maintenance norms being selected that are incompatible with budget constraints . . .
—Reduction in users' costs does not necessarily lead to an increase in the funds available for maintenance . . . .

"The . . . serious omission in most existing pavement management methods is their apparent failure to specify a quantifiable statement of goals and objectives that compares the positive and negative impacts of pavement states, intervention levels, and technique on all concerned parties—i.e., highway authorities, users and community at large. . . . Particularly important is the lack of consideration given to quantifying the impacts on users' costs of pavement management decisions."

We had in mind several facts when deciding to introduce these costs. (a) Extensive investigations have been performed in this field from the time this report was written and their results were successfully implemented. (b) The definition of total life-cycle costs of a highway project in European and World Bank contractor countries comprises costs of investment (initial construction), maintenance (routine maintenance plus reinvestment, i.e., rehabilitation), users' costs (time, operating, discomfort), and social costs (traffic disruption, accident and environmental costs). Different countries use different numbers for these particular costs according to the extent and accuracy of their data banks. (c) The criteria for the network-level management must be as close as needed to the project level if we consider both as stages and accuracy levels of a unique process. (d) These costs may be decisive by choosing the sections when all the other effects are equal.

We neglected some redistribution of traffic caused by improvement of pavement quality and assumed that only changes in costs due to changes in roughness and slipperiness are decisive for the rehabilitation strategies. Thus, we calculated the increase in vehicle operating costs related to the costs on a harsh and even pavement by means of vehicle operating cost (VOC) Module 4 of HDM-III (*8*). Using only the additional users' costs, we hope to overcome an eventual error caused by the inconvenience of the VOC module for the saturated traffic flows that we also considered.

Extra Routine Maintenance Costs

Occasionally, additional expenditures related to the costs of the routine maintenance of excellent pavement are needed to provide the viability without improving its condition. The proportion between routine maintenance costs for pavements in particular condition categories is almost the same as that between corresponding rehabilitation costs, so the priority sequence is the same as when backlog is an optimization criterion. How quickly pavement deteriorates depends largely on the routine maintenance level, but no quantification of such relationships was available to us. So they are only one of the effects considered to enable further economic calculations.

The pavement lifetime spent in a particular condition, as well as the service life of repair measures (i.e., pavement performance curves), may be defined by the user. Any deterioration model may be adopted in such a way. The data for pavement lifetime in initial considerations were taken from Bates's PMF model (*5*). The AASHTO and HDM-III curves were included later. The effects shown in this paper are calculated according to PMF data because they correspond to the asphalt concrete HDM-III curve for regional factor cca 0.65, which is not far from the recommendations for our region. The substantial differences we found between the AASHTO and HDM-III performance curves adapted for the same year of failure with the aforementioned regional factor (using the Sayers correlations for roughness) represent two facts: (a) longer service life in excellent condition and (b) some slower deterioration of pavement structure for minor traffic volume, both in the HDM-III model.

Knowing that according to the PMF model the "best-first" strategy was always the best, we searched a set of input data that would possibly give some other priority sequence from any point of view. Thus, we adopted a few hypothetical combinations that represent only a frame in which data could appear. In reality, the highest contributions usually make good pavements.

The combinations adapted are as follows:

- Initial general network condition
  —Good: 65% excel. 20% good 10% fair 5% poor pavement
  —Fair: 25% excel. 25% good 25% fair 25% poor pavement
  —Poor: 5% excel. 10% good 20% fair 65% poor pavement
- Pavement performance curve expressed as a length of service periods spent in particular conditions
  —PMF
  —AASHTO
  —HDM-III ($m = 0.65$)
- Funding levels of $1,200, $2,400, $3,600, $4,800, and $6,000/km/year. For a funding level of $2,400/km/year the following alternatives were considered:
  —$2,400 $/km/year
  —$4,800/km every second year
  —$7,200/km every third year

Backlog functions as a consequence of different pavement rehabilitation strategies on the poor network are presented in Figure 2. The optimal users' strategy is identical to the best-first strategy; optimal investor's provides slightly better results. The step-by-step

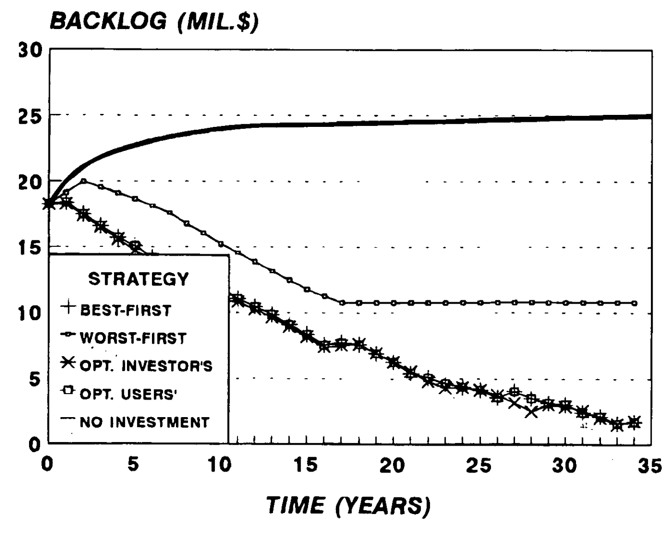

FIGURE 2 Backlog for $3,600/km/year budget as a function of different strategy implementations.

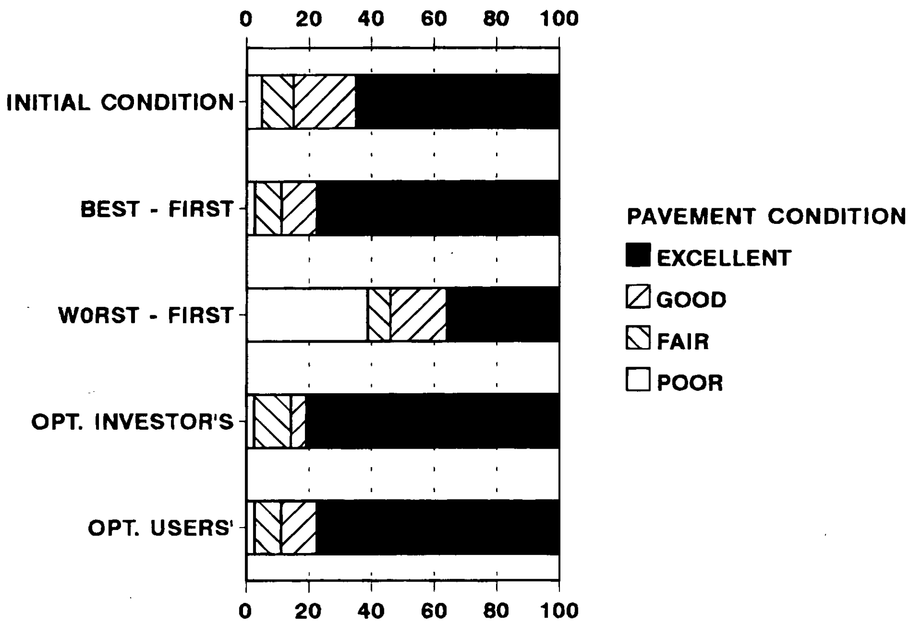

FIGURE 3 Pavement condition for $3,600/km/year budget as a function of different strategy implementations.

analysis showed that sections in fair condition are given first priority under the investor's strategy. Though backlog is almost the same for the three better strategies, pavement condition obtained by those strategies is quite different (Figure 3).

Figures 4 and 5 show backlog and extra users' costs, respectively, depending on strategy and initial network condition. The shape and general orientation of both effects are the same. Whatever the initial network condition, after a long enough period they will have the same values depending only on the budget level. The ninth year of the good network function is interesting for managers; a serious investment must be made at this time. The effects of different budgeting levels are presented in Figure 6.

Based on results of strategy comparison for one-step construction, several general conclusions were reached.

- The effects of the best-first, optimal investor's, and optimal users' strategies are very close; the differences are under the level of accuracy for the model itself.
- The effects of the worst-first strategy are much worse than the effects of the other three strategies (Figures 2 and 3), and the steady

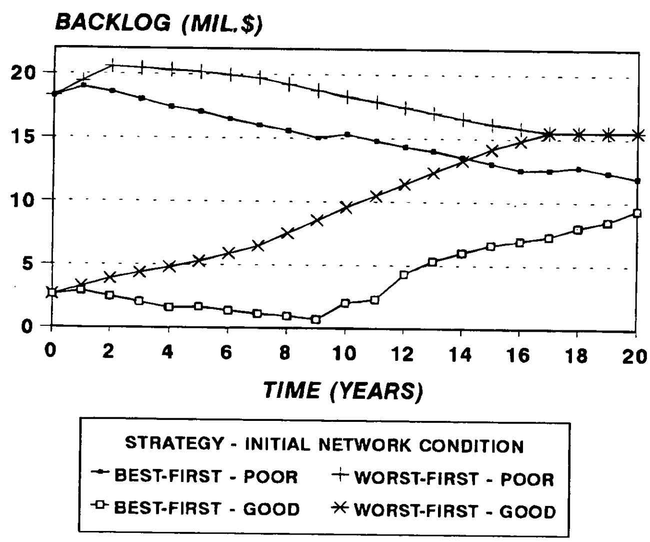

FIGURE 4 Backlog for $2,400/km/year budget and different initial network conditions.

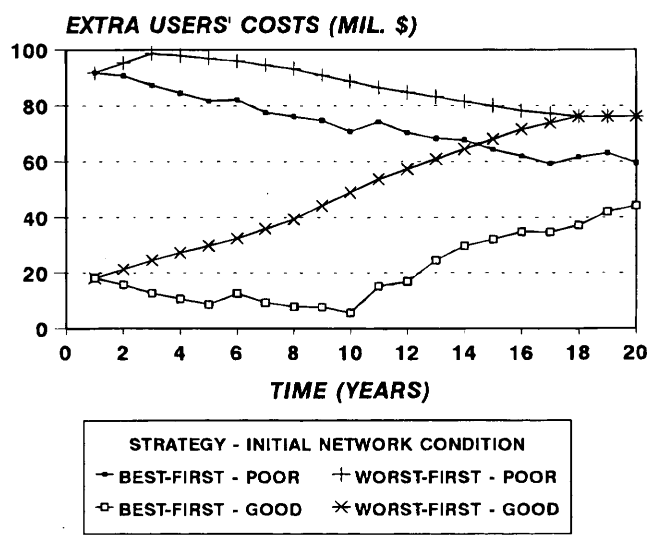

FIGURE 5 Extra users' costs for $2,400/km/year budget and different initial network conditions.

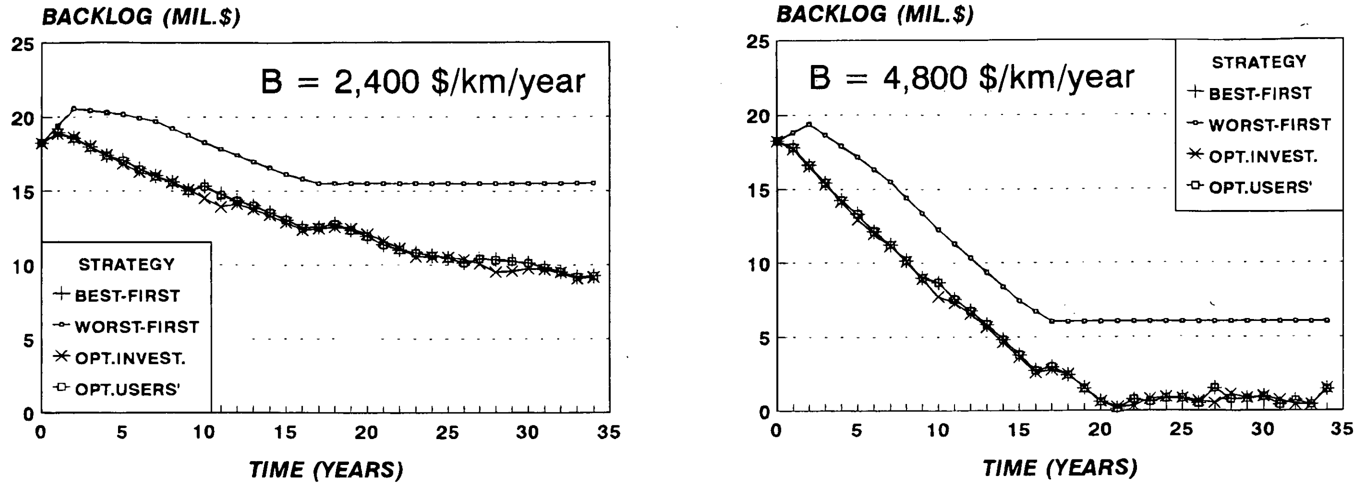

FIGURE 6 Influence of budget level on poor initial network condition.

state begins after the service life of "the youngest" pavement has expired.

- The initial network state has no great influence on the state after 10 years; the available resources are decisive (Figures 4–6).
- Backlog is the most favorable criterion.

It must be pointed out that in the steady state process, no improvement can be expected without additional resources.

ADAPTATION OF MODEL TO STAGED CONSTRUCTION

Two types of questions dealing with the staged construction must be answered:

- Are the effects of staged construction better or worse than those of one-step construction when considered at the network level? and
- Have the strategies been numbered before the same priority sequence as in one-step construction?

Staged construction is more difficult to define at the network level than at the project level. In this study, staged construction was defined as *raising the quality level in two steps. In the first step, all the segments in fair and poor condition are brought into good condition by repair measures with shorter service lives; in the second step, they are brought from good to excellent condition by measures that substantially prolong the pavement lifetime*. For this purpose, the network state vector and the transition matrix were enlarged with the separate class ($\alpha^i_{13,k}$) for the first step interventions. A limitation was imposed so that the improvement from good to excellent condition could be performed after all the other repairs had been accomplished. In the meantime, all sections deteriorate according to the defined functions and are subjected to routine maintenance measures. The transition matrix appears as follows:

$$\begin{bmatrix}
P^i_{10,k} & & & & 1-P^i_{11,k} & & & \\
 & P^i_{11,k} & & & 1-P^i_{11,k} & & & \\
 & & P^i_{12,k} & & & & 1-P^i_{12,k} & \\
a^i_{13,k} & & & P^i_{13,k}(1-a^i_{13,k}) & (1-P^i_{13,k})(1-a^i_{13,k}) & & & \\
 & a^i_{20,k} & & & P^i_{20,k})(1-a^i_{20,k}) & & (1-P^i_{20,k})(1-a^i_{20,k}) & \\
 & & a^i_{21,k} & & & P^i_{21,k})(1-a^i_{21,k}) & (1-P^i_{21,k})(1-a^i_{21,k}) & \\
 & & & a^i_{3,k} & & & P^i_{3,k})(1-a^i_{3,k}) & (1-P^i_{3,k})(1-a^i_{3,k}) \\
 & & & a^i_{4,k} & & & & 1-a^i_{4,k}
\end{bmatrix} \tag{2}$$

The priority of repairs in the optimal investor's strategy is determined by the sequence of $FI^{SC}(a^i_{j,J,k})$ magnitudes, where $a^i_{j,J,k}$ represents the percentage of roads with traffic load class k in condition j to be repaired to condition J in year i, and where SC signifies staged construction. These magnitudes depend only on actual probability of changing the condition and construction prices:

$$\begin{aligned}
FI^{SC}(a^i_{20,11,k}) &= \frac{G_{3,1,k}}{G_{2,1,k}} - p^i_{20,k}\left(\frac{G_{3,1,k}}{G_{2,1,k}} - 1\right) \\
FI^{SC}(a^i_{21,12,k}) &= \frac{G_{3,1,k}}{G_{2,1,k}} - p^i_{21,k}\left(\frac{G_{3,1,k}}{G_{2,1,k}} - 1\right) \\
FI^{SC}(a^i_{3,13,k}) &= \frac{G_{4,1,k} - G_{2,1,k}}{G_{3,13,k}} - p^i_{3,k}\,\frac{G_{4,1,k} - G_{3,1,k}}{G_{3,13,k}} \\
FI^{SC}(a^i_{4,13,k}) &= \frac{G_{4,1,k} - G_{4,2,k}}{G_{4,13,k}}
\end{aligned} \tag{3}$$

where: $G_{jJ,k}$ are construction costs of improvement of 1 km of road in class k from condition j to condition J.

The smaller the probability of staying in actual condition, the higher the position on the priority list for improvement. This position is independent of the amount of resources, and the highway manager must adapt the length of segments to be repaired to the obtained a values. This is the basic difference from the so-called priority assessment models, in which a project with a very high rank may be rejected because the segment was too long.

The priority of repairs in the optimal users' strategy is defined similarly to the optimal investor's strategy. It is determined by the sequence of $FU^{SC}(a^i_{j,J,k})$ magnitudes.

$$FU^{SC}(a^i_{20,11,k}) = Q^i_k \frac{T_{3,k} - T_{1,k} - p^i_{20,k}(T_{3,k} - T_{2,k})}{G_{2,1,k}}$$

$$FU^{SC}(a^i_{21,12,k}) = Q^i_k \frac{T_{3,k} - T_{1,k} - p^i_{21,k}(T_{3,k} - T_{2,k})}{G_{2,1,k}}$$

$$FU^{SC}(a^i_{3,13,k}) = Q^i_k \frac{T_{4,k} - T_{2,k} - p^i_{3,k}(T_{4,k} - T_{3,k})}{G_{3,13,k}}$$

$$FU^{SC}(a^i_{4,13,k}) = Q^i_k \frac{T_{4,k} - T_{2,k}}{G_{4,13,k}} \tag{4}$$

where

Q^i_k = mean AADT*365 in the ith year on the road in the kth class; and

$T_{j,k}$ = vehicle operating costs per vehicle kilometer for the traffic composition on roads in the kth class and jth condition.

These relations show that the priority of intervention in the optimal users' strategy depends on the traffic volume and the ratio *(operating costs)/(construction costs)*. The results for two initial network conditions and for the annual budget of $2,400/km for a two-lane road are presented in Figures 7 to 14.

Figures 7 and 8 show backlog for one-step and staged construction, respectively, whereas Figures 9 and 10 exhibit extra users' costs for the same scenarios. Initially, the network is assumed to be in poor condition. Figures 11–14 are similar to Figures 7–10; the difference is that the assumed initial condition of the network is good. Though the backlog and extra users' costs are almost the same for both types of construction, general network condition differs considerably (Figure 15).

