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Development and Implementation of a New Rehabilitation Information and Priority Programming System (RIPPS)

M. A. KARAN, R. C. G. HAAS, A. CHEETHAM, T. J. CHRISTISON, and S. M. KHALIL

ABSTRACT

In November 1980 the province of Alberta (Alberta Transportation) initiated a project to develop and implement a pavement management system. Stage 1 involved the design and implementation of a pavement information and needs system (PINS), which was completed in October 1982. Stage 2 involved the design and implementation of a rehabilitation information and priority programming system (RIPPS), which was completed in June 1983. PINS includes the use of recursive performance models for predicting future riding comfort index, structural adequacy index, and visual condition index. These parameters are also combined into a pavement quality index to provide an overall measure of performance. The performance predictions are used to identify the current and future needs for rehabilitation improvements for each inventory section in the network. RIPPS involves the selection of candidate rehabilitation alternatives for each section, so that economic and performance analyses of each alternative for each possible implementation year can be conducted. A heuristic procedure has been developed as a priority programming model that employs marginal cost-effectiveness analyses. The model can be operated in two modes: (a) cost minimization (given performance constraints), and (b) effectiveness maximization (given annual budget constraints). The cost minimization method produces a program of rehabilitation improvements and the required annual budgets that will meet the desired level of network performance. The effectiveness maximization method produces a program of rehabilitation improvements and the resulting network performance that meets the available funds. In this paper an overview of PINS is given, and the major components of RIPPS and its development are described. Sample outputs are provided to illustrate the results obtained from the two modes.

One of the larger highway networks among states and provinces in North America is in Alberta. It has approximately 7,000 miles of paved primary highways and approximately 2,000 miles of paved secondary roads. Over the past decade the system has been expanding at an average rate of 200 miles per year. This represents a substantial investment of many millions of dollars, and like any other investment, it requires good management.

Realizing that pavement management is the process by which this investment can properly be managed, Alberta Transportation initiated a project in November 1980 to develop and implement a pavement management system (PMS) for the province.

In the first phase of the project a comprehensive plan for the project was developed (1). This was carried out as a planning project and it identified six successive, stand-alone stages for the overall, total PMS development and implementation project. These stages, which are briefly summarized in Figure 1, considered Alberta Transportation's goals and objectives, organizational structure, current practices, manpower and equipment resources, and financial constraints.

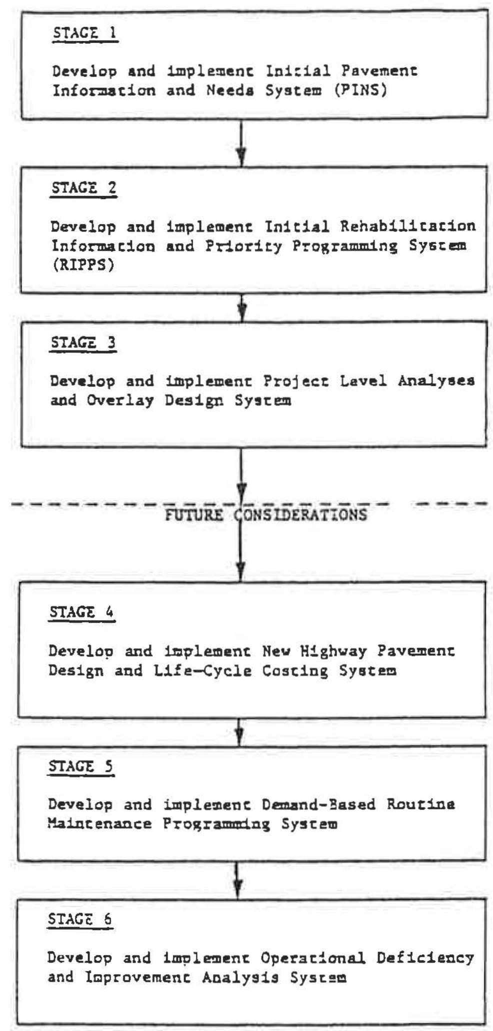

FIGURE 1 Staging of the project.

Stages 1 and 2 of the project, which were identified as the development and implementation of a pavement information and needs system (PINS) and a rehabilitation information and priority programming system (RIPPS), have been completed and implemented. In this paper an overview of PINS, which is described in detail elsewhere (2,3), is given, and the major components of RIPPS, including its subsystems and outputs, are described.

STAGE 1: PINS

The overall objective of Stage 1 was to produce a computerized system for determining the status of the highway network as well as pavement rehabilitation needs. This is the PINS.

PINS processes pavement management data from the pavement data base currently available in Alberta Transportation and generates for immediate and future use of department personnel the following items:

1. Present status of the network in terms of pavement quality index (PQI) and its components of structural adequacy index (SAI), riding comfort index (RCI), and visual condition index (VCI);
2. Remaining service life (in structural or serviceability terms or both) of each section in the network, based on the performance prediction models that have been developed;
3. Pavement improvement needs ranked with respect to PQI and the individual components of RCI, SAI, and VCI; and
4. Summary statistics (in tabular and graphical forms) of the present status of the highway network and improvement needs for each region.

The PINS program has the capability of first determining the present status of a section in terms of its RCI, SAI, VCI, and PQI parameters, as shown in Figure 2. These analyses can be conducted for every section in the network or in a region or on a highway. Once the analyses are completed for every section, the program produces detailed output for every such section as well as a status report for the network, region, or highway.

The next step in the analysis is to predict the performance for each performance parameter (i.e., RCI, SAI, VCI, and PQI). Prediction models specifically calibrated to Alberta conditions are used in the analysis. The development of these models is described in detail elsewhere (4,5).

Similar to present status analysis, performance prediction and needs analyses can be conducted for every section in the network or in a region or on a highway. The program produces graphical outputs (i.e., performance curves) for every section; it also gives the year in which the parameter will reach its minimum acceptable level. A sample output is shown in Figure 3.

The needs analysis can be conducted over a predetermined programming period, which can be 5, 10, 20, or 30 years. Thus pavement improvement needs (based on RCI, SAI, VCI, or PQI) are established for each year in 5-, 10-, 20-, or 30-year programming periods.

Although PINS does not establish a true priority program (this requires economic analysis and optimization), it does have the capability of ranking the sections in the order of their improvement needs and in terms of each performance parameter. This constitutes the network summary information that PINS produces. Figure 4 shows an example ranking list based on RCI. Also, three-dimensional histograms, like the one shown in Figure 5, are produced so that regions, districts, or highways in Alberta can be compared.

Needs tables are also produced for each performance parameter and for each year in the programming period. Figure 6 shows an example needs table.

In summary, the PINS program developed for Alberta analyzes the data base to (a) determine the present status, (b) predict performance, and (c) establish needs for each performance parameter for each year in the programming period of 5, 10, 20, or 30 years. The results are detailed in tabular form and graphical format for every section. Network summary information is also produced in tabular and graphical formats.

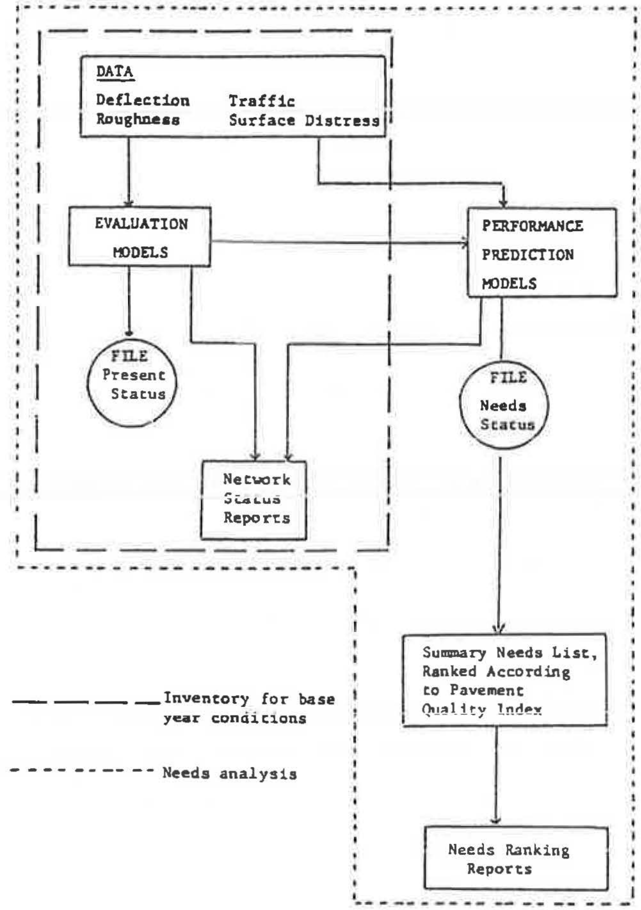

FIGURE 2 General structure of PINS.

The PINS program has been installed on Alberta Transportation's computer facilities and is now fully operational and is being used on a day-to-day basis.

STAGE 2: RIPPS

System Overview

The overall objective of Stage 2 was to produce a computerized system to analyze alternative rehabilitation strategies for the needs identified in PINS and to produce an optimum program of projects to be implemented over the programming period of up to 10 years. This is the RIPPS.