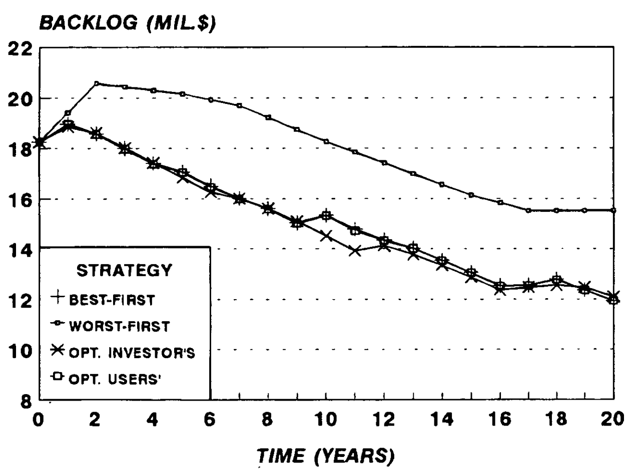

FIGURE 7 Backlog for one-step construction with $2,400/km/year budget and poor initial network condition.

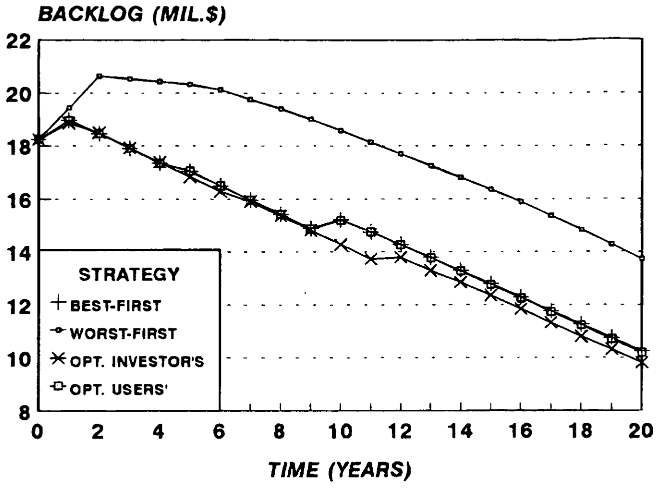

FIGURE 8 Backlog for staged construction with $2,400/km/year budget and poor initial network condition.

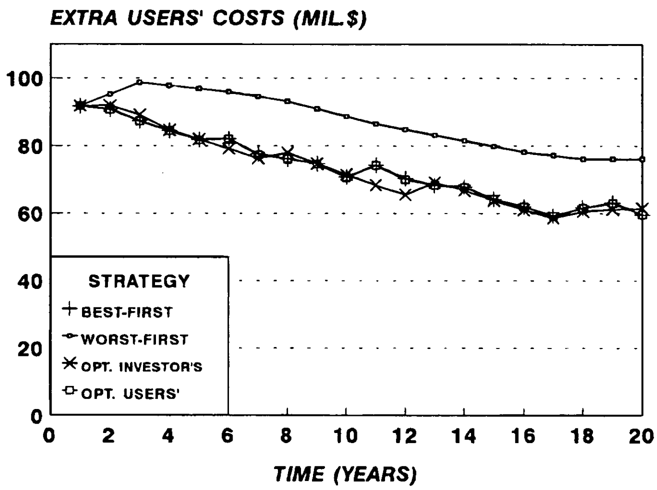

FIGURE 9 Extra users' costs for one-step construction with $2,400/km/year budget and poor initial network condition.

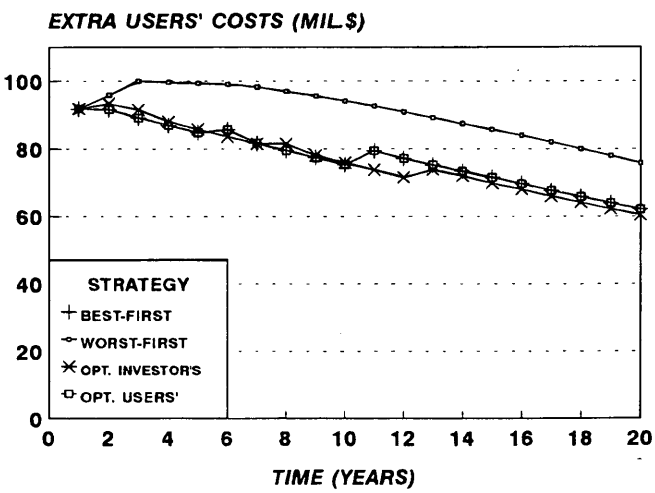

FIGURE 10 Extra users' costs for staged construction with $2,400/km/year budget and poor initial network condition.

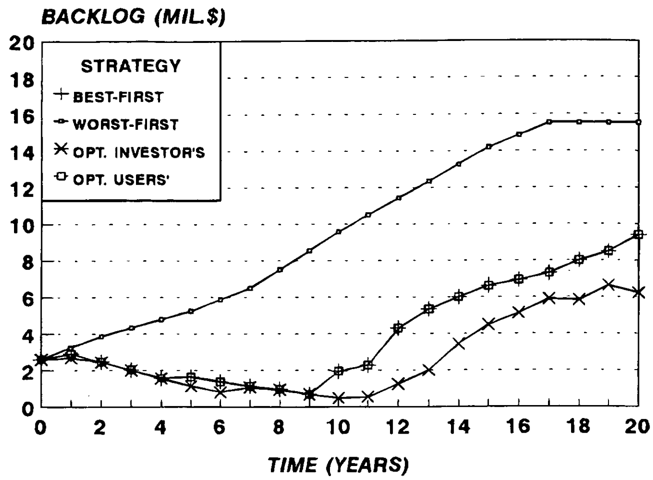

FIGURE 11 Backlog for one-step construction with $2,400/km/year budget and good initial network condition.

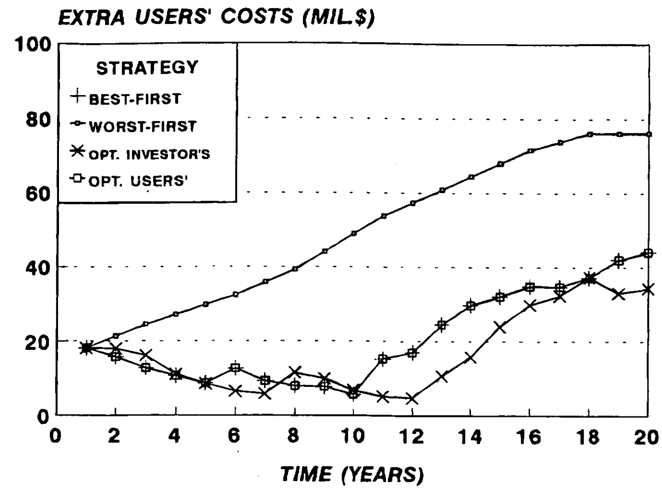

FIGURE 13 Extra users' costs for one-step construction with $2,400/km/year budget and good initial network condition.

FINDINGS AND CONCLUSION

Based on the analysis presented, it could be concluded that in the case of staged construction, the basic relation between worst-first and other strategies, obtained for one-step construction, is not disturbed. Staged construction produces neither exceptional savings nor extra costs during the first 15 years. After that, such construction seems to be even more favorable.

Important effects of pavement improvement may be expected in incidence of fewer accidents. The highest risk is usually recorded on slippery, but not very rough, pavements that belong to the "good" pavement category. So, the greatest benefits in safety may appear as a consequence of surface treatment or even some routine maintenance treatments. This fact supports the best-first strategy but does not influence the relation between one-step and staged construction because it concerns only the layer's lifetime. No appropriate accident data base was available to provide a true picture of these relations. Thus, the entire segment of accident costs has not yet been introduced. Some other facts dealing with long-term thin and thick layer's performance could not be incorporated before a precise calibration. Though more refined data and a more detailed analysis are needed for such investigations, a general conclusion may be drawn from the results: If no great savings in future investments can be expected when applying staged construction, there are good reasons to introduce high standards for capital maintenance immediately.

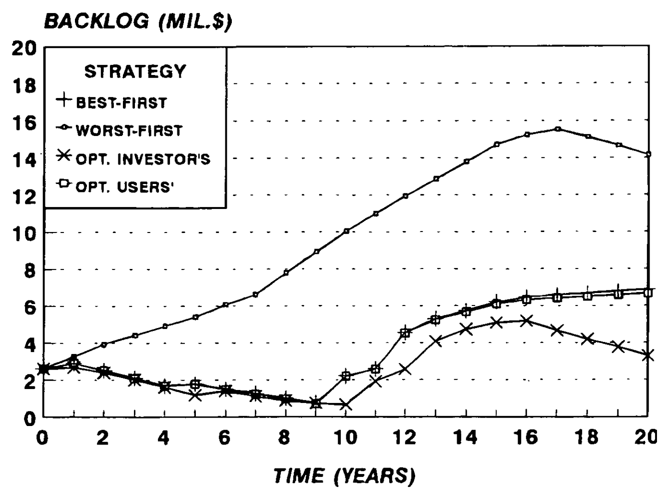

FIGURE 12 Backlog for staged construction with $2,400/km year budget and good initial network condition.

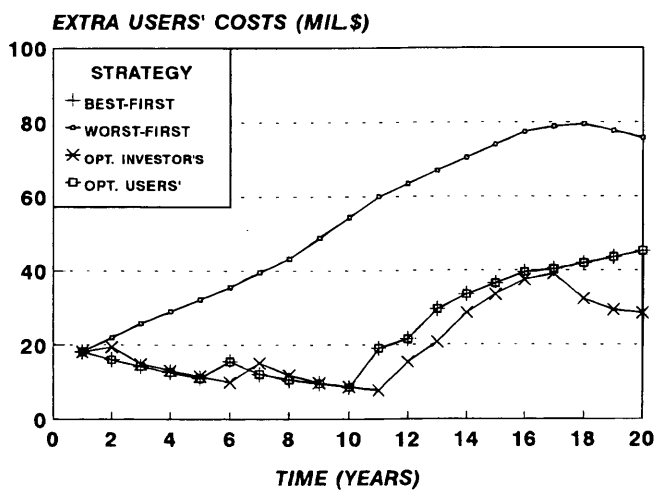

FIGURE 14 Extra users' costs for staged construction with $2,400/km/year budget and good initial network condition.

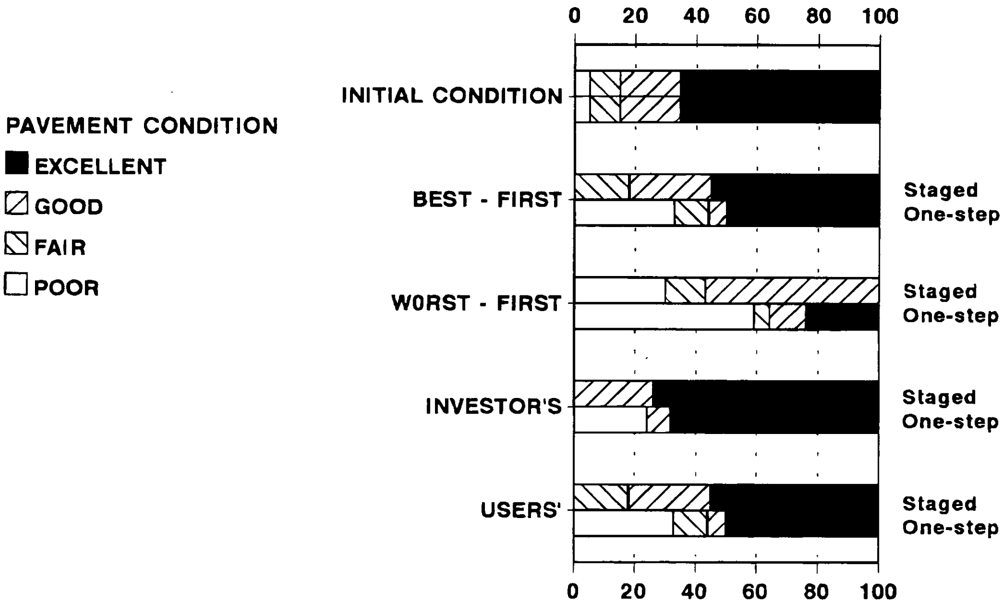

FIGURE 15 Pavement condition after 20 years for $2,400/km/year budget and good initial network condition.

REFERENCES

1. *AASHTO Guidelines for Pavement Management Systems.* American Association of State Highway and Transportation Officials, Washington D.C., 1990.
2. Nowak, E. C., Jr., and W. H. Kuo. Role of Pavement Management System Analysis in Preservation Program Development. In *Transportation Research Record 1344,* TRB, National Research Council, Washington, D.C., 1992, pp. 1–8.
3. Mijuskovic, V., D. Banjevic, and G. Mladenovic. The Results of Prioritization Depending on Different Economic Criteria in the Field of Road Network Maintenance Management Systems. *Proc., International Symposium on Economic Evaluation and Built Environment.* CIB W55/W95. Lisbon, Portugal, Vol. 3, 1993, pp. 50–61.
4. Mijuskovic, V., D. Banjevic, and G. Mladenovic. The Priority of Road Network Maintenance Strategies Defined as Guiding Rules. *Proc., IRF Regional Conference—Roads to 21st Century: A Key to Competitiveness.* Calgary, Alberta, Canada, Vol. 3, 1994, pp. E67–E87.
5. Bates, E. G., Jr., et al. *Pavement Management Forecasting Model.* Boston Metropolitan Area Planning Council, McTrans, Boston, Mass. 1987.
6. Haugodegard, T., J. M. Johansen, D. Bertelsen, and K. Gabestad. Norwegian Public Roads Administration: A Complete Pavement Management System in Operation. *Proc., 3rd International Conference on Managing Pavements,* San Antonio, Tex., Vol. 2, TRB, Washington D.C., 1994, pp. 25–33.
7. *Pavement Management Systems, Road Transport Research.* OECD Scientific Expert Group, 1987, pp. 57, 127.
8. Archondo, R., and A. Dareshwar. *Vehicle Operating Cost Model, Part of Highway Design and Maintenance Model (HDM-III).* World Bank, Washington, D.C., 1986.

Virginia Aviation Pavement Management System: A Historical Perspective

MARGARET BROTEN AND STEVEN MCNEELY

The use of pavement management systems for individual airports and systems of airports is a relatively recent application compared to roadway pavement management system implementations. Several papers have been published that discuss the initial development and implementation of an airport pavement management system. However, published information evaluating the usefulness and success of in-place airport pavement management systems is scarce. This dearth of documentation has contributed to the slow acceptance of pavement management practices for aviation applications. The Virginia Department of Aviation (VDOA) initiated its pavement management efforts in 1990. VDOA uses its pavement management system to prioritize and schedule pavement maintenance and rehabilitation activities. The use of the system has contributed significantly to the increase in the overall condition of Virginia's paved runways from an area-weighted pavement condition index (PCI) of 76 in 1990 to a PCI of 84 in 1993. Use of the pavement management system permitted VDOA to obtain this increase in condition without having to obtain a substantial increase in funding. In addition, the PMS is expected to fully satisfy the requirements of the 1994 Airport Improvement Program reauthorization legislation pertaining to airport pavement maintenance management. A case study of a state aviation pavement management system is presented. The implementation process and the impact of the system on the selection of pavement maintenance and rehabilitation projects are documented. Further, condition and budget information prior to, and after, full implementation and use of the pavement management system is presented.

Since 1928, the Commonwealth of Virginia has taken an active role in the development of an efficient statewide air transportation system. Today, Virginia's air transportation system consists of 70 public-use airports and 1 heliport. Aviation is very important to the commonwealth, because airports link Virginia with commercial markets, serve as gateways for tourism, generate $1.5 billion in wages, and contribute $7.2 billion to the economic activity of Virginia. These airports will continue to play a vital role in the economic health of the commonwealth well into the 21st century, and the protection of their physical infrastructure is of critical importance.

Prior to implementing a pavement management system (PMS), the Virginia Department of Aviation (VDOA) relied heavily on each airport sponsor's knowledge of project justification when pavement-related funding was requested. Often, these sponsors did not have experience in identifying pavement-related needs at airport facilities. Historically, this lack of experience led to problems. Some projects that could have been delayed were funded, whereas other projects that were needed went unrecognized and unfunded.

As a result of a commitment and responsibility to maintain a safe and efficient system of air transportation, and to protect the large capital investment that the airports represented, Virginia implemented a PMS to enable the VDOA to assist sponsors in managing airport pavements. It is used to store, analyze, and retrieve pavement condition data. The PMS permits the department to efficiently monitor pavement condition, correct airport deficiencies, and take advantage of limited budgets. In addition, the program facilitates the development of annual maintenance plans and the preparation of long-term (5- to 20-year) capital improvement programs (CIPs).

VDOA began the process of implementing a PMS in 1990 (*1*). The first step in the implementation process involved determining the current condition status of the pavements at selected airports, as measured by the pavement condition index (PCI) (*2*). In 1990, the overall area-weighted PCI of Virginia's airport pavement network (runways, taxiways, and aprons) was 76. The area-weighted PCI for the runways was also 76. In 1993, after the PMS had been fully implemented and in use for 3 years, the area-weighted PCI for the entire pavement network had increased to 82, and the runway area-weighted PCI had increased to 84. VDOA attributes much of this improvement in pavement condition, obtained without a significant increase in pavement-related expenditures, to its use of its PMS during the past 4 years.

The objective of this paper is to provide a historical perspective of the implementation and long-term use of a PMS for the management of a state aviation pavement network. The paper documents the implementation process and the impact the system has made on the selection of pavement maintenance and rehabilitation projects. Finally, the paper presents condition and budget information before and after full implementation and use of the PMS. Because very little published information is available on the actual results of using an airport PMS, it is hoped this paper will partially fill the void that has contributed to the relatively slow acceptance of pavement management practices for aviation applications.

MANAGEMENT OF PAVEMENTS BEFORE IMPLEMENTATION OF PMS

Prior to 1990, VDOA did not have a pavement management system. Pavement-related projects were typically undertaken when requested by an airport sponsor. Due to a lack of current and accurate pavement condition information, VDOA did not have the means to evaluate these requests for appropriateness or to prioritize one request over another. During this period, however, VDOA was able to begin its move toward more proactively managing its airport pavements through the implementation of a highly successful maintenance program (which is still in operation).

Determination of Annual Pavement Project List

Prior to the implementation of a PMS in 1990, VDOA approved the majority of pavement-related projects based upon the airport spon-

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sors' requests. In many cases, these sponsors consisted of some form of a commission or authority in which the members were appointed by their respective governing bodies. The sponsors' lack of experience in pavement-related issues often created problems. In some situations, sponsors did not request funding for projects soon enough, which resulted in much more extensive work being required when the work was finally identified and scheduled. In other cases, work was scheduled sooner than necessary. Because the department had no easy way to review pertinent pavement condition information when evaluating pavement projects, the sponsors' requests were normally granted if the funding was available.

Implementation of Maintenance Program

A very successful pavement-related effort, the VDOA Airport Maintenance Program, was initiated in 1980. The program was started in an effort to discourage airport sponsors from purposely allowing a pavement to deteriorate through lack of timely maintenance. The FAA does not currently provide funds for pavement maintenance activities; rather, it depends upon airport sponsors to fund and conduct these activities. However, the FAA does provide funding for major pavement rehabilitation and reconstruction. For this reason, sponsors often perceive it to be in their best interest to forgo any maintenance activities and wait until the pavement deteriorates to a point where federal funding can be obtained.

VDOA recognized this as a very expensive management approach. Timely application of the appropriate maintenance activities can significantly increase the life of a pavement for a relatively small investment. VDOA is committed to this program, and the amount of funding provided for pavement maintenance through the Airport Maintenance Program has grown steadily over the past 14 years.

Before implementation of the PMS and the periodic visual pavement inspections, the VDOA depended upon the sponsor to request funding from the Airport Maintenance Program. The department did not have the information readily available to evaluate the timing or appropriateness of the type of maintenance proposed by the sponsor. In addition, VDOA had no way of knowing when an airport was in need of pavement maintenance when it was not requested by the sponsor. Further, the department did not collect the historical condition information and work history information necessary to evaluate which maintenance techniques and materials were working well within the commonwealth.

INITIAL IMPLEMENTATION OF VIRGINIA'S AVIATION PMS

In 1990, VDOA contracted with ERES Consultants, Inc. (ERES) for the implementation of a comprehensive PMS for airfield pavement evaluation and management. Sixty-one commercial, reliever, and general aviation airports were included in the initial study. The PMS implemented by the team members was ERES' Decision Support Software (DSS).

The work performed to meet the project objectives included gathering information pertaining to pavement history, defining the pavement network, conducting visual condition surveys, and establishing a PMS data base. Maintenance and repair cost estimates and prioritization schemes were integrated into the PMS software, and computerized maps showing network layouts and condition ranges were developed and linked to the PMS. Individual airport reports were prepared, the PMS installed, and training conducted. The initial project was completed in 1991.

Records Review

Prior to conducting field work, the project team reviewed existing records to determine the pavement structure and age. These records included as-built construction records, airport layout plans, and FAA 5010 Airport Master Records. Local airport officials were contacted to obtain information if the records were incomplete or unavailable. The information collected was used to divide the airport pavements into distinct pavement sections and to identify pavement performance trends on which future maintenance and rehabilitation requirements could be based.

Pavement Network Definition

The next task involved dividing the pavements at each airport into units referred to as facilities, sections, and sample units, according to procedures outlined in FAA Advisory Circular (AC) 150/5380-6. A facility is a single entity that serves a distinct function. For example, a runway is considered a facility because it serves a single function (allowing aircraft to take off and land). On an airfield, a facility typically represents an entire runway, taxiway, or apron.

Because of the disparity of characteristics that can occur throughout a facility, it is further subdivided into units called sections. A section is a portion of the pavement that has uniform construction history, pavement structure, traffic patterns, and condition throughout its entire length or area. Sections are used as a management unit for the selection of potential maintenance and rehabilitation projects. The subdivision of facilities into sections is one of the most important tasks conducted during the implementation of the PMS. The best guideline to use in deciding the location of section breaks is to think of the section as the "repair unit," or a portion of the pavement that will be managed independently and evaluated separately for pavement maintenance and rehabilitation.

During the actual survey, it may be necessary to define additional section divisions if there is a definite change in condition or surface. Pavement sectioning should account for differences that will affect pavement performance over time. On pavements receiving heavy loads, it is important to separate heavily trafficked areas from non-trafficked or lightly trafficked areas, because the deterioration patterns associated with these areas may be markedly different. When defining the pavement sections, it is extremely important to exercise diligence, as poor sectioning can lead to erroneous results. The value of any PCI survey is dependent directly on the successful completion of this task.

Pavement sections are further subdivided into sample units for inspection purposes. FAA AC 150/5380-6 states that "a sample unit for jointed rigid pavement is approximately 20 slabs; a sample unit for flexible pavement is an area of approximately 5,000 square feet" (*2*). To determine an overall assessment of the network pavement condition and to identify sections in need of repair within the planning period, not all sample units need be inspected. A network sampling rate that is acceptable to the agency is normally used. In areas that have experienced rapid deterioration or high traffic volumes, a high-density inspection rate may be recommended. Additionally, localized areas of weakness may be selected for a more comprehensive evaluation.

For Virginia's PMS to work efficiently, some unique identifiers were added to the data base. The facility numbers were designed to assist in identification of the pavement area. The first character is either an A, R, or T (for apron, runway, or taxiway). The second and third characters are used to identify the pavement section. The last two characters represent an airport code that is unique for each airport and is used to avoid duplication of a facility number throughout the 61 airports.

Pavement use, rank, zone, and category were defined for each pavement section. Pavement use refers to the primary function a section serves and is always a runway, taxiway, or apron. A pavement rank of "primary" or "secondary" has been assigned to all taxiway and apron pavements. At airports with multiple runways, runways are identified as either "primary" or "secondary." The Virginia Department of Aviation provided assistance in assigning pavement rank to sections.

Zones are used to separate the individual airports within the data base. The FAA airport designator has been used to define each airport's zone. Finally, categories are used to identify the region of the state in which an airport is located, as well as whether the airport is classified commercial, reliever, or general aviation.

Map Preparation

Maps were prepared for all pavement areas to be inspected and included in the PMS. These maps provide important pavement dimensions and the location of feature, section, and sample unit boundaries. The maps were generated using a computer-aided design (CAD) package, because computer-generated maps are far more flexible and can present greater amounts of information than their conventional, manually drawn counterparts. Furthermore, these programs possess a powerful layering capability. Once a base map is created, layers can be generated that use the base map to show the location of any desired feature, such as lighting and landing systems, drainage structures, and so forth.

Virginia's PMS links each CAD airport map to the data base, thus allowing information stored in the data base and analysis results to be displayed on these maps. As a tool, one of the most important functions of a PMS is to convey pavement needs to the government body that approves funding. The ability to create high-quality maps and graphics with the PMS assists VDOA in communicating pavement-related needs to the FAA, the Board of Aviation, and the public, making it clear to even those unfamiliar with pavements that funding levels can have a dramatic impact on current and future pavement condition and can significantly affect future expenditures.

Pavement Condition Index Survey

The PCI procedure, outlined in FAA AC 150/5380-6 for airfield pavements, and further defined in the ASTM Standard D5340, is used by the aviation industry and the military to assess current airport pavement conditions. The PCI was developed to provide engineers with a numerical value indicating overall pavement condition. The final calculated PCI value is a number from 0 to 100, with 100 representing a pavement in excellent condition.

The airport PCI surveys were conducted using the standardized method outlined in AC 150/5380-6. This manual defines distress types and severities and specifies how to measure and record the distress. Specially trained and highly experienced engineers and engineering technicians were used to complete this task, because accurate condition ratings are imperative for identifying appropriate maintenance and repair alternatives.

It was important to inspect all of the pavements within each airport, including new pavements and those in very poor condition, to establish the rate and cause of deterioration. This information was vital during the development of pavement deterioration curves and during the determination of suitable maintenance and repair alternatives.

The survey crews consisted of two team members. To check the validity of the data collected, the quality assurance approach used was to require that at least 5 percent of the sample units be inspected independently by each inspector. During the PCI survey, 35-mm photographs of each section were taken. These photographs provided an overview of typical conditions and covered any unusual or severe distress identified in the field.

PMS Software Implementation and Data Base Development

All information collected was input into a PMS. An interim software delivery included the Corps of Engineers PMS, Micro PAVER, supplemented by ERES's software. This system was later converted to ERES's PMS software, DSS.

PMS Customization

Once the PMS data base had been established, the system was customized for the department. Deterioration models were established for similar types of pavements based on the results of the PCI field surveys. Maintenance and rehabilitation alternatives for use at the airports, along with associated unit costs, were identified. Decision trees were constructed that defined the situations under which each alternative was applicable. Finally, a prioritization scheme was developed with the department to identify the highest priority pavements for the allocation of available funding. VDOA's engineers were consulted throughout this customization process to ensure that the annual maintenance plans and the long-range rehabilitation programs produced by the PMS actually reflect their management philosophy as it is practiced.

Pavement Condition Prediction

Pavement management involves forecasting needs based on pavement performance predictions. By projecting the rate at which the pavement condition will change over time, a meaningful life-cycle cost analysis can be performed to compare the costs of various maintenance and rehabilitation alternatives. In addition to identifying the most economical repair alternative through condition prediction modeling, the optimal time for applying treatments can be estimated. Typically, the optimal repair time is the point at which a gradual rate of deterioration begins to increase at a much faster rate. It is critical to identify this point in time to avoid higher maintenance and rehabilitation costs caused by excess deterioration.

Many methods for predicting condition are available. DSS uses an advanced modeling technique that involves organizing the pavement network into "families" of pavements that perform in a similar manner (*3*). For example, asphalt pavements that have never received an overlay and are subjected to heavy traffic may be grouped into a family. If the PMS is being implemented for a state, a further separation of families may be based on geographic location. By plotting the condition and age of all pavement sections that fit within a given family description, a curve can be generated that represents the performance trends of that particular family.

A meeting was held during the initial implementation project in 1990 during which decisions pertaining to the customization of the software were made. One of the first steps was to divide the network into families of pavements, which were developed to establish deterioration curves that reflect the actual performance of these pavement types. The families distinguished among pavement use, pavement type, traffic levels, and geographic location. These performance models were revised after the pavements were reinspected in 1993 and more data points were available. The revised performance models are provided in Table 1.

Selection of Feasible Repair Alternatives

Once an acceptable method for predicting performance was in place in the PMS, the next step was to define a rehabilitation decision matrix. DSS permits the user to define feasible rehabilitation treatments. The user sets the condition level at which each treatment is considered feasible, as well as any other factors that would influence the selection of a treatment. The objective of this type of program is to develop an automated version of the thought process used to identify feasible rehabilitation strategies. The analysis program uses this information to determine an optimized and prioritized project list that contains only agency-specific feasible rehabilitation options.

During a meeting with VDOA, the applicability of various rehabilitation types in different situations and to repair different types of pavements was discussed, as were the impacts on condition and typical costs. This step in the customization process ensures that the recommended treatments are directly applicable to the existing techniques used by the department. Table 2 contains the VDOA treatment matrix.

Selection of Most Desirable Repair Alternative

The next analysis routine that a PMS needs to function is one that is used to select a single rehabilitation method from a list of feasible alternatives to repair a given section. DSS uses a benefit-cost analysis that evaluates not only the additional pavement life anticipated by the application of a treatment but also the change in condition provided by that treatment. The result is a benefit-cost ratio that can be used to rank treatments based on their overall cost-effectiveness. DSS allows the selections indicated by the program to be overridden if political or managerial factors prohibit the selection of the recommended treatment, or if projects were already "in the pipeline" prior to implementing the program.

Preparation of Prioritization Scheme

A prioritization scheme was also developed during this project. The priority matrix is used to assist in ranking pavement rehabilitation alternatives according to the practices used by VDOA. This allows the department to weight certain projects more heavily than others based on the pavement section's importance to the Virginia aviation system. Table 3 contains the VDOA prioritization matrix.

Data Analysis

The PMS was used to prepare a multiyear CIP and an annual maintenance program for each airport in the data base. A benefit-cost

TABLE 1 Pavement Performance Models

Pavements Modeled	Mathematical Equation of Curve
AC Aprons at Commercial and Reliever Airports	$-0.00022754\ age^4 + 0.01484700\ age^e - 0.24818000\ age^2 - 1.580\ age + 100$
AAC Pavements at Commercial and Reliever Airports	$-0.00079944\ age^4 + 0.02846000\ age^e - 0.29900000\ age^2 - 0.238\ age + 100$
APC Pavements at Commercial and Reliever Airports	$+0.00014068\ age^4 - 0.01534000\ age^e + 0.42896000\ age^2 - 4.690\ age + 100$
PCC Pavements at Commercial and Reliever Airports	$-0.00016512\ age^4 + 0.00912700\ age^e - 0.12000000\ age^2 - 0.990\ age + 100$
AC Runways at Commercial and Reliever Airports	$-0.00025938\ age^4 + 0.01388200\ age^e - 0.17810000\ age^2 - 1.89\ age + 100$
AC Taxiways at Commercial and Reliever Airports	$-0.00021172\ age^4 + 0.01567000\ age^e - 0.30786000\ age^2 - 1.070\ age + 100$
AAC Aprons at General Aviation Airports	$-0.00062051\ age^4 + 0.02678600\ age^e - 0.40332000\ age^2 - 0.092\ age + 100$
AC Aprons at General Aviation Airports	$-0.00022547\ age^4 + 0.01149400\ age^e - 0.12942000\ age^2 - 2.400\ age + 100$
APC Aprons at General Aviation Airports	$+0.00004399\ age^4 - 0.00450000 age^e + 0.16443000\ age^2 - 4.680\ age + 100$
PCC Pavements at General Aviation Airports	$+0.00000833\ age^4 + 0.00095900\ age^e - 0.1130200\ age^2 - 0.051\ age + 100$
AC Runways at General Aviation Airports	$-0.00033938\ age^4 + 0.01969400\ age^e - 0.2867000\ age^2 - 1.580\ age + 100$
AAC Runways and Taxiways at General Aviation Airports	$-0.00000766\ age^4 + 0.00095000\ age^e - 0.10054000\ age^2 - 0.900\ age + 100$
APC Runways and Taxiways at General Aviation Airports	$+0.00012715\ age^4 - 0.01354800\ age^e + 0.44581000\ age^2 - 6.540\ age + 100$
AC Taxiways at General Aviation Airports	$-0.00022197\ age^4 + 0.00955900\ age^e - 0.04599000\ age^2 - 3.080\ age + 100$

AC = asphalt concrete; PCC = portland cement concrete; APC = asphalt overlay on PCC; AAC = asphalt overlay on AC

TABLE 2 Treatment Matrix

Treatment	PCI	Surface Type	Deducts due to Load	Pavement Use	Unit Cost (sq m)
Slurry and Crack Seal	75 - 90	AC, AAC, APC	≤ 10%	All General Aviation	$0.97
Slurry and Crack Seal	75 - 90	AC, AAC, APC	≤ 10%	Commercial/Reliever Aprons	$0.97
AC Overlay and Seal Coat	40 - 75	All	≤ 50%	Aprons	$11.84
AC Overlay	40 - 80	All	≤ 50%	Runways/Taxiways	$9.47
Mill, Overlay, and Seal Coat	40 - 75	AC, AAC, APC	50 - 75%	Aprons	$11.84
Mill and Overlay	40 - 80	AC, AAC, APC	50 - 75%	Runways/Taxiways	$9.47
Total AC Reconstruction and Seal Coat	0 - 60	All	> 75%	Aprons	$16.75
Total AC Reconstruction	0 - 60	All	> 75%	Runways/Taxiways	$14.32
Total PCC Reconstruction	0 - 60	All	> 75%	All	$22.71
Partial AC Reconstruction and Seal Coat	0 - 40	All	≤ 50%	Aprons	$14.32
Partial AC Reconstruction	0 - 40	All	≤ 50%	Runways/Taxiways	$11.95

AC = asphalt concrete; PCC = portland cement concrete; AAC = asphalt overlay on AC; APC = asphalt overlay on PCC

ratio is determined for each feasible alternative, and the highest ranking ratio is selected as the recommended treatment for that particular section. Benefit is determined as the area between the section's deterioration curve, assuming no repair is done, and the new deterioration curve for the section following repair. Costs are determined on a life-cycle cost basis so that alternatives with differing useful lives can be evaluated on an equal basis. Based on user input budget estimates and the department's prioritization scheme, the ratios are ranked for each of the years in the budget analysis.

The distress data, treatment matrix, priority matrix, maintenance policies, and budget parameters are all used during this analysis. Because the PMS can accommodate multiple treatment, priority, budgeting, and maintenance alternatives, VDOA can quickly and easily analyze different scenarios, such as what effect a reduction in pavement-related funding will have on future pavement condition levels and funding requirements.

Report Preparation

A separate report was prepared for each airport and delivered to VDOA in August 1991. These reports document the work that was accomplished at each airport and present the field survey results. Each airport report also includes a network map, showing the location of all sections and sample units, and a color-coded map showing the pavement condition rating of each section. Color photographs of typical distress types were included in these reports. A summary report presenting the multiyear CIP and annual maintenance program was also prepared.

PMS Installation and Demonstration

The PMS was installed at VDOA. An important consideration in the PMS implementation process is the proper training of the individuals who will be using the system. At the completion of the training process, VDOA personnel had all the skills necessary to operate the program efficiently and effectively. Training included formal sessions that covered topics such as the PCI procedure, PMS concepts, and so forth. More importantly, training was ongoing throughout the implementation process.

Update of Virginia's Aviation PMS

The VDOA PMS was updated during 1993. Sixty airports were reinspected using the PCI procedure. The performance models, treatment matrix, priority matrix, and maintenance policies were revised at that time. The PMS data base and maps were updated and revised; a comprehensive analysis of the collected data was conducted; and reports were prepared. A refresher course in the use of the PMS program was conducted.

The timing of the update was excellent. The 1994 Airport Improvement Program (AIP) reauthorization legislation enacted by Congress has mandated that airport sponsors have a pavement maintenance management program in place as a condition to receiving federal funding for pavement rehabilitation and reconstruction projects. It is expected that the VDOA PMS will fully meet this requirement.