RIPPS basically processes the output of PINS and generates the following for department personnel:

1. Complete engineering and economic evaluation of alternative rehabilitation strategies for every needs section identified in PINS,
2. For a given set of annual budgets it produces an optimum (based on effectiveness maximization) priority program of pavement improvements for a programming period of up to 10 years, and
3. For a given set of annual performance standards it produces an optimum (based on cost minimization) financial plan (i.e., annual budgets required) for a programming period of up to 10 years.

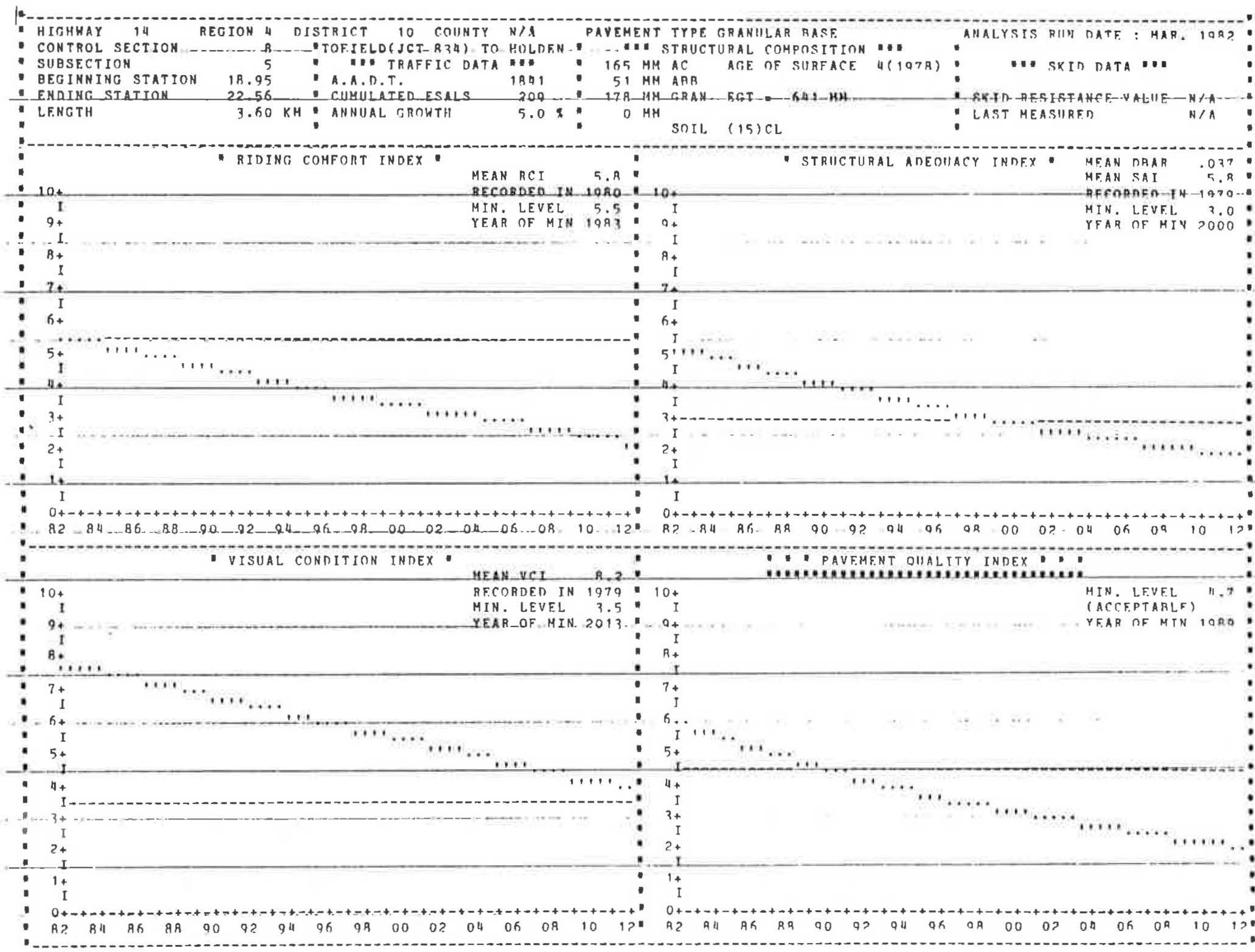

FIGURE 3 Sample of sectional PINS output.

RANK	DISTRICT	CONTROL SECTION		CONTROL SECTION DESCRIPTION	INVENTORY SECTION	BEGIN STATION	END STATION	RCI	VCI	SAI	PQI	SKID
1	4	1-12		JCT 9 TO JCT 21	1	15.48	18.76	0.0	2.7	2.8	1.2	N/A
~~2~~	~~4~~	~~10-08~~		~~ECL DRUMHELLER TO EAST COUL~~	~~1~~	~~0.00~~	~~5.79~~	~~0.0~~	~~1.9~~	~~2.3~~	~~1.2~~	~~N/A~~
3	3	10-08		ECL DRUMHELLER TO EAST COUL	3	6.92	10.23	0.0	2.4	2.5	1.2	N/A
4	3	10-08		ECL DRUMHELLER TO EAST COUL	6	14.50	21.27	0.8	2.5	2.7	1.5	N/A
5	3	10-08		ECL DRUMHELLER TO EAST COUL	2	5.79	6.92	1.2	4.6	3.2	1.8	N/A
6	4	1-14		JCT 21 TO JCT 956	1	0.00	11.44	1.3	2.3	1.1	1.4	N/A
7	4	9-04		JCT 21 TO SCL DRUMHELLER	1	0.00	0.39	1.7	1.4	3.6	1.8	N/A
~~8~~	~~3~~	~~10-08~~		~~ECL DRUMHELLER TO EAST COUL~~	~~4~~	~~10.23~~	~~11.71~~	~~1.8~~	~~4.7~~	~~1.4~~	~~1.8~~	~~N/A~~
9	3	21-14		JCT 9 TO JCT 27	12	25.74	28.32	1.8	5.0	3.9	2.2	N/A
10	4	2A-04		JCT 2 TO JCT 23	5	21.77	22.35	2.5	5.1	4.8	2.8	N/A
11	3	21-14		JCT 9 TO JCT 27	22	41.05	42.64	2.6	1.8	0.1	1.4	N/A
12	3	21-14		JCT 9 TO JCT 27	23	42.64	44.05	2.6	2.6	0.1	1.6	N/A
13	4	2A-04		JCT 2 TO JCT 23	1	0.00	18.78	2.7	0.8	2.0	1.7	N/A
~~14~~	~~4~~	~~21-14~~		~~JCT 9 TO JCT 27~~	~~21~~	~~39.47~~	~~41.05~~	~~2.9~~	~~1.6~~	~~0.0~~	~~1.4~~	~~N/A~~
15	3	21-16		JCT 27 TO JCT 42	2	2.48	3.07	2.9	1.9	0.2	1.5	N/A
16	3	21-16		JCT 27 TO JCT 42	1	0.00	2.48	3.0	1.9	0.1	1.5	N/A
17	4	22-16		JCT 1 TO CREMONA	7	9.28	10.93	3.0	4.2	0.2	1.9	N/A
18	4	2A-10		S.CROSSFIELD TO N.CROSSFIEL	1	0.00	2.04	3.1	1.4	1.9	1.7	N/A
19	3	21-14		JCT 9 TO JCT 27	20	38.54	39.47	3.1	2.0	0.1	1.5	N/A
~~20~~	~~4~~	~~1-10~~	~~WBD~~	~~BLACKFT TR(CAL) TO JCT 9~~	~~3~~	~~7.96~~	~~9.12~~	~~3.2~~	~~4.8~~	~~1.4~~	~~2.4~~	~~N/A~~
21	4	1A-08		ECL CALGARY TO JCT 1	2	13.39	14.64	3.2	5.4	0.2	2.2	N/A
22	4	2-12	SBD	DEWINTON INTERCHANGE	1	27.51	28.40	3.2	5.4	0.6	2.2	N/A
23	3	21-14		JCT 9 TO JCT 27	5	9.98	13.95	3.2	3.7	0.6	1.9	N/A
24	4	27-06		ECL SUNDRE TO JCT 2	3	25.20	26.26	3.2	4.8	0.1	2.1	N/A
25	4	1-12	EBD	JCT 9 TO JCT 21	1	0.00	0.37	3.3	4.2	0.9	2.0	N/A
~~26~~	~~4~~	~~10-04~~		~~JCT 840 TO 16 AVE NW CALGAR~~	~~3~~	~~12.16~~	~~13.63~~	~~3.3~~	~~2.5~~	~~0.3~~	~~1.6~~	~~N/A~~
27	3	21-14		JCT 9 TO JCT 27	10	24.14	24.46	3.3	5.5	4.3	3.3	N/A
28	4	1-10	EBD	BLACKFT TR(CAL) TO JCT 9	6	19.28	26.61	3.4	3.9	0.3	1.9	N/A
29	4	1-12	EBD	JCT 9 TO JCT 21	2	0.37	6.05	3.4	4.2	1.6	2.4	N/A
30	4	22-16		JCT 1 TO CREMONA	6	8.50	9.28	3.4	4.2	0.7	2.0	N/A
31	4	1-12	WBD	JCT 9 TO JCT 21	4	1.83	2.99	3.5	4.0	0.9	2.0	N/A
~~32~~	~~4~~	~~1A-02~~		~~JCT 1 E. CANMORE TO JCT 1X~~	~~4~~	~~18.41~~	~~26.95~~	~~3.5~~	~~5.5~~	~~4.4~~	~~2.5~~	~~N/A~~
33	3	9-06		NCL DRUMHELLER TO JCT 27	2	2.43	2.67	3.5	5.9	4.7	3.6	N/A
34	3	21-14		JCT 9 TO JCT 27	11	24.46	25.74	3.5	5.4	2.8	3.1	N/A
35	4	22-16		JCT 1 TO CREMONA	5	8.09	8.50	3.5	4.2	0.3	2.0	N/A
36	4	1-10	EBD	BLACKFT TR(CAL) TO JCT 9	5	17.36	19.28	3.6	4.9	0.4	2.2	N/A
37	4	1-10	WBD	BLACKFT TR(CAL) TO JCT 9	5	14.01	19.08	3.6	5.3	0.5	2.3	N/A
~~38~~	~~4~~	~~2A-04~~		~~JCT 2 TO JCT 23~~	~~4~~	~~21.54~~	~~21.77~~	~~3.4~~	~~4.4~~	~~2.2~~	~~2.0~~	~~N/A~~