RESULTS OF PMS IMPLEMENTATION

Table 4 summarizes the results of the PCI surveys conducted in 1990 and 1993. As this table indicates, the overall network improved significantly during that time period, with runways showing the most dramatic improvement. Table 5 provides the PCI survey results broken out into pavement condition ranges.

Figure 1 depicts the total expenditures for pavement maintenance made by the department before and after the PMS implementation. A modest increase in maintenance funding has been obtained since the implementation of the PMS. Prior to PMS implementation, an annual average of $244,000 was spent on pavement maintenance. This amount increased to an annual average of $313,000 after the PMS was implemented.

TABLE 3 Prioritization Matrix

Condition Range	Air Carrier/General Aviation Primary Runways	Reliever Primary Runways and General Aviation Primary Taxiways	Air Carrier/Reliever Primary Taxiways and General Aviation Aprons	Air Carrier/Reliever Aprons and General Aviation Secondary Runways	Air Carrier Secondary Runways and General Aviation Secondary Taxiways	Reliever Secondary Runways	Air Carrier/Reliever Secondary Taxiways
Excellent	6	12	18	24	30	36	42
Very Good	5	10	15	20	25	30	35
Good	4	8	12	16	20	24	28
Fair	3	6	9	12	15	18	21
Poor	2	4	6	8	10	12	14
Very Poor/Failed	1	2	3	4	5	6	7

TABLE 4 Area-Weighted PCI Values

Year	Runways	Taxiways	Aprons	Network
1990	76	77	78	76
1993	84	82	80	82

TABLE 5 PCI for Virginia's Airport network in 1990 and 1993

PCI Range	% Area Runways		% Area Taxiways		% Area Aprons	
	1990	1993	1990	1993	1990	1993
86 - 100	42	55	40	49	45	19
71 - 85	30	26	26	28	21	29
56 - 70	11	14	19	11	20	33
41 - 55	10	2	9	3	8	9
26 - 40	5	1	5	8	3	5
11 - 25	2	2	1	1	3	3
0 - 10	0	0	0	0	0	2

Figure 1 also shows the department's total expenditures for pavement rehabilitation projects before and after the PMS implementation. Overall, the expenditures made for pavement rehabilitation prior to the implementation of the PMS ($3,170,250 annually) have remained almost unchanged since the implementation of the PMS ($3,203,000 annually).

Project rehabilitation expenditures initially increased after the implementation of the PMS but declined rapidly beginning in 1994. As a result of the initial study, it was recognized that many pavements did not meet the department's expectations; substantial funding was required in 1992 and 1993 to rehabilitate those pavements. Once those projects were completed, a lower level of funding was needed to maintain the pavement network. This situation is expected to continue as long as timely maintenance continues to be applied at the airports.

VDOA uses the PMS data base and analysis routines to evaluate sponsor requests for maintenance and rehabilitation funding. In several cases during the past 3 years, the PMS helped the department identify inappropriate requests, determine optimal timing of project work, and identify projects that should have been requested but had not been. In one case, an airport sponsor requested a major runway rehabilitation project. Prior to the implementation of the PMS, this request would have been granted if funding was available based primarily on the airport sponsor's justification. Using the PCI data contained in the PMS data base, VDOA was able to determine that the type of deterioration exhibited by the runway could probably be corrected with a less major repair. Further project-level investigation of the runway determined that this was, in fact, the case. In another situation, VDOA was able to use the PMS to identify a runway that required immediate attention, although the airport sponsor had not requested funding for its repair.

Through the use of the system, VDOA is able to better allocate limited resources and assist the sponsors in managing the airport pavements. In addition, VDOA now has a tool to provide objective prioritization of pavement projects. The program allows VDOA to quickly analyze "what if" scenarios to respond to the Board of Aviation's frequent questions about the airport network, such as "What if funding is reduced by 10%?" or "What if that project is delayed for 5 years? What will be the impact on the condition of the pavement due to that delay, and how will it affect feasible repair alternatives at the end of the delay?" Analysis that used to take VDOA many days can now be performed quickly, enabling the department to be more responsive to the FAA, the Board of Aviation, and the public.

SUMMARY

The Virginia Department of Aviation has used a state-of-the-art PMS for the past 4 years. It contains an up-to-date data base and can be easily operated by the department's staff. Through this program, VDOA is able to select specific rehabilitation methods based on

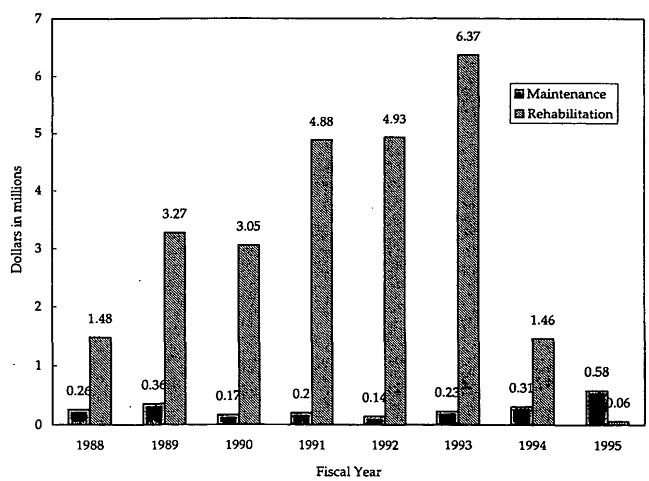

FIGURE 1 Pavement expenditures.

both engineering and economic considerations. In addition, the program helps the commonwealth and the FAA prioritize pavement rehabilitation work.

Through the program, VDOA can demonstrate to the Board of Aviation, the legislature, the FAA, and the public that it is managing the pavements at the public airports in a fiscally responsible manner. Because the system establishes a time frame when rehabilitation work should take place, it permits the better budgeting and allocation of funds. In addition, it enables VDOA to better use its existing Airport Maintenance Program, which provides funds for extending the life of pavements through routine maintenance.

REFERENCES

1. Broten, M. R., and S. McNeely. Integrated Approach to Statewide Airport Management in the Commonwealth of Virginia. In *Transportation Research Record 1423*, TRB, National Research Council, Washington, D.C., 1993, pp. 18–25.
2. *Guidelines and Procedures for Maintenance of Airport Pavements.* FAA Advisory Circular 150/5380-6. FAA, U.S. Department of Transportation, 1982.
3. Shahin, M. Y., M. M. Nunez, M. R. Broten, S. H. Carpenter, and A. Sameh. New Techniques for Modeling Pavement Deterioration. In *Transportation Research Record 1123*, TRB, National Research Council, Washington, D.C., 1987, pp. 40–46.

An Interactive Graphical Pavement Management System: Windows ILLINET

LI WANG, YAN LU, MICHAEL I. DARTER, KATHLEEN T. HALL, AND DAVID L. LIPPERT

How can a pavement management system (PMS) be presented to maximize its usefulness to pavement managers in making rehabilitation decisions? This research developed an interactive, graphical multimedia PMS called Windows ILLINET, which is a user-friendly, Windows-based software. It applies color graphics, text, and digitized video images to display current and historical pavement condition, rehabilitation information, and predicted pavement performance. It also offers multiple decision-making options. Moreover, users can be interactively involved in the decision-making procedure. The results from the research should be very valuable in the development of future PMS software.

The main purpose of developing a pavement management system (PMS) is to create a tool that will assist pavement engineers in making decisions concerning the management of pavement facilities. While any PMS can store vast amounts of data, it is imperative that data be transformed into useful information to aid in making rehabilitation decisions. This paper describes a robust tool called Windows ILLINET, a newly developed graphical, interactive PMS.

Windows ILLINET is a Windows-based system developed for the Illinois Department of Transportation (IDOT) by the Department of Civil Engineering at the University of Illinois at Urbana-Champaign. This system applies color graphics, text, and digitized motion video images to display current and historical pavement condition, rehabilitation information, and predicted pavement performance. It is coded in C++ 4.0 and runs in Microsoft Windows 3.1.

The supporting data base is the Illinois Pavement Feedback System (IPFS). This new PMS has several special features. First, the system can indicate graphically almost all of the information in the data base and the predicted pavement performance. Second, it offers a large number of combinations of network-level and project-level decision-making options. Third, the interactive decision-making option allows users to be involved and apply their own knowledge. If the results generated by the algorithms are not desirable, the users can modify either the parameters in the decision-making procedure or the rehabilitation decisions, based on their own experience. Fourth, this system has a user-friendly interface. Users can easily learn and master it. Many of the concepts were suggested by IDOT personnel. The previous work done by Mohseni was also very helpful (*1–3*).

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GRAPHICAL DATA INTERPRETATION

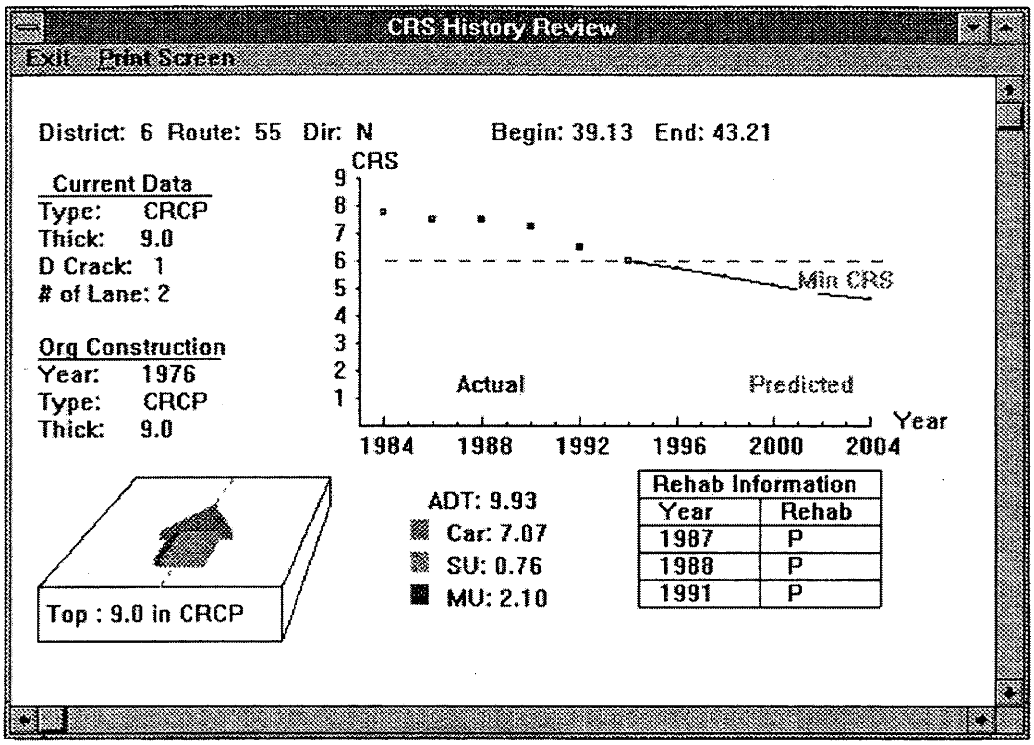
Data must be formatted into understandable information to facilitate any necessary decisions. "Unexplained numbers are not information" (*4*) and cannot be used by decision makers unless the numbers are presented in a meaningful context. To make data more understandable, the numbers need to be analyzed and then interpreted in various formats, such as verbal explanations, tabular summaries, and visual representations, such as graphs.

The key problem to successfully implement the data analysis and interpretation component is being able to quickly access an overwhelming amount of data. As Wildbur has pointed out, people often "mistakenly refer to the information explosion in the world today. In fact, there is no information explosion—it's a numbers explosion, and it falls to designers to turn the numbers into useful information" (*4*). The use of computer graphics can help quickly turn numbers into useful information. Colors, sounds, and motion videos can also be applied to sharpen the images and make the displayed information more easily understandable. All these technologies make "visual thinking" possible. Friedhoff and Benzon stress that visual thinking is "a legitimate and distinctive mode of thinking" and that the relatively effortless processing capabilities of the human visual system are properly exploited by the use of graphics (*5,6*).

Graphical Display of Source Information

The available data and information can be depicted on screen in graphics or digitized images. Audio information can be supplied if available. The pavement condition and traffic histories are displayed in graphics format upon users' request when these history data are available in IPFS. This greatly helps decision makers understand the data stored in the data base. The graphics include a historical pavement Condition Rating Survey (CRS), predicted future CRS, and rehabilitation information for each section. Traffic history data can also be provided on screen, including average daily traffic (ADT), single-unit and multiunit truck volumes, equivalent single axle loads, and growth rate histories of each.

The relationship between the traffic volume and the highway capacity for a specified pavement section can be displayed as well, as indicated by the pavement diagram in the lower left corner of Figure 1. The width of the tail of the arrow shown on the pavement is proportional to the ADT. If the arrow width exceeds the pavement width, the traffic volume exceeds the assumed capacity of 2,000 passenger cars per lane per hour. This occurs for some highway sections in the Chicago area and indicates that the traffic is congested and cannot travel at the speed or with the ease of movement that the



Note: 1 in = 2.54 cm, 1 mile = 1.61 km

FIGURE 1 View history information.

drivers would like. The graphical display of this relationship is easier for IDOT managers to quickly understand than a mere presentation of the traffic volume numbers would be.

Figure 1 also illustrates a graphical display of the past and predicted future condition of the pavement section. The information at the top of the screen identifies the pavement section. Key information about the current and original pavement section is shown at the left. The CRS history and predicted CRS for the specified section over a period of 20 years appear in the graph at the upper right of the screen. To help the user relate the CRS at any time to the CRS level at which rehabilitation is needed, a broken line that represents the CRS value identified by the user as a rehabilitation trigger value is indicated as Min CRS. Traffic information is also provided on this screen, as described above.

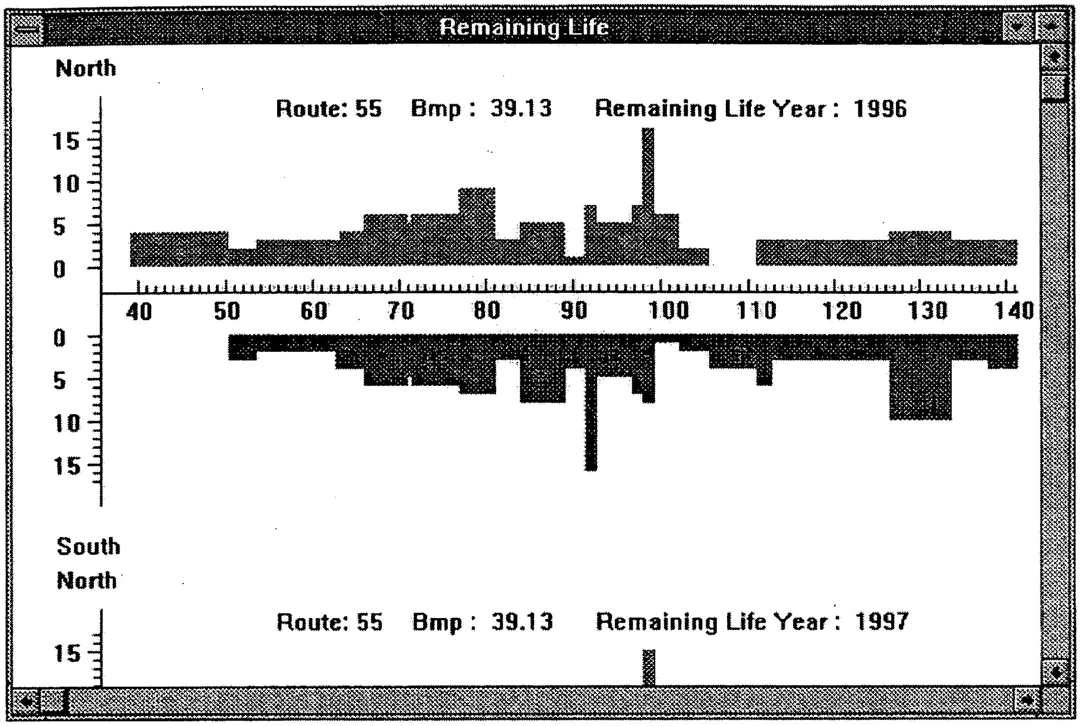
The rehabilitation history information is tabulated at the bottom right corner of the screen. This information helps the user interpret the CRS history curve. More detailed pavement rehabilitation history and construction information are provided on other screens.

Video images have been taken during pavement inspections of Illinois highways and are stored by IDOT. Those images, along with sound, are converted into digitized files that can then be provided on the computer when requested by the user. These digitized video images give decision makers an appreciation for the current pavement condition. It makes users feel as though they are driving along the highway and inspecting the condition. To display PMS video images, an analog/digital capture board is used to digitize the images and sound, and the digitized files are indexed by route number and milepost. The digitized files are then written to CD-ROMs. A compact disc drive is used to search through the files and read the file for a user-specified highway number and milepost to the screen.

Graphical Results

A PMS needs to display the data collected and stored in the data base, as well as the analysis results and predicted pavement performance. Such displays will assist the decision makers who are eager to know the probable consequences of their rehabilitation decisions. The system can display graphically the network analysis results, including quantified benefits, average network CRS, remaining life, budget requirements, and so forth, as well as detailed section-by-section information on predicted pavement performance, remaining life, and rehabilitation plans. Bar graphs and line graphs are used to demonstrate the pavement conditions and rehabilitation plans. Colors are used to illustrate pavement attributes. Also, network rehabilitation strategies and performance can be depicted in map format to make the information vivid. This helps the user interpret the analysis results.

An example of graphically displaying network analysis results is the remaining life screen in Figure 2. The interstate route number and beginning milepost for a selected section in the selected IDOT district are located at the top of the screen. The horizontal axis indicates the mileposts for a length of highway that contains the selected section; the vertical axis indicates the remaining life in years. The upward bars represent the remaining lives of each section in one direction of the highway; the downward bars represent the remaining life of the sections in the other direction. A bar with zero height indicates no remaining life left for that section in that year (i.e., the current or predicted CRS in that year is already less than or equal to the minimum CRS). The user may scroll down this screen to see the remaining life chart for this length of highway for each year in the analysis period. From this graph, users can quickly see and appre-



Note: 1 in = 2.54 cm, 1 mile = 1.61 km

FIGURE 2 Display yearly predicted remaining life.

ciate the remaining life changes for the selected section and how they compare to the remaining lifes of other sections along the route.

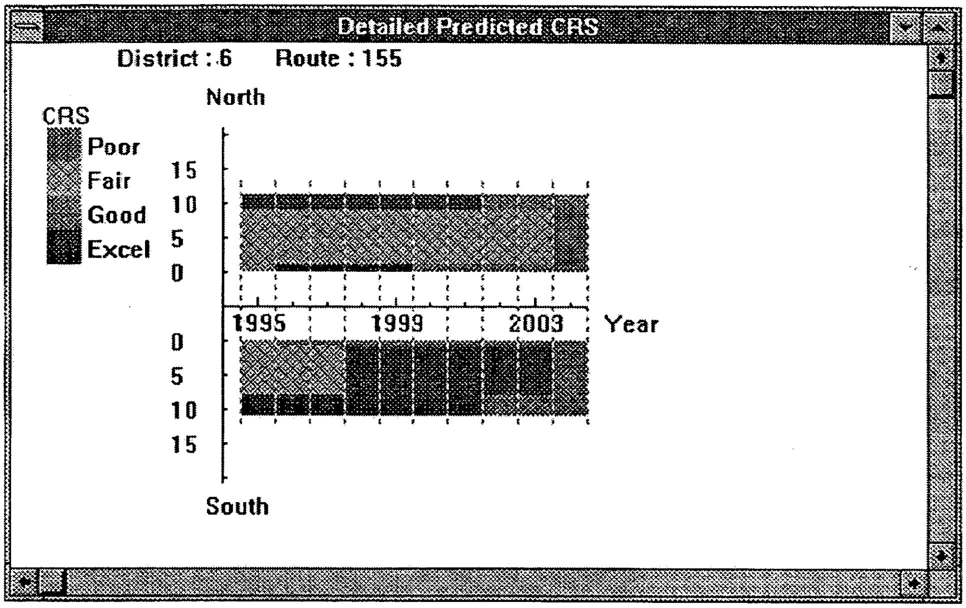
Detailed predicted pavement condition can be depicted in chart as well as map formats. Figure 3 charts the predicted pavement condition by milepost and year. In this chart, the horizontal axis represents time, and the vertical axis represents the milepost along the route. Four colors are used to identify the pavement conditions: "Excellent" in blue, "Good" in green, "Fair" in yellow, and "Poor" in red. (The CRS values that identify these condition categories are defined by IDOT.) These four color identifications are well known by the IDOT central office and district engineers. When the color chart appears on the screen, users can immediately understand the pavement condition situation at specified locations and years. Furthermore, in this chart, from left to right, users can understand the predicted changes in pavement condition through the years for a particular section. Also, from bottom to top, they can find out the pavement condition changes along the route for a specified year.

The yearly network rehabilitation strategy may also be depicted in a map format, as illustrated in Figure 4. The map contains the rehabilitation plan for District 6 in 1995. On this screen, the user can select a pavement section by identifying its route number, direction, and beginning and ending mileposts at the top left part of the screen. The user can also choose the year to be displayed. The middle left part contains the updated pavement information stored in the data base. The bottom left part displays the predicted CRS and rehabilitation plan for the selected section. The right part of the screen contains the network rehabilitation plan in the year selected at the "Year" box (which is 1995). Two red dots on the map indicate the beginning and ending mileposts of the specified section. Different colors are used to indicate different types of rehabilitation. This graph can clearly indicate where, when, and what kind of rehabilitation is planned.

Network statistical analysis results are also depicted in graphs (see Figures 5 through 8). The network summaries are provided in four separate screens: (a) benefit and cost, (b) rehabilitation and cost, (c) condition (CRS), and (d) remaining life. Each screen indicates one or several network-level analysis values. For example, Graph A illustrates the total benefit and total cost of a multiyear rehabilitation plan for one district at a time or for all districts in the state. For the example in Figure 5, the total benefit is 40 091 000.94 veh*km (24,901,000.83 veh*mile) and the total multiyear cost is $67.41 million. The yearly benefit and cost are depicted in two line graphs. To clearly indicate the relationships among benefits, expenditures, and available budget, a table is used.

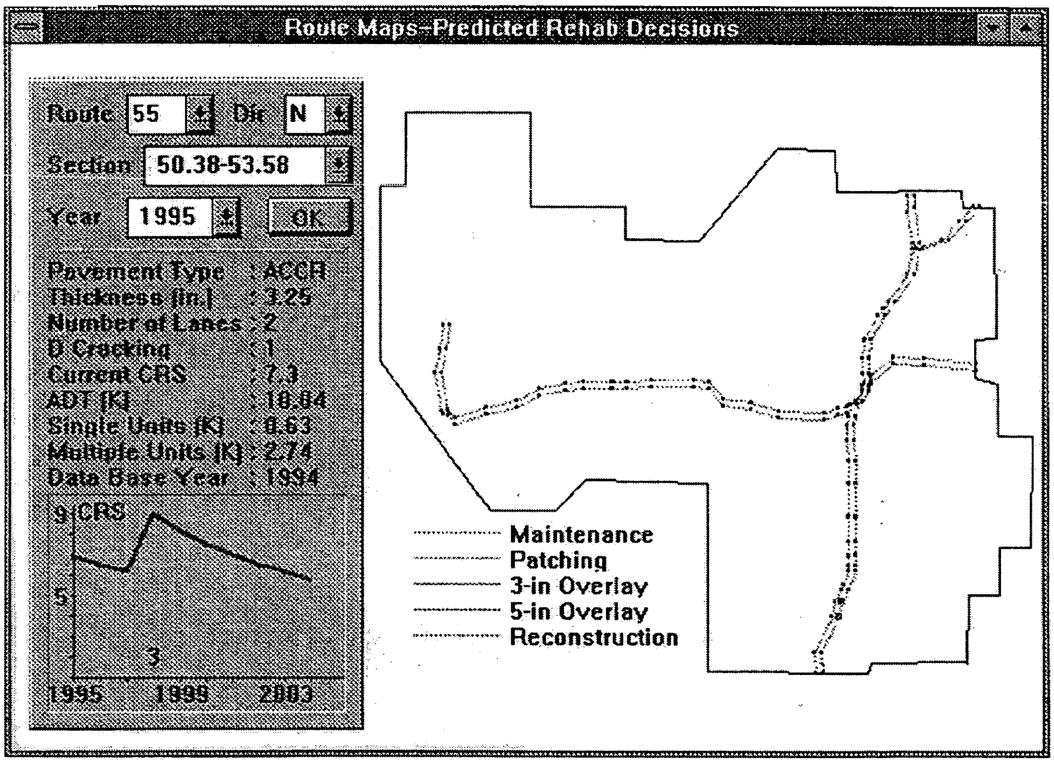
In Figure 6, the upper graph depicts the mileage percentages that should receive various types of rehabilitation (different colors are used to identify different rehabilitation types). The lower graph in Figure 6 presents the total rehabilitation cost each year in millions of dollars. The bars indicate the percentages of the total cost that are allocated for different kinds of rehabilitation in each year. For example, in 1999, approximately $7 million will be spent, of which 12 percent will be used for patching, 66 percent for 7.6-cm (3-in.) overlays, 22 percent for 12.7-cm (5-in.) overlays, and none for reconstruction.

Predicted pavement conditions and remaining lives are illustrated in Figures 7 and 8, respectively. The bar graph in the upper part of Figure 7 depicts the percentages of pavement length that will fall in the Backlog, Accruing, and Adequate categories. The bottom part of the screen indicates the average CRS in each year and the CRS values that encompass 90 percent and 100 percent of the sections. For example, in 1997, 41 percent of the mileage will be in Adequate



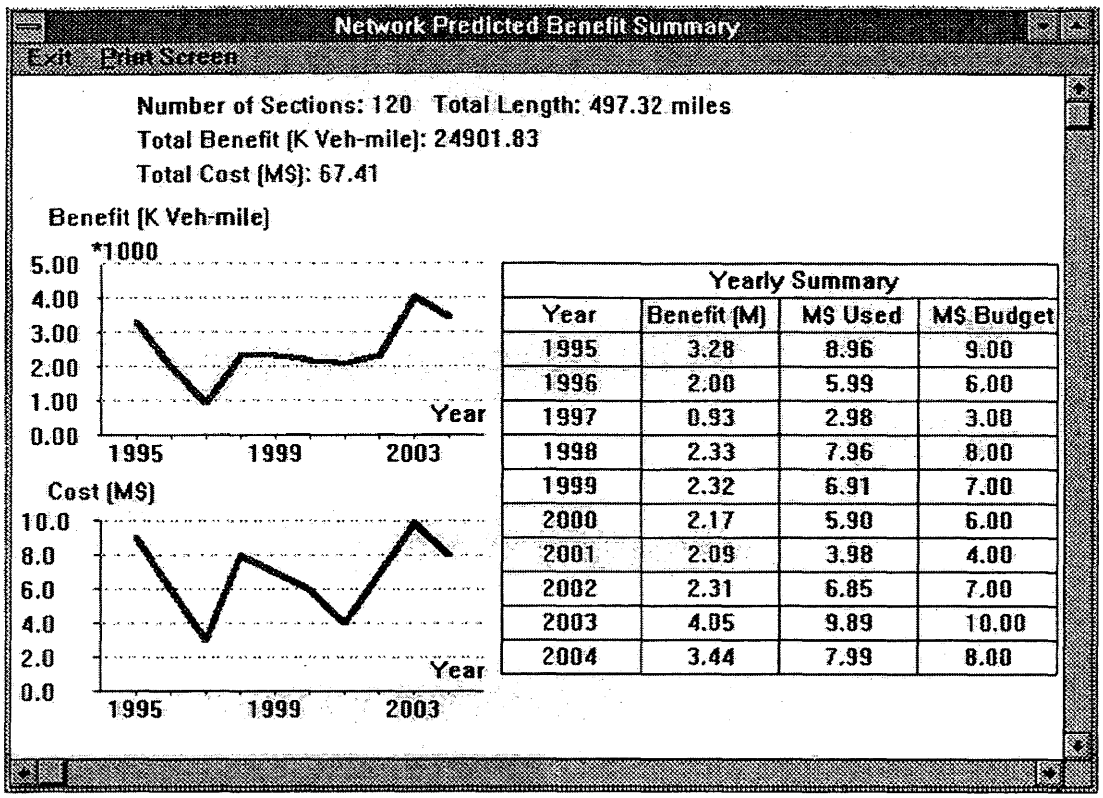
Note: 1 in = 2.54 cm, 1 mile = 1.61 km

FIGURE 3 Detailed predicted pavement condition chart.



Note: 1 in = 2.54 cm, 1 mile = 1.61 km

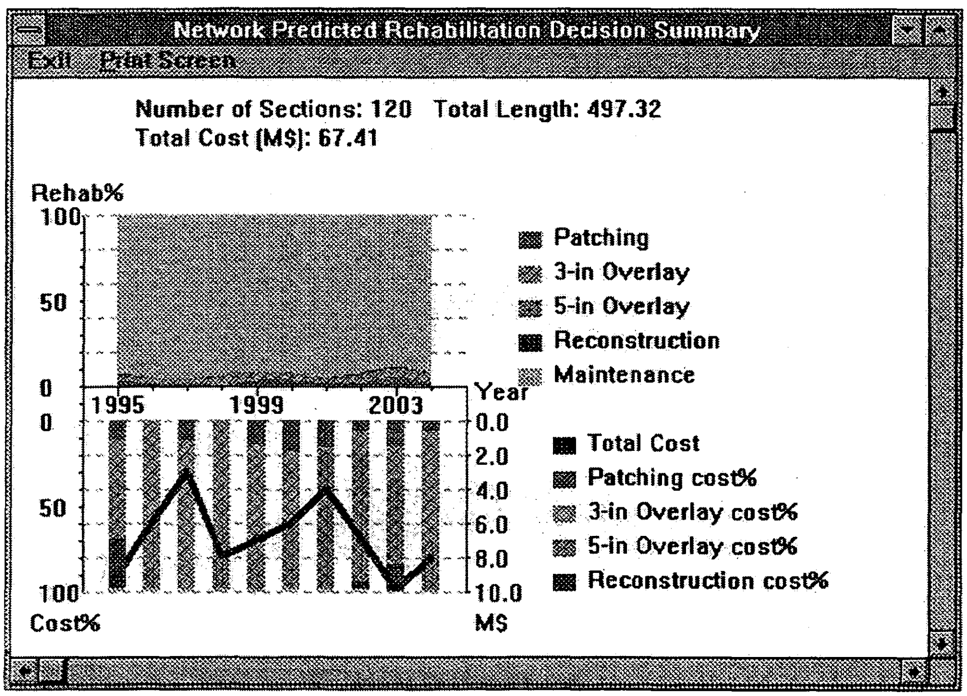
FIGURE 4 Display rehabilitation plan in map format.



Year	Benefit (M)	M$ Used	M$ Budget
1995	3.28	8.96	9.00
1996	2.00	5.99	6.00
1997	0.93	2.98	3.00
1998	2.33	7.96	8.00
1999	2.32	6.91	7.00
2000	2.17	5.90	6.00
2001	2.09	3.98	4.00
2002	2.31	6.85	7.00
2003	4.05	9.89	10.00
2004	3.44	7.99	8.00

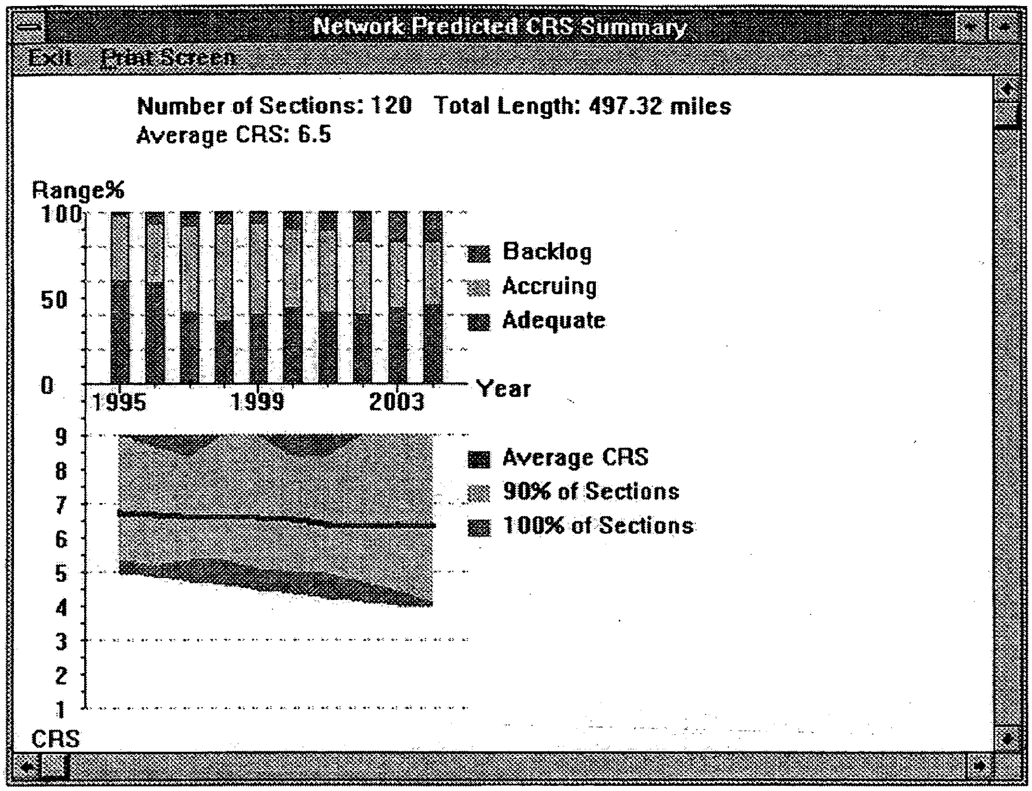
Note: 1 in = 2.54 cm, 1 mile = 1.61 km

FIGURE 5 Network benefit and cost analysis summary.



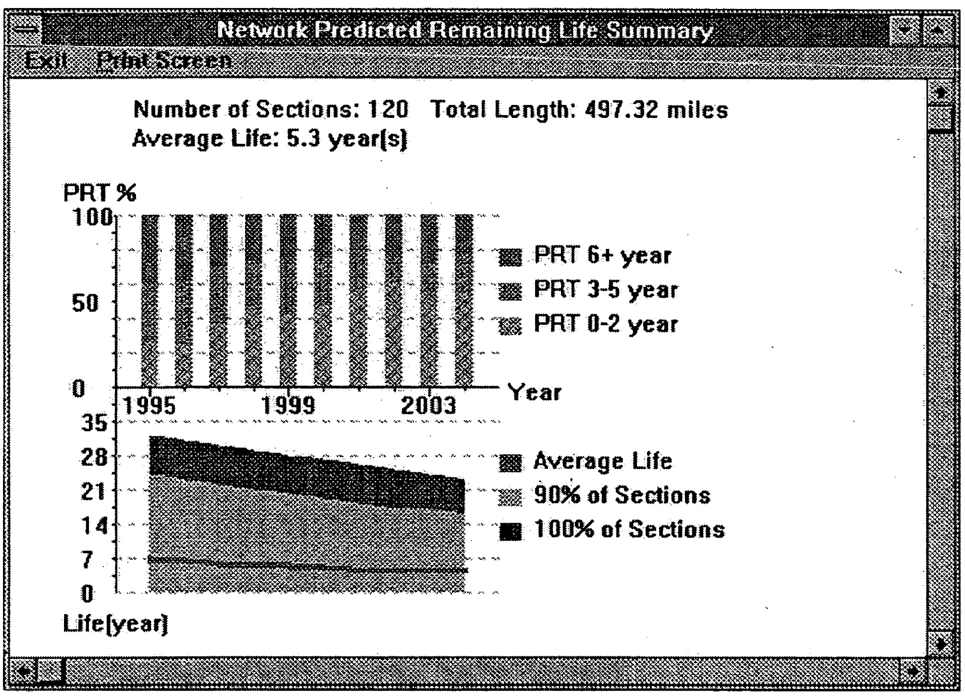
Note: 1 in = 2.54 cm, 1 mile = 1.61 km

FIGURE 6 Rehabilitation and cost allocation summary.