FIGURE 4 Sample RCI ranking list produced by PINS.

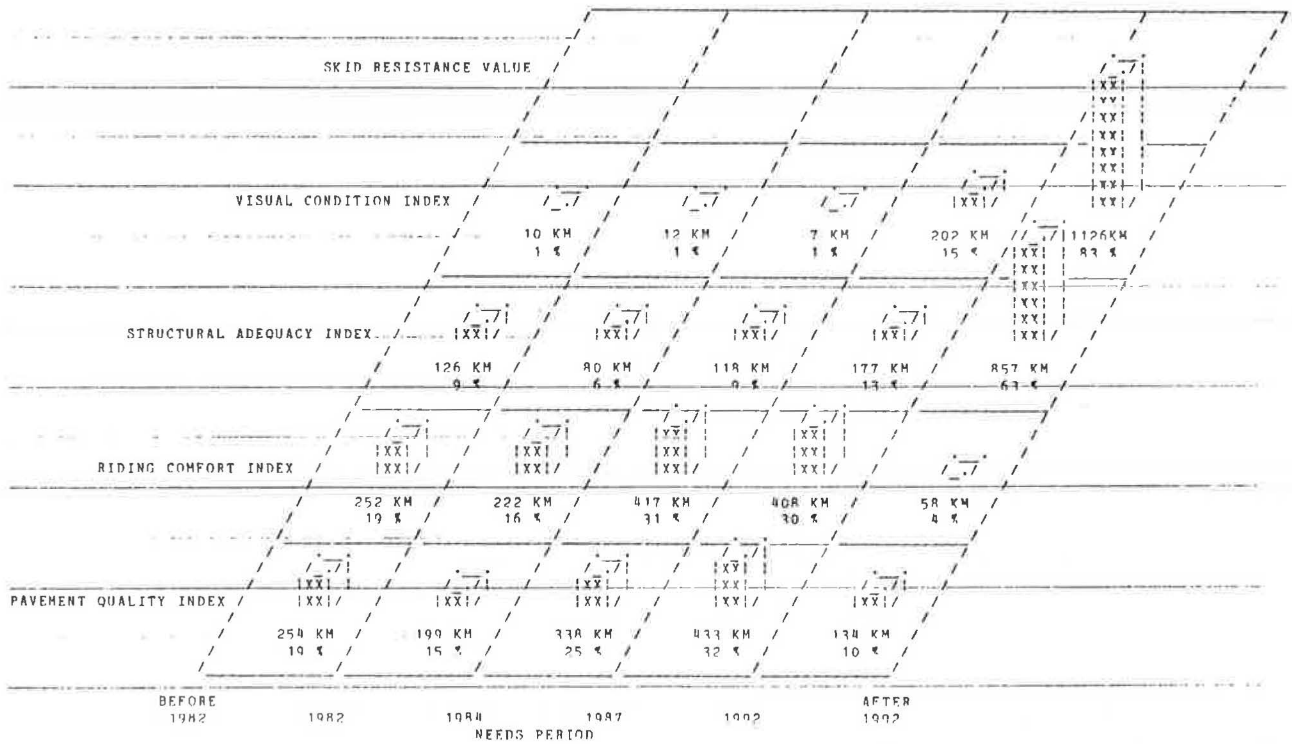

FIGURE 5 Sample three-dimensional histogram produced by PINS.

RIPPS, which is described in more detail by Cheetham et al. (6), is shown in outline form in Figure 7. It has two main subsystems: rehabilitation analysis (REHAB) and priority programming (PRIORITY), also shown in outline form in Figures 8 and 9.

The REHAB subsystem (Figure 8) performs rehabilitation analyses for each section and involves generation of rehabilitation alternatives, performance prediction of the alternatives, economic analyses, and effectiveness analyses.

The PRIORITY subsystem (Figure 9) uses the output files from REHAB and conducts priority or financial

NEEDS

DIST	HWY - CONTROL SECTION	CONTROL SECTION DESCRIPTION	INV. SECTION	BEGIN KM.	END KM.	PAVEMENT QUALITY INDEX	RIDING COMFORT INDEX	STRUCTURAL ADEQUACY INDEX	VISUAL CONDITION INDEX	SKID RESISTANCE VALUE
10	14-14	JCT 870 TO JCT 41	4	25.18	29.33	4.6(1985)	5.4	5.7(2000)	3.9(1990)	N/A
10	14-16	JCT 41 TO SASK BDY	11	42.06	44.36	4.4(1984)	5.4	4.3(1995)	4.5(1993)	N/A
10	14-16	JCT 41 TO SASK BDY	12	44.36	44.88	5.0(1987)	5.4	5.3(2002)	5.5(1999)	N/A
10	14-16	JCT 41 TO SASK BDY	13	44.88	44.99	5.0(1987)	5.4	5.3(2002)	5.5(1999)	N/A
10	15- 8	JCT 45 TO JCT 16	1	0.00	6.11	4.7(1985)	5.5	5.9(2004)	4.0(1990)	N/A
10	15- 8	JCT 45 TO JCT 16	6	21.03	22.06	5.2(1989)	5.5	3.5(1990)	7.6(2013)	N/A
10	15- 8	JCT 45 TO JCT 16	7	22.06	25.42	5.1(1987)	5.4	4.3(1998)	6.8(2003)	N/A
10	16-22	ELK ISLAND ACCESS TO JCT 15	4	7.42	8.48	5.1(1988)	5.5	6.4(1998)	4.9(1999)	N/A
10	16-24	JCT 15 TO JCT 36	34	18.09	18.63	6.0(1993)	5.4	7.7(2004)	7.1(2013)	N/A
10	16-30	JCT 41 TO SASK BDY	4	4.28	5.89	4.3(1984)	5.5	3.2(1986)	5.4(1998)	N/A
10	16-30	JCT 41 TO SASK BDY	14	39.68	46.29	4.1(1983)	5.4	3.3(1986)	4.4(1993)	N/A
10	16-30	JCT 41 TO SASK BDY	18	53.63	57.51	5.4(1990)	5.4	6.5(1998)	6.0(2006)	N/A
11	28-10	JCT 36N TO JCT 28A	3	5.34	7.02	3.6(1981)	5.4	2.5(1983)	4.0(1990)	N/A
11	28-16	JCT 41 TO JCT 28A	3	3.04	4.88	4.5(1984)	5.5	2.5(1982)	6.6(2003)	N/A
11	28-16	JCT 41 TO JCT 28A	5	7.61	10.81	4.7(1985)	5.5	3.0(1985)	6.5(2003)	N/A
11	28-16	JCT 41 TO JCT 28A	7	14.46	23.76	5.2(1988)	5.5	4.3(1996)	6.7(2004)	N/A
11	28-20	ARDMORE TO JCT 55 @ COLD L.	16	31.62	32.49	5.3(1989)	5.4	5.4(1999)	6.2(2000)	N/A
11	8A- 4	JCT 28 TO JCT 28	2	15.61	17.22	4.4(1984)	5.4	4.0(1990)	4.7(1995)	N/A
11	8A- 4	JCT 28 TO JCT 28	4	18.99	20.13	4.8(1986)	5.5	5.3(1994)	4.7(1995)	N/A
11	8A- 4	JCT 28 TO JCT 28	12	36.69	40.00	4.3(1984)	5.4	3.3(1987)	5.2(1997)	N/A
10	36-20	JCT 16 TO JCT 45	3	16.54	20.93	4.1(1984)	5.4	2.2(1983)	6.0(1999)	N/A
11	41-22	JCT 45W TO JCT 28	4	21.80	22.98	4.4(1984)	5.4	2.7(1983)	6.2(2002)	N/A
10	45- 4	JCT 15 TO ANDREW ACCESS	4	10.99	14.34	5.2(1988)	5.5	8.1(2006)	4.2(1992)	N/A
10	45- 4	JCT 15 TO ANDREW ACCESS	5	14.34	15.56	5.1(1987)	5.4	7.9(2006)	4.2(1992)	N/A
10	45- 4	JCT 15 TO ANDREW ACCESS	6	15.56	16.49	5.3(1989)	5.4	8.7(2013)	4.5(1997)	N/A
10	45- 6	ANDREW ACCESS TO TWO HILLS	5	5.25	6.98	4.1(1984)	5.5	1.9(1983)	6.4(2000)	N/A
10	45- 6	ANDREW ACCESS TO TWO HILLS	12	23.88	24.36	3.7(1981)	5.4	1.7(1979)	5.8(2000)	N/A
10	45- 8	TWO HILLS TO JCT 41	4	10.51	13.71	4.7(1985)	5.4	3.9(1994)	5.7(1999)	N/A
11	55-12	JCT 63 TO JCT 36	12	11.70	11.91	5.5(1989)	5.4	6.3(2007)	6.4(2002)	N/A
11	55-12	JCT 63 TO JCT 36	17	19.65	21.45	5.3(1988)	5.4	6.1(2004)	5.9(1999)	N/A
11	55-12	JCT 63 TO JCT 36	18	21.45	23.43	5.1(1987)	5.5	5.3(2001)	5.6(1999)	N/A
11	63- 2	W.ATMORE TO N WANDERING R.	6	10.36	11.26	3.9(1982)	5.4	2.1(1981)	5.6(1998)	N/A
11	63- 2	W.ATMORE TO N WANDERING R.	7	11.26	12.12	4.1(1983)	5.4	2.5(1983)	5.6(1998)	N/A
11	63- 2	W.ATMORE TO N WANDERING R.	8	12.12	12.95	2.8(1978)	5.5	0.9(1978)	5.0(1997)	N/A
11	63- 2	W.ATMORE TO N WANDERING R.	9	12.95	13.81	4.3(1984)	5.5	3.3(1987)	5.1(1997)	N/A

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FIGURE 6 Example output of needs report by PINS.

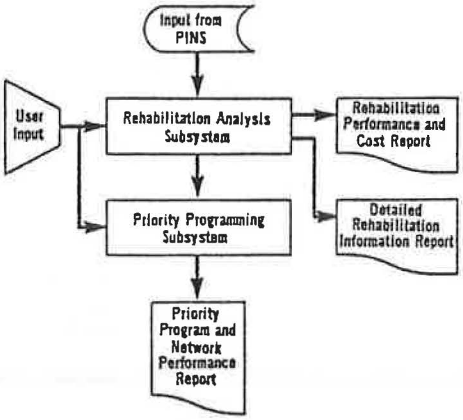


FIGURE 7 Outline of RIPPS.

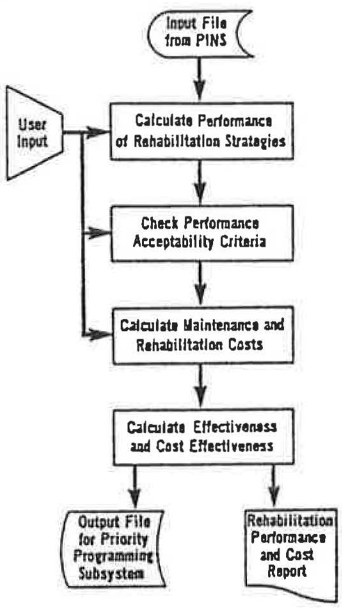


FIGURE 8 REHAB subsystem.

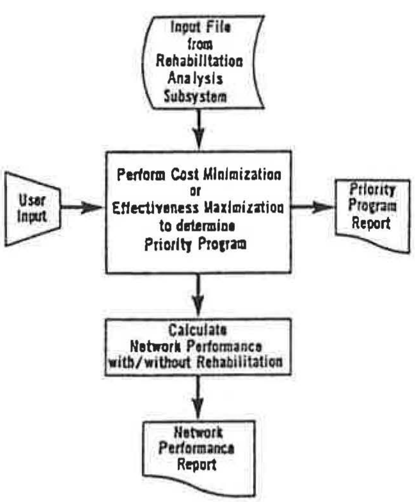


FIGURE 9 PRIORITY subsystem.

planning analysis by using a heuristic optimization procedure employing marginal cost-effectiveness analyses. The priority analysis can either be run in cost minimization or effectiveness maximization modes.

REHAB Subsystem

Alternative Selection

The subsystem allows for the analysis of up to five different types of alternatives for each inventory section analyzed. The alternative types from which the five can be selected are as follows: overlay, milling, milling plus overlay, recycle, recycle plus overlay, recycle plus seal coat, widening plus surfacing, widening plus overlay, heater plane plus overlay, heater plane plus seal coat, and reconstruction.

The selection of the five alternatives for each section using this master list can be accomplished by using one of the four methods built into the subsystem:

1. Defining a fixed set for the network;
2. Specifying different alternative sets for different sections, with default to the fixed set for unspecified sections;
3. Using an automatic alternative type set selection procedure; and
4. Specifying different alternative sets for different sections, with default to the automatic selection procedure for unspecified sections.

Performance Prediction

The performance of the alternatives is predicted by using the same recursive models used in PINS. For some of the alternatives, the models have been modified to reflect the difference in performance expected from these alternatives. The performance prediction models used in PINS and RIPPS are described in detail elsewhere (2,4).

Economic and Effectiveness Analyses

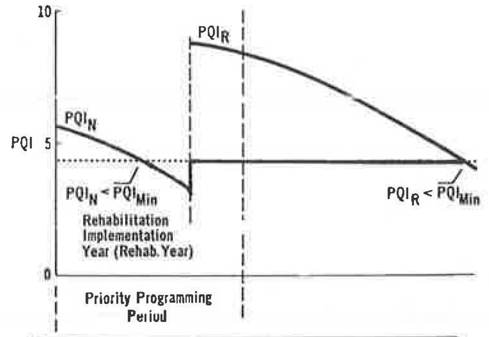
Each rehabilitation alternative that meets the minimum life constraints is subject to an economic analysis. This involves calculation of the capital cost and expected annual maintenance cost streams for a 25-year period from the start of the priority programming period. Inflation of the rehabilitation and maintenance costs can also be considered through the input of an inflation rate. The present worth of the total costs (rehabilitation plus maintenance) is calculated for use in determining the cost-effectiveness of the strategy.

The effectiveness of each rehabilitation alternative is also calculated. It is related to the difference between the rehabilitated PQI performance curve and the nonrehabilitated performance curve, as shown in Figure 10. The difference between the two curves in each year is weighted by annual average daily traffic (AADT) and section length and summed over time to determine the total effectiveness of the alternative.

The total effectiveness is used to calculate the cost-effectiveness for the alternative. This is similar to a benefit-cost ratio, except that the benefits (total effectiveness) are not in terms of dollars.

Output Reports

The REHAB subsystem, in the network mode, produces a rehabilitation information report for each section



$$\text{EFFECTIVENESS} = \sum_{\text{Rehab. Year}}^{PQI_R<PQI_{Min}} (PQI_R - PQI_N^*) \bullet \text{AADT} \bullet \text{Length} - \sum_{PQI_N<PQI_{Min}}^{\text{Rehab. Year}} (PQI_{Min} - PQI_N) \bullet \text{AADT} \bullet \text{Length}$$

* where PQI_N is the non-rehabilitation index value or the critical value, which ever is greater

FIGURE 10 Effectiveness calculation.

analyzed, as shown in Figure 11. This report consists of an information matrix of 5 columns of rehabilitation alternative types and 10 rows of implementation years, which represent a maximum of 50 strategies (alternative/implementation year combinations) that can be analyzed for a section. For each information cell in the table the alternative description, rehabilitation capital cost, present worth of the maintenance costs, and the cost-effectiveness are given for the strategy.

In each implementation year the alternative with the highest cost-effectiveness is highlighted with a line of dashes above and below the information cell. The strategy with the overall highest cost-effectiveness for the section is highlighted with a line of asterisks above and below the information cell.