Note: 1 in = 2.54 cm, 1 mile = 1.61 km

FIGURE 7 Pavement condition summary.



Note: 1 in = 2.54 cm, 1 mile = 1.61 km

FIGURE 8 Pavement remaining life summary.

condition, 9 percent will be in Backlog condition, and the rest will be in Accruing condition. The average CRS will be 6.6 in 1997; the CRS for 90 percent of the sections will be between 5.4 and 8.4; the highest CRS will be 9; and the lowest CRS will be about 4.7. Figure 8 indicates that in 1997, 40 percent of the mileage will be in need of rehabilitation within 0 to 2 years; 30 percent will need rehabilitation in 3 to 5 years; and the remaining 30 percent will need no rehabilitation for at least 6 years. The average remaining life will be about 5.5 and will range from 0 to 30 years for various sections in the district. Ninety percent of the sections will have a remaining life of 21 years or less.

ALTERNATIVE DECISION-MAKING ALGORITHMS

A PMS should help managers select the "best" or most nearly "optimal" strategies after evaluating the costs and benefits with regard to various constraints. In recent years, many ranking and optimization algorithms and theories have been developed to help in making rehabilitation decisions (*2,7–10*). Each algorithm has advantages and disadvantages, and satisfies different requirements. However, none can meet all the desired decision-making needs. For example, if someone wants to repair the worst pavement sections first, he or she may choose the ranking algorithm, which will prioritize projects in worst-first order regardless of the rehabilitation type or life-cycle costs. Another person may want to consider both network-level prioritization and project-level rehabilitation type selection. He or she may choose the incremental benefit-cost ratio algorithm. A PMS that provides options for network-level and project-level decision making is more useful to a wider range of users in the agency who have a range of different concerns, priorities, and goals.

Windows ILLINET offers a total of 60 different decision-making options: 5 network-level algorithms, 6 project-level rehabilitation selection algorithms, and 3 different types of benefits. The system can run the algorithms only or can combine the algorithms with a user-defined committed rehabilitation plan, in which the user identifies specific rehabilitation projects that must be done at specific times.

Figure 9 contains all of the algorithm alternatives offered by Windows ILLINET. Committed Rehab Only, Needs, Ranking, B/C Ratio (Benefit/Cost), and Incremental B/C (Incremental Benefit Cost) are the five network algorithms. Committed Rehab Only means the system will do the analysis based only on the user-specified committed plan, without selecting any other rehabilitation projects or employing any other optimization algorithm. The other four algorithms can be run either alone or in combination with the committed rehabilitation plan. Decision Tree, Life-Cycle Cost, and Single Rehab options of Patching, 7.6-cm (3-in.) Overlay, 12.7-cm (5-in.) Overlay, or Reconstruction are the six project-level alternatives. The Decision Tree method uses decision tree cutoffs to determine which rehabilitation type should be selected for a given project. The critical CRS (Min CRS) is used in the Single Rehab method to select projects for rehabilitation. The ratio of predicted pavement life after rehabilitation to rehabilitation cost is the selection criterion used in the Life-Cycle Cost method. The three benefits the user can select are (a) Vehicle-Miles traveled on adequate pavements, a benefit measure that considers both pavement condition and traffic volume; (b) Average CRS over the multiyear analysis period, which is related to the area under the CRS performance curve; and (c) User Cost, which puts a dollar amount per person per mile on operational costs as a function of pavement condition.

INTERACTIVE SYSTEM

Once the decision maker understands the pavement conditions and related information, he or she can make decisions by applying his or her own experience to evaluating the rehabilitation recommendations generated by the optimization algorithms supplied by the PMS. The decisions made by the PMS include alternative maintenance or rehabilitation strategies for given roadway sections based on the functional, structural, and material deficiencies found. It is true that several optimization algorithms exist in the system that can generate network rehabilitation strategies based on predicted pavement performance and can be accomplished by a computer without involving the pavement engineers or decision makers to a great extent. However, in this case, decision makers have no way of being involved in the decision-making process. The whole procedure follows a predetermined path until it generates the output. This is why a PMS is often called a "black box." Moreover, the outputs of the optimization algorithms are often not trusted or accepted. Therefore, there is a great need to involve decision makers to a greater degree in the process.

Windows ILLINET allows users to input their own policies and constraints. The decision-making policies may be different for each district and agency. For example, how many times can the highway be patched between two overlays? What conditions must be met to warrant reconstruction? What is the maximum pavement life that should be considered in calculating benefit? Windows ILLINET provides defaults for policy issues such as these; however, users can modify the defaults to reflect their own policies and constraints. Users can also specify any time period for the multiyear rehabilitation planning.

The procedure for using the interactive capability of Windows ILLINET to make the best rehabilitation decisions is as follows. The initial plan can be input by the user or generated automatically by the optimization algorithms supplied in the system, or a combination of the two. After prediction and analysis, the user can save and modify the rehabilitation plan generated by the system based on experience and then, evaluate the plan again. This procedure can be repeated until a satisfactory plan is obtained.

The screen that allows users to modify and create rehabilitation plans is depicted in Figure 10. A user can modify the rehabilitation plan (rehabilitation year, type, and cost) for any section using the Edit Plan option. He or she can also view the total cost spent each year and save the edited plan or cancel it.

USER-FRIENDLY INTERFACE

A user-friendly interface is applied in this interactive PMS. The interface's purpose is to make the system easy to learn, use, and control. "Easy to learn" means users can master this system in a shorter time compared to other pavement management systems. To do this, the system offers pull-down menus on the screen. Users can manipulate a mouse to pick up many items rather than using the keyboard. Ease of use is greatly enhanced by the on-line help function, available at any time, which provides detailed guidance that is relevant to the screen at which the user requested help. The system allows users to

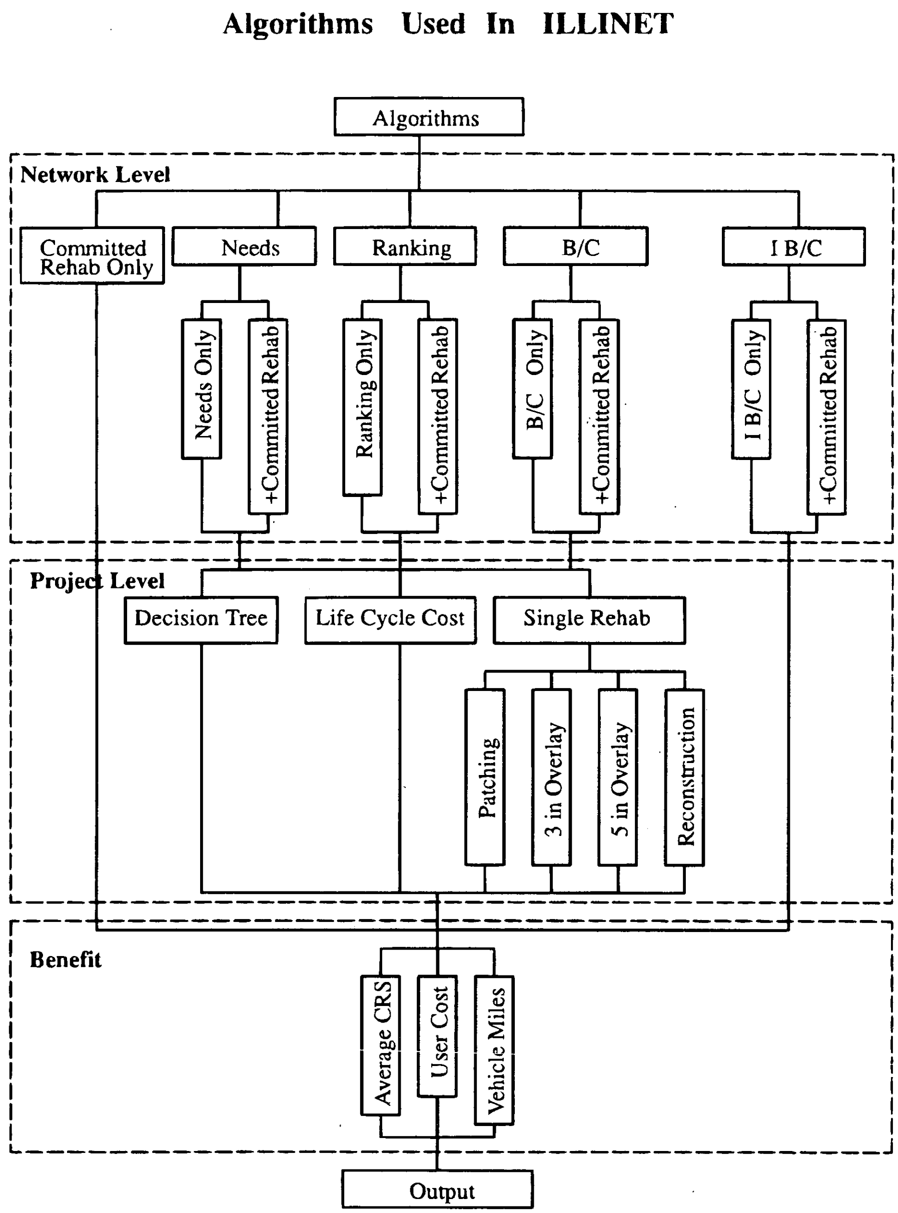
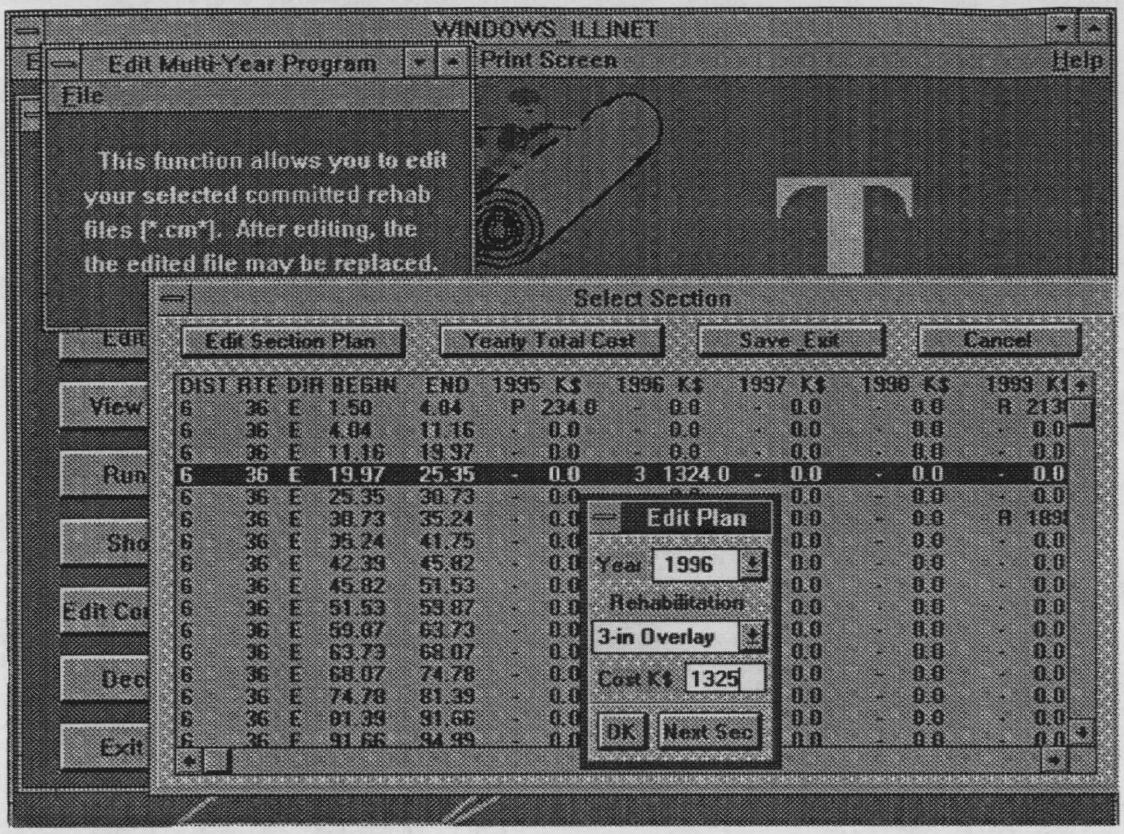


FIGURE 9 Algorithms used in Windows ILLINET.

easily start and cancel jobs whenever they want, which makes the system easy to control. Figure 11 illustrates all of Windows ILLINET's functions. Notice that almost all functions have more than one "entry" and "exit," which means the system is very flexible.

CONCLUSIONS

This paper describes some important features of Windows ILLINET. This system is a user-friendly, graphical interactive PMS. As a robust pavement management tool, this system interprets numerical data in a graphical manner that can greatly assist pavement managers in understanding pavement situations quickly and correctly. Multimedia applied in the system help demonstrate the pavement inspection motion video on the computer to vividly show pavement conditions. Furthermore, the interactive decision-making option allows users to use their own knowledge in the decision-making procedure, which helps avoid decisions that the users would not trust and would not consider acceptable. All in all, this system is a powerful PMS that has been enthusiastically welcomed by IDOT personnel, many of whom have been very involved in its development and testing. The results from the research shed light on important information-providing and decision-making capabilities of future PMS software. Implementation of Windows ILLINET is progressing and new ideas are being suggested as it is used more and more by IDOT engineers and managers.



Note: 1 in = 2.54 cm, 1 mile = 1.61 km

FIGURE 10 Interactive decision-making interface.

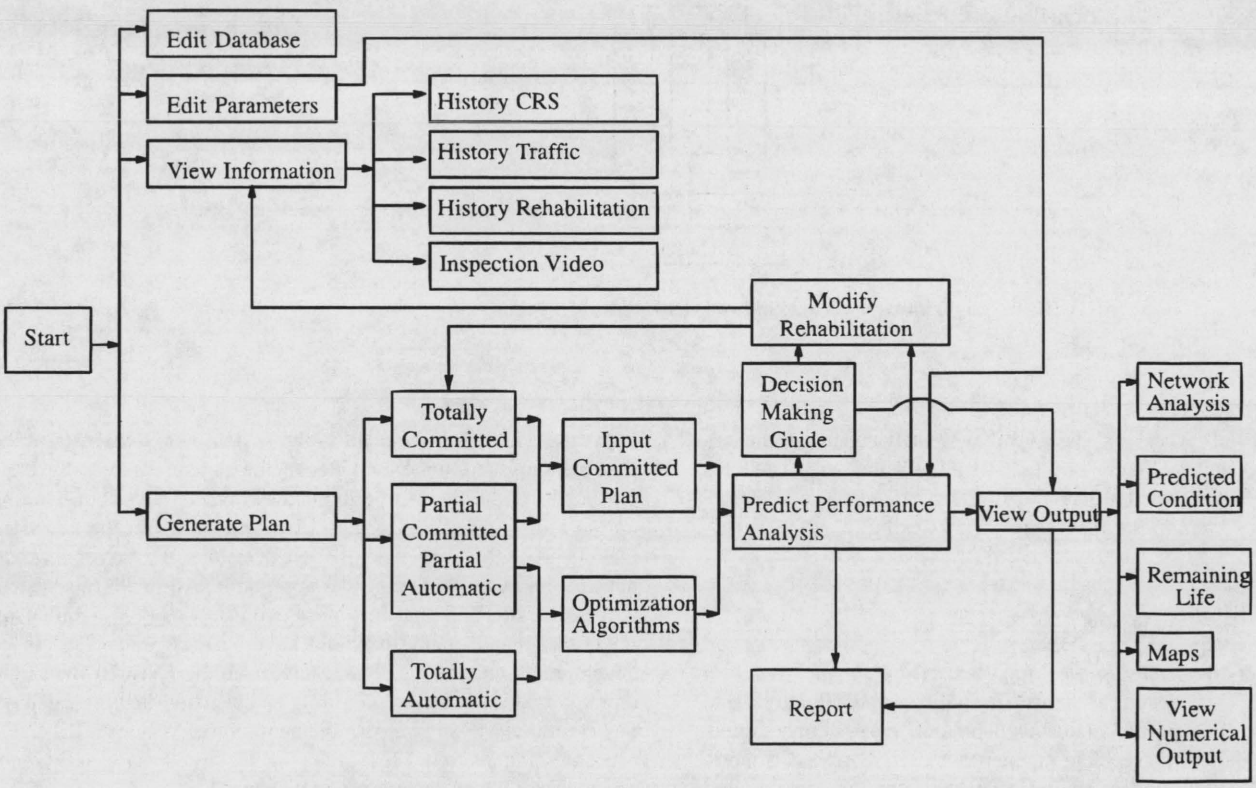


FIGURE 11 Function flow chart of Windows ILLINET.

REFERENCES

1. Darter, M. I., and J. P. Hall. *Illinois Pavement Feedback Data and Management System, Final Report.* Research Report 517-7F. Department of Civil Engineering, University of Illinois, Urbana-Champaign, 1990.
2. Mohseni, A., M. I. Darter, and J. P. Hall. *Alternative Methods for Pavement Network Rehabilitation Management.* Research Report 529-1, Department of Civil Engineering, University of Illinois, Urbana-Champaign, 1992.
3. Hall, K. T., Y. Lee, M. I. Darter, and D. L. Lippert. Forecasting Pavement Rehabilitation Needs for the Illinois Interstate Highway System. Presented at 73rd Annual Meeting of the Transportation Research Board, Washington, D.C., 1994.
4. Wildbur, P. *Information Graphics.* Van Nostrand Reinhold, New York, 1989.
5. Todd, S. *Scientific Visualization and Object Displays.* Master's thesis. University of Illinois at Urbana-Champaign, 1991.
6. Friedhoff, R., and W. Benzon. *The Second Computer Revolution: Visualization.* Harry N. Abrams, New York, 1989.
7. Haas, R., M. A. Karan, A. Cheetman, and S. Khalil. Pavement Rehabilitation Programming: A Range of Options. *Proc., North American Pavement Management Conference*, Vol. 2, 1985, pp. 6.41–6.66.
8. Shahin, M. Y., S. D. Kohn, R. L. Lytton, and W. F. McFarland. Pavement Budget Optimization Using the Incremental Benefit-Cost Technique. *Proc., North American Pavement Management Conference*, Vol. 2, 1985, pp. 6.96–6.107.
9. Cook, W. D., and R. L. Lytton. Recent Developments and Potential Future Directions in Ranking and Optimization Procedures for Pavement Management. *Proc., 2nd American Conference on Managing Pavements*, Vol. 2, 1987, pp. 2.183–2.194.
10. Haas, R., and W. R. Hudson. *Pavement Management Systems.* McGraw-Hill, New York, 1978.