In the detail mode a rehabilitation performance report is produced for each section, rehabilitation alternative, and implementation year combination analyzed. Figure 12 shows an example rehabilitation information report from a detailed analysis. In this analysis mode the output consists solely of a report of this type for each section, alternative, and implementation year specified for detail analysis.

```
* HIGHWAY    16      REGION 5  DISTRICT    8  COUNTY  N/A     PAVEMENT TYPE GRANULAR BASE           ANALYSIS RUN DATE : MAR. [illegible]
* CONTROL SECTION          6    JCT 47 TO JCT 32          *   *** STRUCTURAL COMPOSITION ***   *
* SUBSECTION               6    *   *** TRAFFIC DATA ***  *  406 MM AC    AGE OF SURFACE 9(1974) *    *** SKID DATA ***
* BEGINNING STATION    11.10    * A.A.D.T.          4980  *  107 MM ABD                          *
* ENDING STATION       11.50    * CUMULATED ESALS   1950  *  406 MM GRAN  EGT = 1504 MM          * SKID RESISTANCE VALUE   N/A
* LENGTH             0.40 KM    * ANNUAL GROWTH    5.0 %  *    0 HH                              * LAST MEASURED           N/A
* PAVEMENT WIDTH         N/A    *                         *            SOIL  (15)CL              *
*------------------------------------------------------------------------------------------------------------------------
* NEEDS YEAR WITHOUT REHABILITATION : PQI : 1983   RCI : 1981   SAI : 1990   VCI : 1990
*------------------------------------------------------------------------------------------------------------------------
*   -------------------------- *                         *                          *                          *
* 1 60 MM OVERLAY              *  MILL 20 & 60 MM O/L    *                          *                          *
* 9 REHAB COST       17123.    *  REHAB COST    18753.   *                          *                          *
* 8 MAINT COST        5752.    *  MAINT COST     5752.   *                          *                          *
* 3 COST-EFF.           3.7    *  COST-EFF.        3.6   *                          *                          *
*   -------------------------- *                         *                          *                          *
* 1 60 MM OVERLAY              *  MILL 20 & 60 MM O/L    *                          *                          *
* 9 REHAB COST       17123.    *  REHAB COST    18753.   *                          *                          *
* 8 MAINT COST        5643.    *  MAINT COST     5643.   *                          *                          *
* 4 COST-EFF.           4.1    *  COST-EFF.        3.9   *                          *                          *
*   -------------------------- *                         *                          *                          *
* 1 60 MM OVERLAY              *  MILL 20 & 60 MM O/L    *                          *                          *
* 9 REHAB COST       17123.    *  REHAB COST    18753.   *                          *                          *
* 8 MAINT COST        5552.    *  MAINT COST     5552.   *                          *                          *
* 5 COST-EFF.           4.1    *  COST-EFF.        3.9   *                          *                          *
*   -------------------------- *                         *                          *                          *
* 1 60 MM OVERLAY              *  MILL 20 & 60 MM O/L    *                          *                          *
* 9 REHAB COST       17123.    *  REHAB COST    18753.   *                          *                          *
* 8 MAINT COST        5479.    *  MAINT COST     5479.   *                          *                          *
* 6 COST-EFF.           4.5    *  COST-EFF.        4.3   *                          *                          *
*   -------------------------- *                         *                          *                          *
* 1 60 MM OVERLAY              *  MILL 20 & 60 MM O/L    *                          *                          *
* 9 REHAB COST       17123.    *  REHAB COST    18753.   *                          *                          *
* 8 MAINT COST        5427.    *  MAINT COST     5427.   *                          *                          *
* 7 COST-EFF.           4.9    *  COST-EFF.        4.6   *                          *                          *
*   -------------------------- *                         *                          *                          *
* 1 60 MM OVERLAY              *  MILL 20 & 60 MM O/L    *                          *                          *
* 9 REHAB COST       17123.    *  REHAB COST    18753.   *                          *                          *
* 8 MAINT COST        5391.    *  MAINT COST     5391.   *                          *                          *
* 8 COST-EFF.           5.2    *  COST-EFF.        4.9   *                          *                          *
*   -------------------------- *                         *                          *                          *
* 1 60 MM OVERLAY              *  MILL 20 & 60 MM O/L    *                          *                          *
* 9 REHAB COST       17123.    *  REHAB COST    18753.   *                          *                          *
* 8 MAINT COST        5377.    *  MAINT COST     5377.   *                          *                          *
* 9 COST-EFF.           5.6    *  COST-EFF.        4.7   *                          *                          *
*   -------------------------- *                         *  ------------------------ *                          *
* 1 60MM O/L & 20 MM LEV       *  MILL 20 & 60 MM O/L    *  RECYC.50 & 60 MM O/L    *  RECYC.50 & SEAL COAT    *
* 9 REHAB COST       22830.    *  REHAB COST    18753.   *  REHAB COST  [illegible] *  MIN. LIFE CONSTRAINT    *
* 9 MAINT COST        5382.    *  MAINT COST     5382.   *  MAINT COST  [illegible] *  NOT MET FOR             *
* 0 COST-EFF.           4.3    *  COST-EFF.        5.0   *  COST-EFF.   [illegible] *  PRI  SAI                *
*                              *                         *  ************************ *                          *
* 1 60MM O/L & 20 MM LEV       *  MILL 20 & 60 MM O/L    *  RECYC.50 & 60 MM O/L    *  RECYC.50 & SEAL COAT    *
* 9 REHAB COST       22830.    *  REHAB COST    18753.   *  REHAB COST  [illegible] *  MIN. LIFE CONSTRAINT    *
* 9 MAINT COST        5412.    *  MAINT COST     5412.   *  MAINT COST  [illegible] *  NOT MET FOR             *
* 1 COST-EFF.           4.5    *  COST-EFF.        5.3   *  COST-EFF.   [illegible] *  PRI  SAI                *
*                              *                         *  ************************ *                          *
* 1 60MM O/L & 20 MM LEV       *  MILL 20 & 60 MM O/L    *  RECYC.50 & 60 MM O/L    *  RECYC.50 & SEAL COAT    *
* 9 REHAB COST       22830.    *  REHAB COST  [illegible] *  REHAB COST  [illegible] *  MIN. LIFE CONSTRAINT    *
* 9 MAINT COST        5466.    *  MAINT COST  [illegible] *  MAINT COST  [illegible] *  NOT MET FOR             *
* 2 COST-EFF.           4.0    *  COST-EFF.   [illegible] *  COST-EFF.          6.0  *  PRI  SAI                *
```

FIGURE 11 Rehabilitation information report.

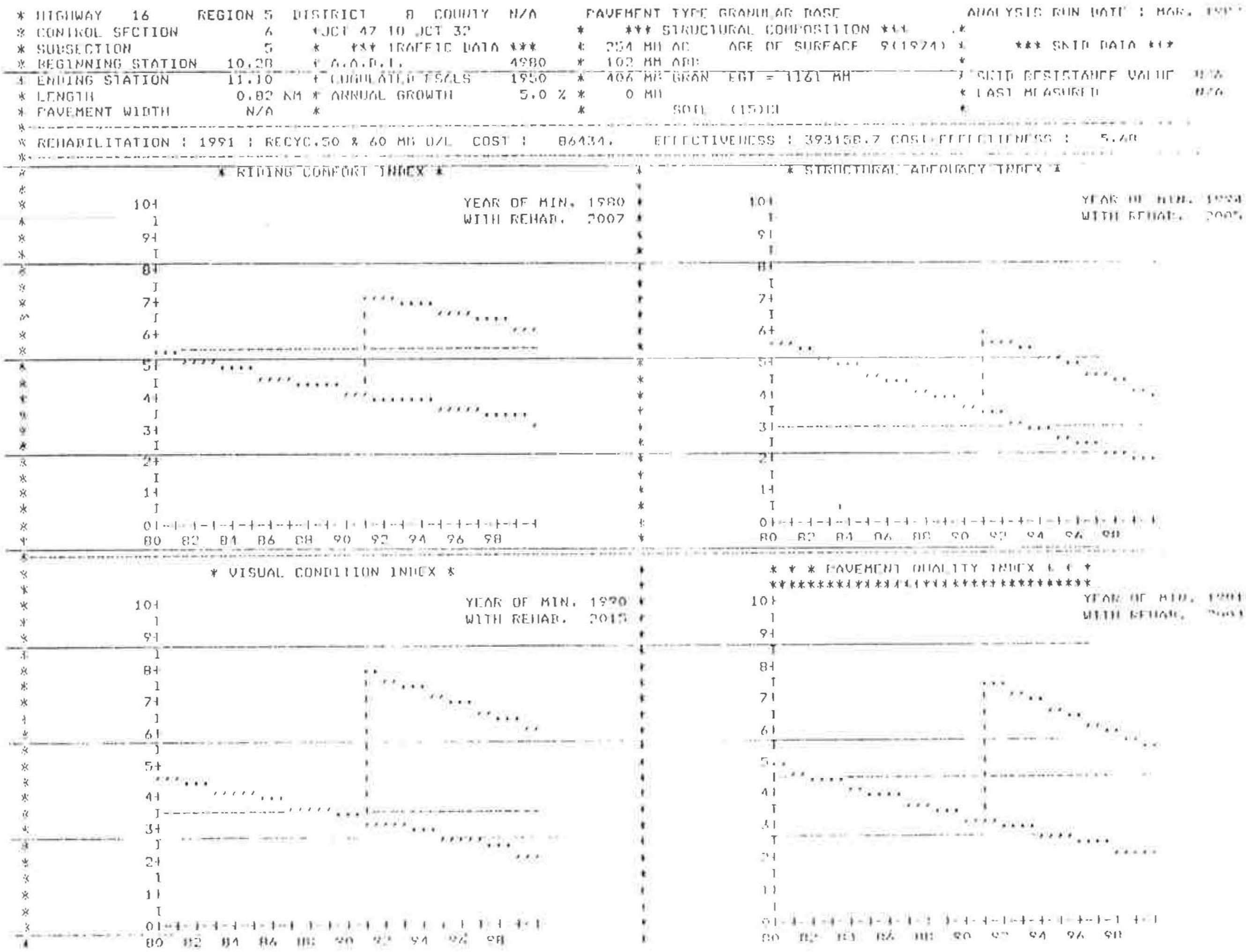


FIGURE 12 Detailed rehabilitation information report.

The report consists of four performance plots similar to those produced by the PINS system. Each plot shows the performance with and without rehabilitation and the needs years with and without rehabilitation. A detailed description of subsystem REHAB and its outputs is given elsewhere (7).

PRIORITY Subsystem

The PRIORITY subsystem forms the priority or financial planning analysis part of the RIPPS system. A heuristic procedure has been developed specifically for RIPPS for the purpose of optimizing investments. The procedure uses the marginal analysis concept and can be employed for either cost minimization or effectiveness maximization purposes or both.

The heuristic procedure developed eliminates the problems related to solving large networks by using linear or integer programming techniques while producing close-to-optimum solutions in an efficient way, as subsequently discussed in the paper. It is believed that this new procedure is a distinct advance over the mathematical programming techniques previously used by the authors of this paper, and others, for optimization and priority analyses.

Cost Minimization Method

Cost minimization is based on highest cost-effectiveness and marginal cost-effectiveness analyses. Strategies are selected on an annual basis because the implementation of an alternative in a given year affects the performance in all subsequent years. When a strategy is selected for a section, the marginal cost-effectiveness is calculated for any other strategy available for that section for that year. Strategies in other years for the section are then eliminated from further consideration. The following performance constraints are built into the procedure (others can easily be incorporated):

1. Annual average network PQI level specified, and
2. Annual percentage of network allowed to be below the minimum acceptable PQI level.

Cost minimization is a budgeting tool because the annual expenditures required to achieve a desired performance level are produced. The program can be rerun with different performance constraints to test the effects of desired performance levels on the required funding level.

Effectiveness Maximization Method

The effectiveness maximization method is also based on highest cost-effectiveness and marginal cost-effectiveness analyses. In this method, however, all of the implementation years are treated simultaneously because the implementation of an alternative affects only the budget for the implementation year. When a strategy is selected for a section, the marginal cost-effectiveness is calculated for all other strategies available for the section. The constraints imposed on this analysis method are simply the annual available rehabilitation budgets.

This method is not a budgeting tool, but rather it is a programming tool for determining rehabilitation programs. The end result of this method is a program of rehabilitation strategies to be implemented that will provide the maximum effectiveness for the available funding levels. The program can be rerun with different budget levels to test the effects of funding levels on the resulting network performance.

The marginal analysis optimization procedure briefly described in the preceding paragraphs has been compared with the linear programming (LP) technique (using MPSX package) by using three different data sets that were available from other projects. The pavement networks involved were the region of Waterloo--rural (23 sections) and urban (66 sections)--and the borough of Scarborough (63 sections). In the effectiveness maximization mode the marginal analysis procedure was 93.1 percent of the LP for the Waterloo rural, 96.7 percent of the LP for the Waterloo urban, and 97.4 percent of the LP for the Scarborough project. The resulting priority lists were almost identical, with the exception of one or two projects. Most of the difference was caused by the fractional solutions that LP produced.

Similarly, in the cost minimization mode, the one test conducted using Waterloo rural data resulted in better effectiveness using marginal analysis, but the total dollars spent was higher. This is simply because of the way the marginal analysis procedure is set up, where effectiveness is also being considered while trying to minimize the cost, which was not the case in the LP formulation. Hence marginal analysis in the test run spent more, but the return for the dollars spent was higher.

These marginal analysis tests were conducted by using a PDP-11-34 minicomputer, and in the maximization mode the following CPU times were observed:

1. Waterloo rural (23 sections, 2 alternative strategies per year, 5-year period): 0.22 min;
2. Waterloo urban (66 sections, 2 alternative strategies per year, 5-year period): 0.89 min; and
3. Scarborough (63 sections, 2 alternative strategies per year, 10-year period): 1.42 min.

Drs. Moore and Magazine of the University of Waterloo, who have been involved in the assessment of the marginal analysis approach, both believe that this procedure is appropriate for the purposes of optimizing pavement investments within the context of pavement management. (Note that this information is from correspondence from Dr. J.B. Moore to Dr. M.A. Karan, July 7, 1983.)

Output Reports

Three types of output reports can be obtained from the PRIORITY subsystem:

1. Priority programming report by highway,
2. Priority programming report by year, and
3. Performance summary report.

REG	DIST	HWY - CONTROL SECTION	CONTROL SECTION DESCRIPTION	INV. SECT.	BEGIN KM.	END KM.	PCI NEED YEAR	REHAB YEAR	REHABILITATION ALTERNATIVE	COST	EFFECT	COST-EFF
5	8	16- 2	JASPER PK BDY TO OBED	1	0.00	4.26	1983	1989	MILL 20 & 60 MM O/L	267647	149.5	5.72
				2	4.26	13.44	1985	1986	MILL 20 & 60 MM O/L	498227	203.5	3.64
				3	13.44	25.87	1995	N/A				
				4	25.87	54.03	1994	N/A				
				5	54.03	54.98	1996	1992	60 MM OVERLAY	63086	21.1	4.12
5	8	16- 4	OBED TO JCT 47	1	0.00	3.22	1997	N/A				
				2	3.22	8.05	1992	N/A				
				3	8.05	15.03	1996	N/A				
				4	15.03	24.79	1997	N/A				
				5	24.79	28.35	1987	1991	60 MM OVERLAY	225150	165.7	8.16
				6	28.35	31.54	1995	1989	MILL 20 & 60 MM O/L	200421	78.6	4.07
				7	31.54	34.19	1997	N/A				
				8	34.19	36.04	1994	N/A				
				9	36.04	41.67	1994	N/A				
				10	41.67	41.99	1991	1990	60 MM OVERLAY	19274	6.4	3.57
				11	41.99	42.65	1990	1983	MILL 20 & 60 MM O/L	30942	6.1	1.51
				12	42.65	45.78	1990	N/A				
				13	45.78	49.24	1990	1983	MILL 20 & 60 MM O/L	162216	42.2	1.99
5	8	16- 6	JCT 47 TO JCT 32	1	0.00	1.19	1992	1990	60 MM OVERLAY	71677	27.8	4.30
				2	1.19	8.27	1989	1985	60 MM OVERLAY	334133	122.7	3.08
				3	8.27	10.09	1980	1991	100 MM OVERLAY	191841	99.1	6.49
				4	10.09	10.28	1980	1992	100 MM OVERLAY	21028	10.2	6.36
				5	10.28	11.10	1981	1990	RECYC.50 & 60 MM O/L	82318	37.3	5.39
				6	11.10	11.50	1983	1991	RECYC.50 & 60 MM O/L	42162	20.9	6.19
				7	11.50	13.95	1987	1990	60 MM OVERLAY	147570	61.2	4.57
				8	13.95	25.76	1984	N/A				
				9	25.76	42.25	1995	N/A				
5	8	16- 6 EB	EDSON ONE-WAY COUPLET	10	6.07	11.75	1989	1992	MILL 20 & 60 MM O/L	413113	397.2	12.42
5	8	16- 8	JCT 32 TO CHIP LAKE	1	0.00	7.69	1990	1988	60 MM OVERLAY	420127	195.2	4.67
				2	7.69	8.05	1985	1989	60 MM OVERLAY	20651	6.3	3.13
				3	8.05	8.61	1987	1991	60 MM OVERLAY	35416	12.4	4.01
				4	8.61	9.85	1988	1984	MILL 20 & 60 MM O/L	61041	14.5	1.92
				5	9.85	10.72	1985	1987	60 MM OVERLAY	45267	14.4	2.96
				6	10.72	10.97	1986	1990	60 MM OVERLAY	15058	5.3	3.81
				7	10.97	12.20	1987	1985	MILL 20 & 60 MM O/L	63577	17.4	2.31
				8	12.20	13.55	1988	1983	MILL 20 & 60 MM O/L	63292	14.6	1.76
				9	13.55	14.53	1988	1984	MILL 20 & 60 MM O/L	48242	11.1	1.86
				10	14.53	16.65	1988	1985	60 MM OVERLAY	100051	28.2	2.36
				11	16.65	19.55	1987	N/A				
				12	19.55	23.33	1989	N/A				
				13	23.33	27.40	1992	N/A				
				14	27.40	35.88	1990	1984	60 MM OVERLAY	381148	103.9	2.16
5	8	16-10	CHIP LAKE TO JCT 22	1	0.00	1.90	1981	1987	60 MM OVERLAY	98859	32.4	3.00
				3	3.51	6.05	1981	1982	100 MM OVERLAY	289641	112.5	3.97

FIGURE 13 Priority program report by highway.