Developing an Analysis System for Road Infrastructure Deterioration and Its Effect on Regional Economy

O. OMAR, Y. HAYASHI, K. DOI, AND T. OKUDA

Managing an aging infrastructure under budget limitations is becoming a critical issue in many countries. A model system for supporting the process of road infrastructure management is presented. The system provides information on expected road performance in terms of pavement deterioration and need for repair. Also, the direct impacts on the users of the facility and indirect impacts on the regional economy are quantified and given at each performance level. The system is supported by a geographic information system as a tool to facilitate decision making. A case study is carried out using the developed system to analyze the performance of a selected road network under different budget levels. It is shown how losses incurred by users and the regional economy vastly exceed savings from road budget reduction. Therefore, the importance of applying such a system to quantify direct and indirect impacts of road performance is highlighted throughout this analysis.

Until now, necessary road maintenance cost in Japan has been almost fully budgeted through treasury loans and investment. However, with the rapid increase in road infrastructure stock and the aging of the network, it will become difficult to budget for all the maintenance required. Under such circumstances, a certain level of deterioration and increase in road user costs might be inevitable. This may also bring about negative impacts on the regional and national economy as has occurred in other countries, such as the United States (*1*). Therefore, it is necessary to provide information on when, where, and how to repair to minimize the possible future damage cost due to budget shortage. Such information should also include the amount of direct and indirect costs incurred at any damage level.

Having recognized the importance of this issue, we have developed a model system to provide such information, focusing on highway pavements as a typical example. The system is composed of the following elements:

- Model to forecast future deterioration and need for repair, which treats deterioration and repair as stochastic phenomena. It can be applied on the network level to predict the performance of a road network under different repair strategies and budgets (*2,3*).
- System to quantify the direct impacts of deterioration in the form of changes in vehicle operating costs (VOCs) and travel speed. Accordingly, generalized travel cost (GTC) between regions and zones within a study area can be quantified under any road performance level (*4*).
- Model to forecast indirect impacts of road deterioration on the regional economy. It combines an input-output model with a business-industrial location model. The model can estimate the change in the production levels of the sectors of the economy (*4*).
- Geographic information system (GIS) base to support the decision-making process regarding deterioration and repair. This system provides reports and maps that can facilitate further analysis of information (*5,6*).

This paper briefly describes the outlines of each of these elements and how they function together. An example application of the system is also given. More details on model formulation and parameters can be found elsewhere (*2–6*).

SYSTEM OBJECTIVES

Modeling pavement deterioration is regarded as an essential need for the proper management of road infrastructure. Such a need becomes more critical if such management has to be carried out under budget constraints. Applying such models, road infrastructure renewal strategies commonly based on the "fire-alarm strategy" are likely to be abandoned in favor of strategies based on predicted information, leading to efficient use of the available budget. However, under the current trend of governments worldwide to neglect infrastructure repair, such models are not enough. It is also important to ascertain how much direct and indirect cost will be incurred if the infrastructure is left to deteriorate. If such information is known, the repair budget is likely to be raised. It is also important to adapt new technologies to develop computerized systems that can help the management process. With such systems in hand, systematic analysis can be conducted and the aspects of the issue can be clarified.

The main target of this research is to develop a system that can handle the required analysis. The elements and flow logic of a developed model system are shown in Figure 1. Each element is briefly discussed in the following section.

SYSTEM ELEMENTS

Deterioration and Repair Model

The purpose of the deterioration and repair model is to estimate the future performance of the road network considering its pavement condition. Future condition is governed by the deterioration mechanism and repair applications. Thus, a model is developed to simulate two simultaneous processes: (a) deterioration with age and (b)

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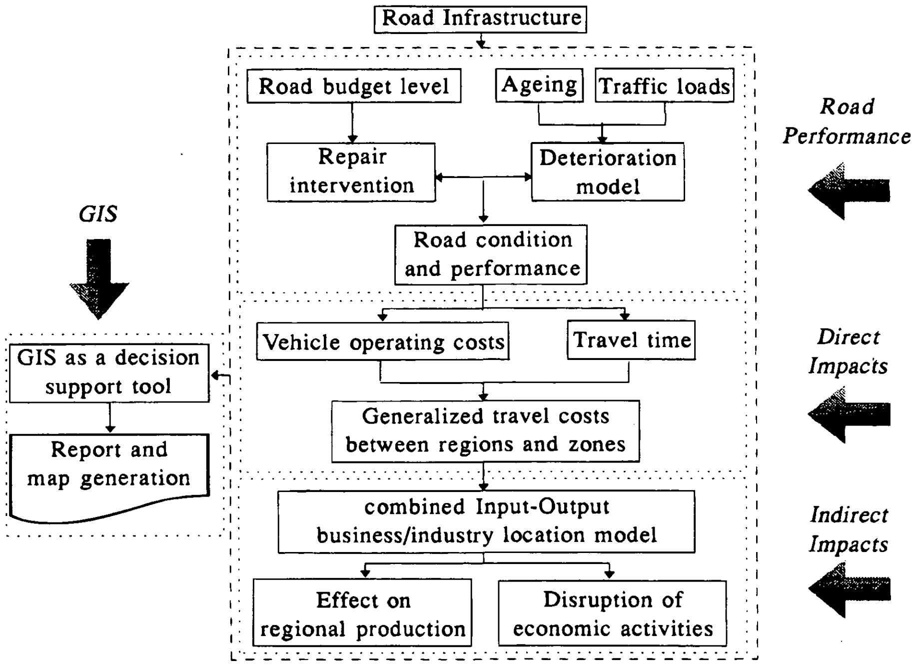


FIGURE 1 Study elements and flow logic.

repair need, application, and effect on condition. To account for the uncertainty in the deterioration mechanism and the subjective nature of repair decisions, the model treats these processes as stochastic phenomena.

Deterioration is represented as successive transitions over subsequent condition states. Condition is defined by the value of the maintenance control index (MCI, an index developed by the Japanese Ministry of Construction that is conceptually similar to the present serviceability index with maximum value of 10 for excellent pavement and minimum value of 0 for damaged pavement); and each state is defined by a range of MCI values. For example condition State 1 represents pavements with an MCI of 10-8, while State 2 has an MCI of 8-6, and so on. The pavement age at which any of such transitions may occur is treated as a stochastic variable. The model is developed for classes of pavements each of which has similar life expectation. Here, classification is based on pavement type, traffic level, and surrounding environment. For each class, a group of probability distribution functions is fitted to historical data on the age at which transition between condition states occurred. Each function gives the probability of transition between a certain pair of successive condition states as a function of pavement age. Therefore, given pavement age, current condition state, and its class, probable future condition state (a specific MCI range) can be predicted. The function used has the following form:

$$f(t) = P(t \le T \le t + \Delta t)/\Delta t = \frac{\alpha}{\beta}\left(\frac{t - t_o}{\beta}\right)^{\alpha - 1} \exp\left(-\left(\frac{t - t_o}{\beta}\right)^{\alpha}\right) \quad (1)$$

where $P(t \le T \le t + \Delta t)$ is the probability that transition will occur at age T, which is between t and $t + \Delta t$ years; t_0 is the minimum age at transition, α and β are parameters.

Equation 1 can be rewritten to yield a new indicator for performance evaluation based on pavement reliability, that is, the probability of staying in the current condition state, and can be given by

$$R(t) = P(T > t) = 1 - \int_0^t f(t)dt = \exp\left(-\left(\frac{t - t_o}{\beta}\right)^{\alpha}\right) \quad (2)$$

where $P(T > t)$ is the probability of no transition for at least t years.

In applying this model to estimate future deterioration, pavement sections divided into cohorts based on age and condition state are partially transferred to successive condition states with a yearly rate that equals

$$\lambda(t) = P(T < t + \Delta t \mid T > t)/\Delta t = \frac{f(t)}{R(t)} = \frac{\alpha}{\beta}\left(\frac{t - t_o}{\beta}\right)^{\alpha - 1} \quad (3)$$

where $P(T < t + \Delta t \mid T > t)$ is the probability that transition will occur before age $t + \Delta t$, given that it has not occurred at or before age t. Applying this rate, the probable condition of any section at any time, MCI_t, can be predicted.

As for repair modeling, transition is assumed to occur between only two states, the "repair not required" state (i.e., only routine maintenance is required) and the "repair required" state (i.e., overlay or reconstruction is required). The probability of occurrence of this transition depends on the pavement class and age. In this case, cohorts of pavement sections from the same class and age are partially selected for repair based on a yearly repair rate given by an equation similar to the transition rate given by Equation 3. Type of repair required is decided based on the class and condition state of sections selected for repair. The effect of repair is simulated as transition to a better condition state that depends on the probable efficiency of the selected repair type.

Prediction of future performance of a road network entails repeating the process of estimating the expected yearly transitions in condition and selection for repair (and thus cost) and its effect, year by year over an analysis period. Since the process is stochastic, it must be repeated a sufficient number of times to obtain the most probable future performance and repair needs. Effect of different budget levels on performance can be estimated by adjusting the repair rate to reflect the change in budget.

The prediction of pavement longitudinal roughness, *LR*, was also modeled since it is the major distress influencing travel speed. This was done as a linear regression relation between the expected amount of roughness and pavement condition (MCI_t) and age.

Evaluation System for Direct Impacts

A second element is used to evaluate the effects of road condition on the direct users of the facility. The effects considered here are the change in VOCs and operating speed, and thus travel time. The total impact is given as GTC between any two zones, including both of the previously mentioned cost factors. The evaluation is based on the estimated road condition according to the deterioration and repair model. The following relations are employed:

1. VOCs:

$$VOC_{tc} = \psi_c + \phi_c \exp(-MCI_t) + \epsilon_c \frac{1}{V_{tc}} \tag{4}$$

in which

$$V_{tc} = V_{oc} - \omega_c LR_t \tag{5}$$

where

VOC_{tc} = VOC of vehicle type c on a given pavement section at time t (yen/km);
MCI_t = expected pavement condition at time t;
V_{tc} = average operating speed of vehicle type c (passenger cars and trucks) on a pavement with condition MCI_t (km/hr),
$\psi_c, \phi_c, \epsilon_c, \omega_c$ = regression parameters that depend on vehicle type c;
V_{oc} = running speed of vehicle type c on similar section with new pavement (km/hr); and
LR_t = longitudinal roughness at time t (mm).

2. Travel time:

$$T_{tc} = \frac{60L}{V_{tc}} \tag{6}$$

where

T_{tc} = average travel time of vehicle c on a given road section at time t (min); and
L = length of road section (km).

3. GTC between zones:

$$C_{ij} = \min \sum_{ij} \sum_{c} (VOC_{tc}L + T_{tc}C_c) r_c \tag{7}$$

where

C_{ij} = GTC from zone i to zone j at time t;
C_c = value of time for vehicle type c (yen/min); and
r_c = ratio of vehicle type c in the traffic stream.

The first summation in Equation 7 is done over the road sections of each alternative route from i to j. Resulting minimum travel cost is taken as the GTC.

Evaluation System for Indirect Impacts

In this study, the indirect impacts are represented by the change in the productivity of each of the economy sectors. Unlike input-output analyses, the change is analyzed on the regional and zone level assuming no change in the total national product. The main purpose of this analysis is to show the consequences of cutting the road repair budget in the study region. A budget reduction might result from a general decline in road budget or a reallocation with a lower share for the study region. The developed model estimates the shares of demand and supply for each production sector located in each region and zone of the nation. Change in accessibility to any region or zone causes rearrangement of the demand-supply shares between regions and zones and thus losses to some sectors at certain locations. The GTC to a region or zone is assumed to reflect its accessibility and thus the attractiveness of production activities in exchange with other regions and zones.

The mathematical model is obtained by combining the concepts of input-output analysis with those of a business-industrial location model. The formulation is as follows (*4*):

The basic relation in the model is the equilibrium between supply and demand as given by

$$X^k = \sum_m A^{mk} X^m + F^k \tag{8}$$

where

X^k = total products of any sector k;
X^m = total products of sector m, ($m = 1, 2, \ldots, k, \ldots, \ldots$);
A^{mk} = input coefficient of materials and services to sector m from sector k (amount of product k required for producing one unit m); and
F^k = final demand for sector k.

The implemented business location model uses the number of employees in each sector rather than the amount of products. Thus, Equation 8 is rewritten as

$$E^k = \sum_m \theta^{mk} E^m + B^k \tag{9}$$

in which

$$\theta^{mk} = \frac{\omega^{mk}}{\sum_m \omega^{mk}}, \quad and \quad \omega^{mk} = x^{mk} \frac{E^k}{X^k} \tag{10}$$

where

E^k = number of employees in sector k;
E^m = number of employees in sector m;
θ^{mk} = input coefficient of employee to sector m from sector k (number of employees in k to serve one employee in m);
B^k = number of employees in k to serve the final demand; and
x^{mk} = sales of k products to sector m.

Equation 9 represents the market only under the assumption that demand creates equal supply. However, in reality, the existence of demand only increases the chance of supply. Thus, Equation 9 has to be written twice, once from the viewpoint of demand and again from the viewpoint of supply. Solution of the two equations yields the market equilibrium.

Equation 9 can be rewritten from the viewpoint of demand, while taking into account the distribution of demand on products of any sector k over g regions ($g = 1, 2, \ldots, h, \ldots, \ldots$), which are further divided into j zones each ($j = 1, 2, \ldots, i, \ldots, \ldots$), as follows.

For any zone i in any region h,

$$D^k_{hi} = \eta^k \sum_m \theta^{mk} \sum_g \sum_j S^m_{gj} P^{mk}_{hij} + \kappa^k \sum_g \sum_j R_{gj} P^k_{hij} \qquad (11)$$

in which

$$P^{mk}_{hij} = \frac{S^k_{hi} \exp(\delta^{mk} C_{hij})}{\sum_g \sum_j S^k_{gi} \exp(\delta^{mk} C_{hij})} \qquad (12)$$

and for the whole region,

$$D^k_h = \sum_j D^k_{hj} \qquad (13)$$

where

- D^k_{hi} = demand (number of required employees) for sector k located in zone hi (i.e., zone i located in region h);
- S^m_{gj} = supply (employees) by sector m located in zone gj;
- R_{gj} = population in zone gj;
- P^{mk}_{hij} = probability of selecting zone hi to supply k to sector m located in zone gj;
- P^k_{hij} = probability of selecting zone hi to supply k to final demand sector located in zone gj;
- η^k, κ^k = regression parameters;
- δ^{mk} = diminishing parameter reflecting the effect of transport cost on the marketing of product of sector k to sector m;
- C_{hij} = the GTC from zone hi to zone gj; and
- D^k_h = total demand (employees) for sector k located in region h.

The physical meaning of Equation 12 is that demand probability rises with the scale of the producer (S) and its closeness to the market (C).

From the viewpoint of supply, the choice of suppliers in this model is where to locate their activities to cover the demand. Under this condition, supply will be located as follows.

For region h,

$$S^k_h = S^k \frac{D^{k\gamma}_h \exp(\delta^k U^k_h)}{\sum_g D^{k\gamma^k}_g \exp(\delta^k U^k_g)} \qquad (14)$$

in which

$$U^k_h = \sum_m \theta^{mk} \ln \left[\sum_g D^m_g \exp(\delta^k C_{hg}) \right] \qquad (15)$$

for any zone i in any region h,

$$S^k_{hi} = S^k_h \frac{D^{k\gamma^k}_{hi} \exp(\delta^k U^k_{hi})}{\sum_j D^{k\gamma^k}_{hj} \exp(\delta^k U^k_{hj})} \qquad (16)$$

in which

$$U^k_{hi} = \sum_m \theta^{mk} \ln \left[\sum_j D^m_{hj} \exp(\delta^k C_{ij}) \right] \qquad (17)$$

where

- S^k_h = supply (employees) by sector k located in region h;
- S^k = total supply (employees) by sector k;
- δ^k = diminishing parameter reflecting the average effect of transport cost on the marketing of product of sector k;
- C_{hg} = the average GTC from region h to region g;
- γ^k = regression parameter;
- U^k_h = expected extreme utility for producing k (considering transport cost) if located in h;
- S^k_{hi} = supply (employees) by sector k located in zone hi; and
- U^k_{hi} = expected extreme utility for producing k (considering transport cost) if located in zone hi.

Equations 15 and 17 mean that business considers both the amount of demand and its distance while evaluating the utility of each possible location for its activities.

The solution of the group of Equations 11 through 17 can be obtained by iteration under the condition

$$D^k_{hi} = S^k_{hj} \qquad (18)$$

GIS Base as a Decision-Supporting Tool

Road networks are inherently geographic as they are extended over a wide area and intersect with different land topography, such as rivers, mountains, buildings, and other roads. Also, network components and events taking place within the network are locational in nature. For example, the extent and shape of links, road intersections, accidents, and pavement conditions cannot be completely defined unless the geographic location of the component or event is given. Thus, spatial considerations in the analysis for different road activities, including maintenance and repair management, are essential and can vastly improve the quality of the decision-making process. However, highway infrastructure management systems are usually based on a central data bank in which only descriptive data are handled. More advanced systems are also supported by computer-assisted drafting systems for map generation. None of these systems permits spatial operations on the data. GIS as a system with spatial analysis capabilities—besides having the characteristics of the above-mentioned systems—particularly matches the geographic nature of road networks. Therefore, we coupled the previously discussed elements with a GIS. The developed system includes the following components:

- A spatial data base that stores data describing the spatial distribution of geographic features in the study area. Examples of such features are roads, city borders, land use, and main utility lines. Each feature is stored as a separate layer and is related with the other features by location as a common key.
- An attribute data base in which representative nongeographic information on the spatial features is stored. Examples of such

information for a road segment are road inventory, traffic volume, and pavement condition.