REG	DIST	HWY CONTROL SECTION	CONTROL SECTION DESCRIPTION	INV. SECTION	BEGIN KM.	END KM.	REHABILITATION ALTERNATIVE	COST	EFFECT	COST EFF
5	8	16- 4	OBED TO JCT 47	10	41.67	41.99	60 MM OVERLAY	19274	6.4	[illegible]
5	8	16- 6	JCT 47 TO JCT 32	1	0.00	1.19	60 MM OVERLAY	71677	27.8	4.30
5	8	16- 6	JCT 47 TO JCT 32	5	10.20	11.10	RECYC.50 & 60 MM O/L	82318	37.3	[illegible]
5	8	16- 6	JCT 47 TO JCT 32	7	11.50	13.95	60 MM OVERLAY	147570	61.2	4.57
5	8	16- 8	JCT 32 TO CHIP LAKE	6	10.72	10.97	60 MM OVERLAY	15058	5.3	3.81
5	8	16-10	CHIP LAKE TO JCT 22	6	10.84	11.55	100 MM OVERLAY	71275	27.5	4.55
5	8	16-10	CHIP LAKE TO JCT 22	7	11.55	12.45	100 MM OVERLAY	90349	34.9	4.56

FIGURE 14 Priority program report by year.

These three reports can be obtained for different levels of the network (i.e., for the province, sorted by region, and sorted by district).

Figure 13 shows an example of a priority programming report by highway. This report lists the sections in the order they appear in the input file and gives the rehabilitation strategies selected for implementation. This report can be produced for each separate district or for each separate region or for the whole network.

Figure 14 shows an example of a priority programming report by year. This type of report is repeated for each year in the programming period. Only those sections that have a rehabilitation strategy selected appear in this type of report. This report can also be produced for each separate district or for each separate region or for the whole network.

Figure 15 shows an example of a performance summary report. This report is produced for the network and can also be produced for each district or for each region. The annual costs shown in this report and the percentage budget usage have different meanings, depending on the mode of operation and the report level. For district and region reports, the annual costs are the total cost for the region or district and the percent budget usages are the annual costs as a percentage of the total annual network costs. For the network report in the effectiveness maximization mode, the costs are the total costs for the network and the budget usages are the annual total costs as a percentage of the input annual budget limitations. For the network report in the cost minimization mode, the costs are the total costs for the network and the budget usage has no meaning and is therefore not written in this case. Figure 16 shows an example output for the cost minimization mode.

The average annual PQIs and annual percentage below the minimum acceptable PQI are weighted by traffic volumes (AADT) and section lengths. These values are also plotted in the performance plots to give a visual representation of the performance trends with and without rehabilitation. A detailed description of the PRIORITY subsystem and its outputs is given elsewhere (8).

Application of RIPPS to Primary Highways in Alberta

RIPPS has been applied to portions of the primary network in Alberta. These test runs have been

YEAR	1983	1984	1985	1986	1987	1988	1989	1990	1991	1992
NETWORK COST	490459	490433	497762	498227	470711	484777	488719	497523	494572	497228
BUDGET USAGE (%)	98	98	99	99	94	96	97	99	98	99
NETWORK AVG PQI (WITH REHAB)	6.0	5.9	5.8	5.7	5.6	5.6	5.5	5.4	5.4	5.3
NETWORK AVG PQI (WITHOUT REHAB)	5.9	5.7	5.5	5.3	5.1	4.9	4.8	4.6	4.4	4.2
PERCENT < MIN PQI (WITH REHAB)	11	17	20	18	18	19	23	20	18	17
PERCENT < MIN PQI (WITHOUT REHAB)	12	19	19	22	26	29	36	43	52	55

FIGURE 15 Performance and cost summary for effectiveness maximization.

YEAR :	1983	1984	1985	1986	1987	1988	1989	1990	1991	1992
NETWORK COST :	614007	1414944	365956	875969	1401206	0	0	0	1588823	1017509
NETWORK AVG PQI : (WITH REHAB)	6.0	6.1	6.1	6.0	6.1	5.9	5.7	5.5	5.6	5.6
NETWORK AVG PQI : (WITHOUT REHAB)	5.9	5.7	5.5	5.3	5.1	4.9	4.8	4.6	4.4	4.2
PERCENT < MIN PQI : (WITH REHAB)	11	10	11	11	13	13	16	16	17	17
PERCENT < MIN PQI : (WITHOUT REHAB)	12	19	19	22	26	29	36	43	52	55

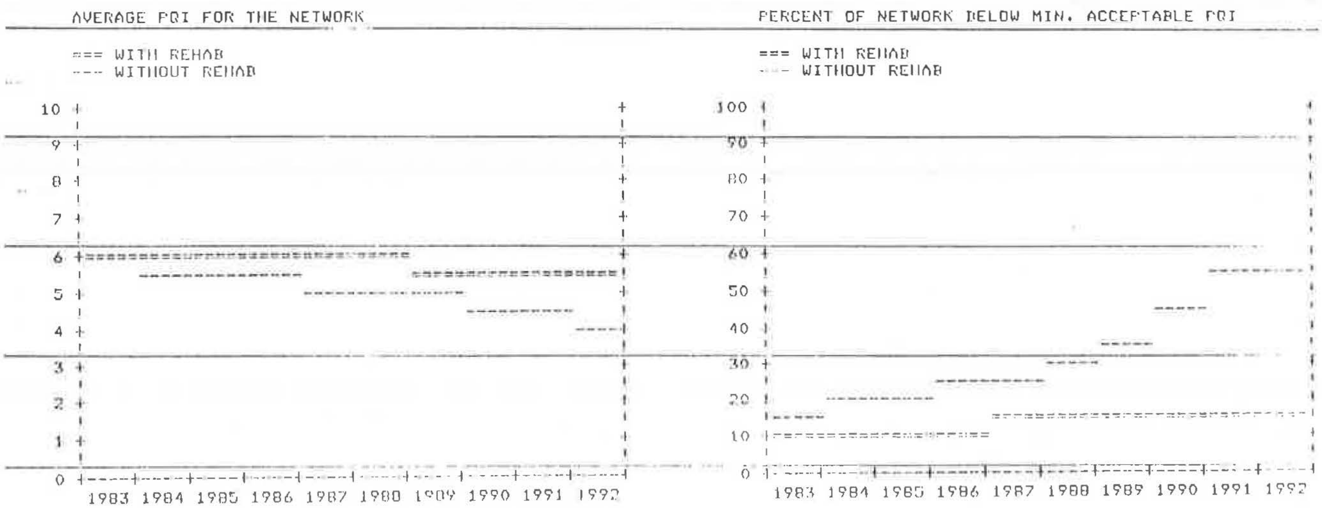


FIGURE 16 Performance and cost summary for cost minimization.

thoroughly evaluated and have resulted in some modifications that are now completed.

The computer programs for RIPPS have now been installed on Alberta Transportation's computer facilities in Edmonton. RIPPS is expected to be fully operational in late 1983 and used on a day-to-day basis along with PINS.

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Dynamic Decision Model for a Pavement Management System

RAM B. KULKARNI

ABSTRACT

Pavements represent gradually deteriorating structures for which observations of advance signs of impending failure are possible. Most agencies collect pavement condition data on a regular basis to identify such signs. However, neither the timing of occurrences of these signs nor the timing of actual failure following the signs can be predicted with certainty. Given this probabilistic behavior of pavements and the availability of periodic pavement condition data, a dynamic decision model is much more appropriate for such pavement management decisions as the selection of cost-effective pavement preservation actions and forecasting of future performance of a highway network. In this paper the basic structures of static and dynamic decision models, and a special class of dynamic decision models called a Markovian decision process, are described. Among the significant advantages of this model are reliable predictions of the future performance of a highway network and the identification of preservation actions that are generally less conservative (and less costly) than traditional choices of actions and yet maintain the network performance at prescribed standards. A successful application of the Markovian decision process to the pavement management system in Arizona is described.

A major objective of a pavement management system (PMS) is to assist highway managers in making consistent and cost-effective decisions related to maintenance and rehabilitation of pavements. An integral part of a PMS is a decision model that can be used to determine the optimum type and timing of preservation actions for different pavement segments. A dynamic decision model is described in this paper that permits the selection of a preservation action for a given pavement based on the most recent information on pavement condition.

Two factors have a major influence on the choice of a decision model to be used in a PMS. First, the future performance of a pavement cannot be predicted with certainty. Thus the behavior of pavements with time is probabilistic in nature. Because the future pavement condition is uncertain, the selection of a rehabilitation action appropriate for a given pavement at some future time is also uncertain. The second factor influencing the choice of a decision model is the periodic collection of pavement condition data. Most highway agencies conduct pavement condition surveys at some selected frequency (e.g., annually or biennially). Therefore the actual choice of a rehabilitation action at some future time can be made based on the most recent condition survey. Because the planning period for any rehabilitation action is relatively short (generally less than 2 years), it is unnecessary and inefficient to choose a rehabilitation action for a given pavement several years in advance.

These two factors strongly suggest that the decision model for a PMS should be dynamic; that is, one in which the choice of a future action depends on the new information that would be available before making the choice. This is in contrast to a static decision model in which future actions are fixed at the present time based on present information.

The following sections of this paper cover the important aspects of a dynamic decision model:

1. An evaluation of dynamic and static decision models (discussion on the shortcomings of a static model and the advantages of a dynamic model),
2. Description of a Markovian decision model (this dynamic model is particularly suitable for a PMS), and
3. A successful application of the Markovian decision process (the development of a PMS for the Arizona Department of Transportation by using a Markovian decision process).