- An analysis module in the form of computer programs that represent the previously mentioned three elements and use data from both the spatial and attributes data bases. Spatial integration of different types of data is also possible to produce new information.
- An output-generation module to summarize data and information and produce reports and maps. The generation of such output can be achieved through programs, user textual queries, or user geographic queries.

The resulting system has the following main advantages: automation of map generation, powerful geographic queries, network analysis and simulation, and spatial analysis and data integration.

SUMMARY OF RESULTS

The developed model system was applied to a part of the trunk road network within the Aichi region of Japan. The purpose of the application was to examine the performance of this network under different levels of repair budget. The corresponding direct and indirect costs were quantified. Also the merits of introducing GIS to the system were examined. This section briefly gives some of the results. Detailed results can be found elsewhere (*2–5*).

Figure 2 illustrates predicted performance in terms of reliability and MCI of the average section of the network at the 80 percent and 60 percent repair budget levels (total 13-year cut of about 4 and 8.1 billion yen, respectively) compared with those at the 100 percent level (current investment level). The results indicate that a 20 percent reduction in the budget from its current level would result in a 20 percent and 17 percent decrease in the possible attainable reliability and MCI by the year 2000, respectively. On the other hand, a 40 percent reduction would result in a 44 percent and 38 percent decrease, respectively. The increasing damage-cut ratios show the effect of cumulative damage due to budget shortage.

Comparison between the trends of curves in Figure 2 indicates that the use of reliability as a performance indicator can lead to conclusions similar to those obtained using a condition index such as the MCI. However, the use of reliability has the advantage of it being able to link directly to the expected total repair need.

As for prediction accuracy, the predicted average MCI value in 1991 under the assumption of current budget level is estimated to be 6.54, while the true average obtained from the condition survey for the same year is 6.44. Comparison with prediction results obtained using other deterministic models reveals that predictions using the developed stochastic performance model are more accurate.

Besides physical damages to the network, the model was employed to estimate the financial losses due to budget cuts. A rapid increase in the yearly budget required for routine maintenance is expected (see Figure 3). Another virtual loss is the increase in the total repair needs of the network that will be incurred if all condition deficiencies are to be properly repaired in a certain future year (cost to recover condition). The expected future recovery costs are indicated in Figure 3. As shown, the costs in the year 2000 are about two and three times as much as the total cut in budget in the 80 percent and 60 percent budget levels, respectively.

Budget limitations cause both physical deterioration to the road system and financial losses to the road agency. However, it is possible to cope with such a situation to minimize these negative impacts by changing policies. The model was employed to examine such policy changes. For example Figure 4 illustrates the progress of the average MCI over the simulation period for the 80 percent budget level assuming different priority ranking criteria (based on pavement age, condition state, and traffic level). It is indicated that such a policy change can result in considerable change in performance level and ultimate condition. The curves indicate that the age-state-traffic ranking criteria are optimal in this case. However, further analysis of different budget levels reveals that it is not necessary that this ranking criteria always be the optimal.

As for the direct user impacts, the yearly relative savings or losses in total VOC were computed (see Figure 5). As indicated, an exponential increase in cumulative VOC losses to the direct users in the cases of limited budgets compared with an almost stable VOC in the 100 percent case. Doubling budget cuts results in more than twice the loss as indicated by the increasing divergence between loss curves in the figure. As a result of the increase in VOC and longer travel time, an increase in the GTC between the region's zones will follow. Figure 5 also depicts the percentage change in average GTC

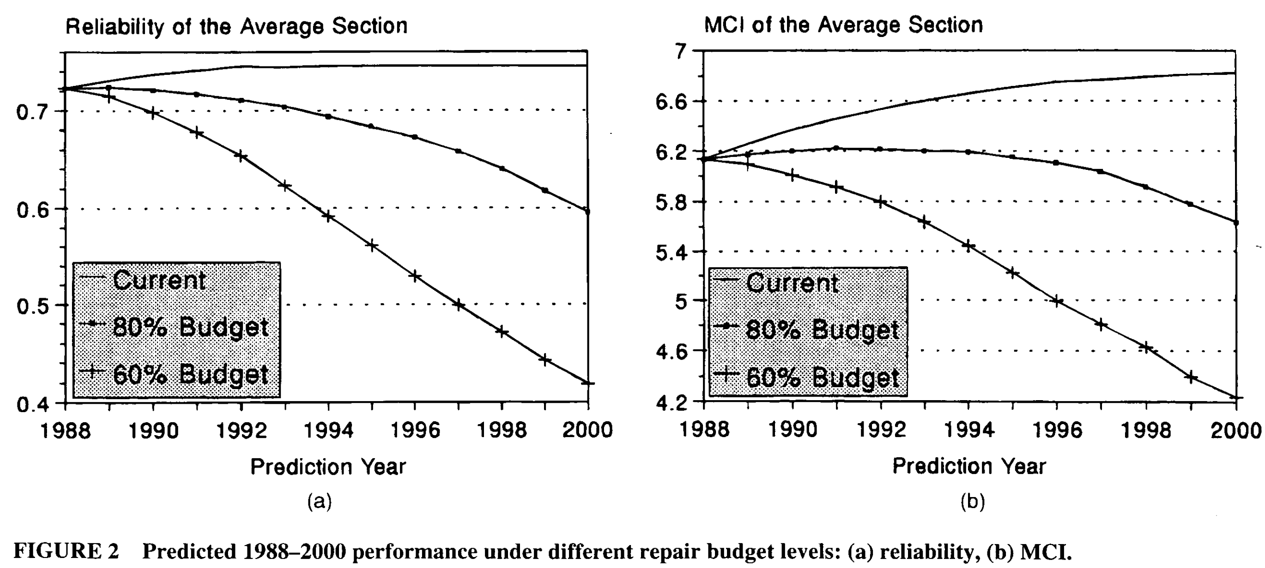


FIGURE 2 Predicted 1988–2000 performance under different repair budget levels: (a) reliability, (b) MCI.

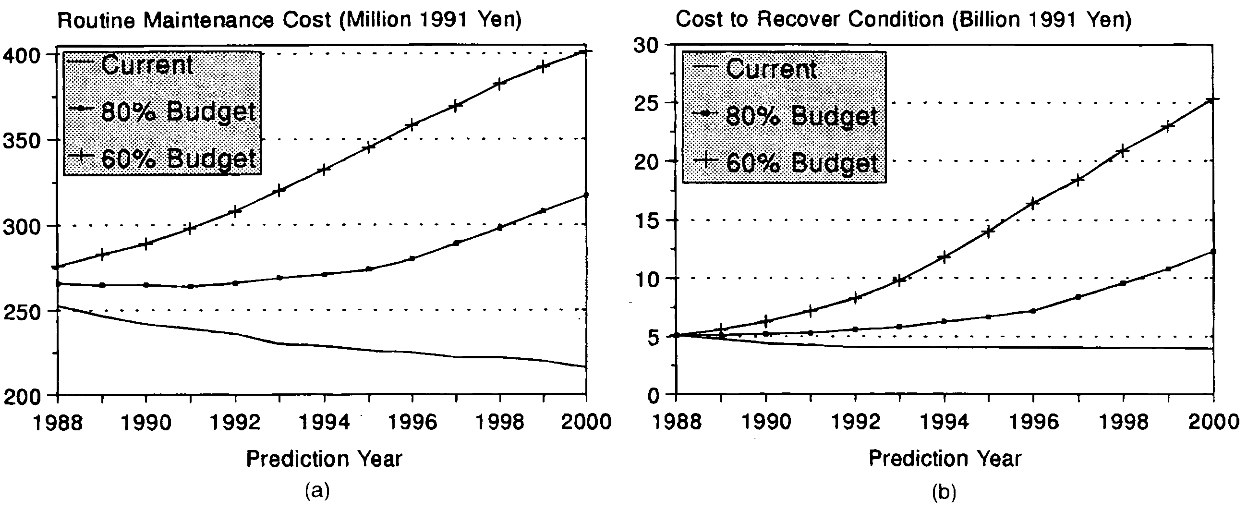


FIGURE 3 Anticipated financial losses due to budget costs: (a) routine maintenance, (b) recovery costs.

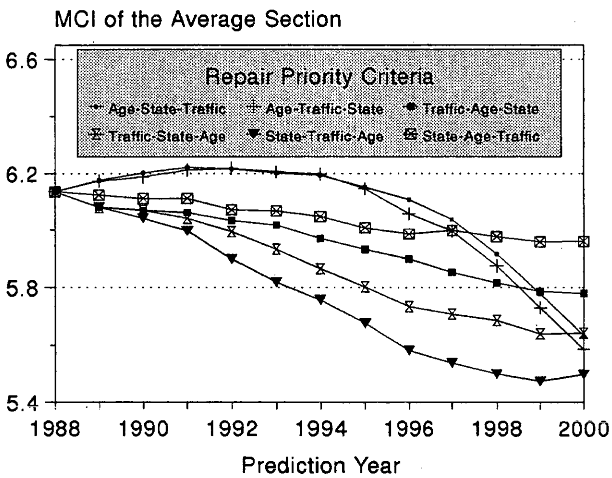


FIGURE 4 Performance assuming different allocation priority criteria.

for each zone at the 10th year of the analysis period. As indicated, rapid increase in such changes is expected with increasing budget cuts. It is also indicated that the differences in the magnitude of the impacts on different zones are almost negligible in the 100 percent case. However, such differences become more noticeable with the increase in budget cuts, clearly illustrated in the 60 percent case. This indicates a possible disruption in the spatial pattern of transportation costs across the region.

As for the indirect impacts, the number of employees in each sector was computed for each road performance level and thus GTC pattern. These numbers were then multiplied by the productivity of the employees in each sector to get the amount of the sector's production. The change in the amount of production of all sectors as a result of different road conditions was considered as the effect on the economy of the study region and its zones.

The impacts in monetary terms on a selected zone, as an example, is illustrated in Figure 6 for each sector under the 80 percent and 60 percent budget levels. Most of the loss is in the manufacturing sector, which is the main economic activity in this zone. Total produc-

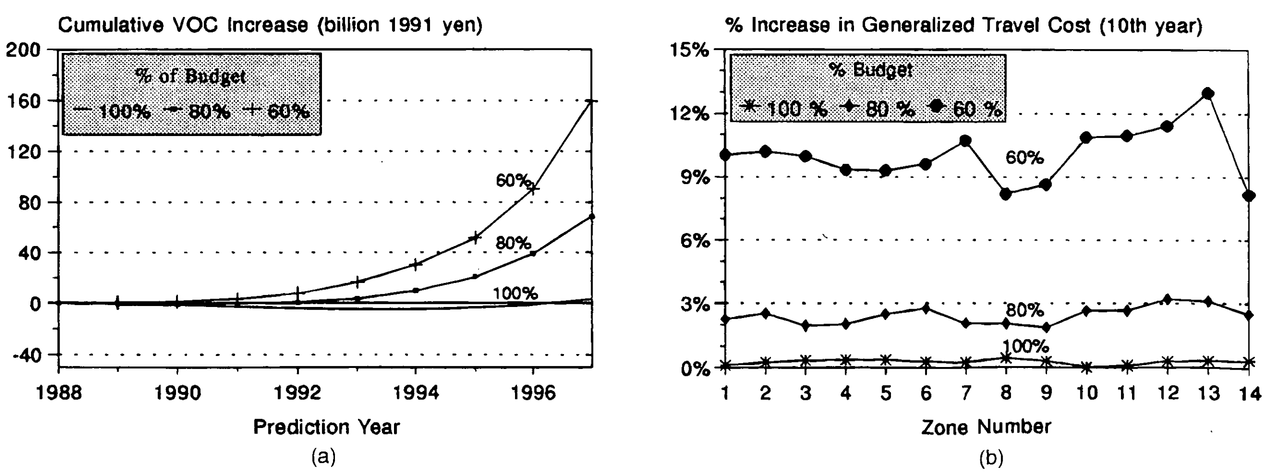


FIGURE 5 Direct impacts: (a) change in VOC, (b) change in GTC.

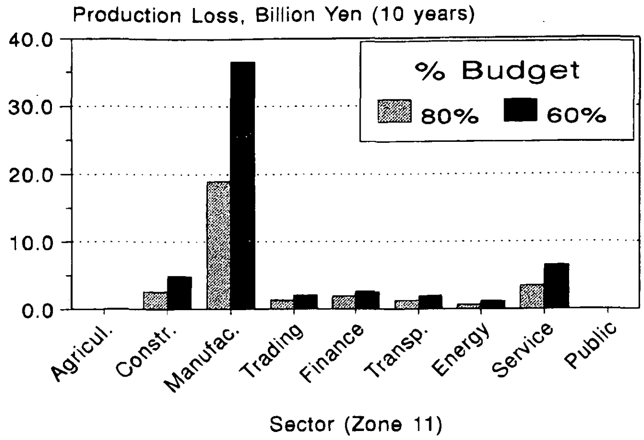


FIGURE 6 Indirect impacts: losses by sector for a selected zone.

tion losses over 10 years account for 30 and 56 billion yen for the 80 percent and 60 percent budget levels, respectively. These losses increase rapidly with time so that the cumulative losses over 20 years, for example, become about 20 times the losses after 10 years.

From the foregoing analysis, it is clear that the losses to the road agency, road users, and the economy largely exceed the savings by repair budget cuts. Such a result can be used to amplify the importance of satisfactory infrastructure performance and thus budget justification.

As for the GIS application, Figure 7 provides an example of an analysis type that becomes possible by introducing GIS to the system. The figure depicts an overlay between a road section scheduled for future rehabilitation and the main waterlines underneath this section. The overlay gives the location, characteristics, and future repair year and authority of those lines intersecting with that road section. Better coordination between timing of road repair and utilities repair and installation can be realized with such analysis.

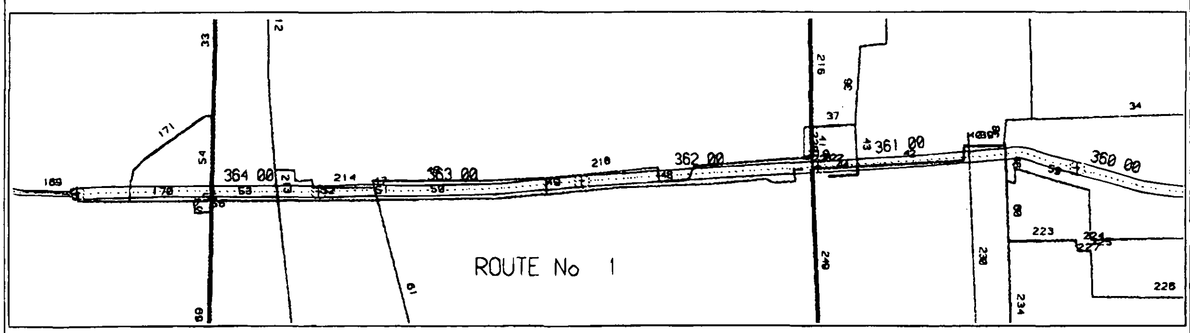
CONCLUSIONS

This paper briefly discussed the development of a model system for supporting road infrastructure management. The system can be implemented on the network level to examine pavement performance under different repair policies and budgeting levels and scenarios. More importantly, the system can be used to quantify negative impacts on road agency, road users, and the regional economy due to unsatisfactory performance levels as a result of repair budget cuts. The results of such a system can be used to amplify the importance of satisfactory infrastructure performance level and, thus, justify required budgets.

Some of the findings obtained throughout system development and application are as follows:

- Modeling deterioration and repair as stochastic phenomena is more realistic. This yields more accurate simulation and prediction of future performance.
- Estimation of the direct and indirect costs incurred at any road performance level is essential to clarify the importance of keeping satisfactory conditions. The results indicate that such costs are much larger than the cost of proper repair of the infrastructure.

LOCATION OF MAIN WATER SUPPLY LINES



WATER-ID	DIAM(CM)	OVERLAP LENGTH(M)	REPAIR YEAR	AUTHORITY
46	60	45 11	199*	******
230	40	45 02	199*	******
42	60	455 52	199*	******
43	30	30 76	199*	******
222	30	181 39	199*	******
221	40	28 81	199*	******
44	60	14 23	199*	******
249	110	16 25	199*	******
48	30	57 79	199*	******
218	40	50 30	199*	******
218	40	45 00	199*	******
47	40	12 47	199*	******
214	40	248 04	199*	******
49	30	45 13	199*	******
51	40	34 30	199*	******
213	40	45 11	199*	******
171	40	45 15	199*	******
54	140	45 00	199*	******
169	30	225 21	199*	******

DIAMETER 300 ~ 400 mm
DIAMETER 500 ~ 800 mm
DIAMETER 900 ~ 1400mm

FIGURE 7 Coordination between timing of road repair and utilities repair and installations.

- Adaptation of new technologies, such as GIS, in the area of infrastructure management is promising and can vastly improve the quality of the decision-making process.

Finally, the developed system framework can also be adapted for other types of infrastructure. With such systems in hand, infrastructure renewal strategies commonly based on the fire-alarm strategy are likely to be abandoned in favor of strategies based on predicted information.

REFERENCES

1. Choate, P., and S. Walter. *America in Ruins.* Council of State Planning Agencies, 1981.
2. Omar, O., Y. Hayashi, and K. Doi. A Method of Maintenance Policy Evaluation Considering Highway Network Reliability, *Selected Proc., 6th World Conference on Transport Research*, 1992, pp. 2363–2374.
3. Hayashi, Y., O. Omar, and S. Kikukawa. A Probabilistic Performance Model for Transport Infrastructure Management: The Case of Highway Pavement, *Journal of Infrastructure Planning and Management* (Japan Society of Civil Engineering) Vol. IV-19, No. 464, 1993, pp. 135–141.
4. Hayashi, Y., O. Omar, K. Doi, and T. Okuda. Effect of Road Pavement Deterioration on the Regional Economy: A Case Study. *Proc. 20th PTRC Summer Annual Meeting*, 1992, pp. 145–157.
5. Hayashi, Y., O. Omar, K. Doi, and K. Kawamata. A GIS-Based Highway Network Deterioration Analysis. *Proc., 19th PTRC Summer Annual Meeting*, 1991, pp. 137–150.
6. Omar, O., Y. Hayashi, and K. Doi. Application of GIS to Highway Infrastructure Management. *Proc., Civil Engineering Annual Meeting*, Chubu Branch, Kanazawa, Japan, 1992.