EVALUATION OF STATIC AND DYNAMIC MODELS

The major differences between the two types of models can be best illustrated by means of a simple example. Assume that the decisions of a rehabilitation action for a given pavement will be based on a single criterion, namely, the present serviceability index (PSI). The minimum acceptable PSI level is considered to be 2 for this example. Assume that the current PSI of the pavement is 3. Only three alternative actions will be considered: routine maintenance only, a 1-in. overlay, and a 3-in. overlay.

Static Decision Model

In a static decision model, future pavement performance following any of the rehabilitation actions is assumed to be known with certainty. Alternatively, only the expected performance is considered, thereby ignoring the possibilities of better- or worse-than-expected performance. Hypothetical performance curves for the three rehabilitation actions are shown in Figure 1. A major rehabilitation action will be selected for the pavement when it reaches

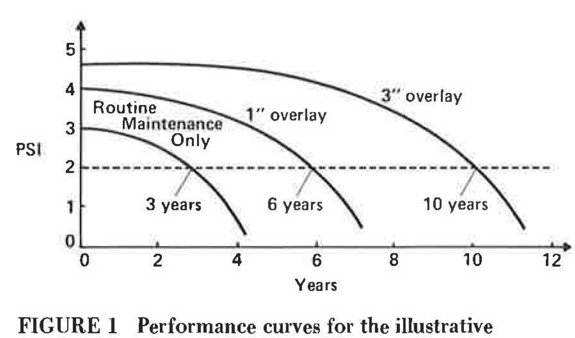

FIGURE 1 Performance curves for the illustrative example.

the threshold PSI of 2.0. Thus one rehabilitation strategy might be to apply a 1-in. overlay at year 3, a 3-in. overlay at year 9, and a 1-in. overlay at year 19 (see Figure 2). This strategy will maintain the pavement condition at or above the PSI of 2.0 during a selected analysis period of 20 years. Several alternative rehabilitation strategies can be defined. The total present worth cost of each strategy during the analysis period can be calculated, including construction cost, maintenance cost, user cost, and salvage value. All alternative strategies are then ranked based on the total present worth cost, and the one with the minimum cost is selected for implementation.

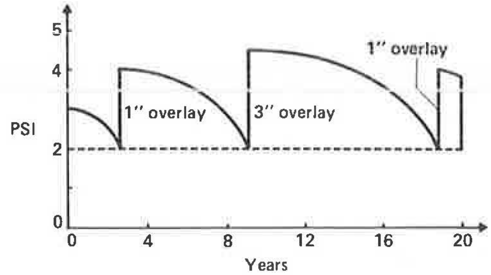

FIGURE 2 Example of a static rehabilitation strategy.

Some implications of this approach are worth noting. The choice of the action at the present time is strongly dependent on the actions selected for future time periods. Yet future actions may not be taken at the designated time periods because the pavement may perform better or worse than expected. This implies that not only the future choices of rehabilitation actions might be inappropriate, but also that the choice of an action at the present time could be ineffective.

For decisions under uncertainties, the expected cost is considered to be a rational criterion for ranking alternative courses of action (1). However, a static model generally would not result in the least expected cost strategy because of the nonlinear relationships between user cost and maintenance cost, and PSI. Thus the cost calculated on the assumption of expected PSI behavior with time would not be equal to the expected cost of that strategy. In fact, it is likely that a strategy with significantly higher expected cost than some other strategy would be selected as being the best (the most cost effective).

Dynamic Decision Model

Consider how a dynamic model would analyze this problem. In this model it is recognized that the future PSI following any of the actions is not known with certainty. However, probabilities of reaching different PSI levels as a function of time can be estimated.

Furthermore, only the decision of what needs to be done right now is to be made at the present time. Decisions of future actions will be dependent (conditional) on the future performance of the pavement. The dynamic model can be illustrated in the form of a decision tree, as shown in Figure 3.

A decision tree consists of two types of nodes--a decision node and a chance node--and several alternatives shown as branches at each of these nodes. At

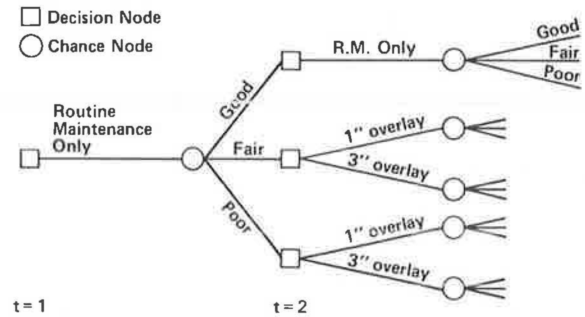

FIGURE 3 Example of a decision tree.

a decision node the branches represent feasible alternative actions. The branches at a chance node represent the possible outcomes of the action taken at the previous decision node. The probabilities of these possible outcomes are estimated.

Now follow this structure for the illustrative example. Because the present PSI of the pavement is 3, the only feasible action is routine maintenance only. This is shown as the only branch at the decision node at present time (t=1).

The PSI of the pavement at the end of one time period cannot be determined with certainty. However, knowing pavement characteristics, traffic, and environmental conditions, probabilities that the pavement will be at different PSI levels can be estimated. For simplicity, consider three discrete levels of PSI: good (greater than 3), fair (2 to 3), and poor (less than 2). These three outcomes are shown as alternative branches at the first chance node in Figure 3. Conditional on each outcome, appropriate alternative actions are selected at the beginning of the second year (t=2). For example, if the outcome is poor PSI, the two alternative actions are a 1-in. overlay and a 3-in. overlay. Following each alternative action, the probabilities of three PSI levels are again estimated at the end of the second time period. This process is continued until the end of the analysis period is reached.

The analysis of a decision tree requires the estimation of probabilities and costs of different outcomes at each chance node. The costs would include construction cost, maintenance cost, and user cost associated with a given PSI level. The analysis is conducted by "folding" the tree backwards. Assuming n to be the analysis period, expected costs are calculated at each chance node at the end of the nth time period. At the decision nodes at the beginning of the nth period, the alternative actions with the minimum expected costs are selected. Then the chance nodes at the end of the (n-1)th time period are considered. Expected costs are again calculated, assuming that the minimum expected cost actions would be selected at the following decision node. The actions with minimum expected costs are again selected at the decision node at the beginning of the (n-1)th time period. This process is continued until the first decision node is analyzed to select the action that has the minimum total expected cost.

Note that the optimum strategy determined from a decision tree fixes the action only at the first time period. At each of the following time periods, the optimal actions are conditional on the possible outcomes at the preceding chance node. Thus the optimum strategy might be identified as follows: Do only routine maintenance at t=1. If the pavement is found to be at a good PSI level at t=2, continue with routine maintenance only; if found at a fair PSI level, select a 1-in. overlay; and if found at a poor PSI level, select a 3-in. overlay.

The size of a decision tree can become extremely large for a real-life problem. This is because several distress types (instead of just PSI) may have to be considered separately in defining pavement condition, and a large number of alternative actions may have to be evaluated at each time period. The problem is further complicated when a network of pavements needs to be analyzed to determine the minimum cost actions subject to the constraints of prescribed performance standards. In these situations it would be impractical to analyze a decision tree by complete enumeration (i.e., by drawing all possible branches of the tree and evaluating each branch to determine the minimum cost actions). Fortunately, a special class of dynamic decision models, called the Markovian decision process, can incorporate several pavement condition variables and alternative actions, and also can analyze a large number of pavement segments. Details of this model are given in the next section.

MARKOVIAN DECISION PROCESS

The problem of determining the optimum pavement preservation policies for a network of pavements can be formulated as a Markovian decision process that captures the dynamic and probabilistic aspects of pavement management. The main components of a Markovian decision process are condition states, alternative pavement preservation actions, and cost and performance of these actions. A condition state is defined as a combination of the specific levels of the variables relevant to evaluating pavement performance. For example, if pavement roughness and cracking were the only relevant variables, one condition state might be defined as the combination of roughness = 50 in./mile and cracking = 5 percent. Note that the definition of a condition state retains the descriptions of individual pavement distresses; consequently, better matching of preservation actions to pavement condition is possible. This is in contrast with an alternative approach in which a combined score is calculated from the levels of individual pavement distresses. In the latter approach the specific causes of deteriorated pavement condition cannot be identified if only the combined scores are predicted for future time periods.

Alternative pavement preservation actions could vary from do-nothing to routine maintenance only to minor and major rehabilitation. The performance of these actions is specified through transition probabilities. A transition probability [$p_{ij}(a_k)$] specifies the likelihood that a road segment will move from state i to state j in unit time (e.g., 1 year) if action a_k is applied to the pavement at the present time. A Markovian process is assumed to have only a one-step memory. Thus the transition probability is assumed to depend only on the present condition state i and not on how the pavement reached that condition state. Note, however, that by including factors such as age and design life of the last rehabilitation action in the definition of a condition state, the one-step memory can be made to consider the effect of type and time of the last action. A preservation policy for the entire network is the assignment of an action to each state at each time period.

Under the assumptions of a Markovian process, the specification of condition states and transition probabilities for alternative actions permits the calculation of the probabilities that a road segment would be in different condition states at any future time period for an assumed preservation policy (2). The probability that a road segment is in a given condition state can also be interpreted as the expected proportion of all segments in that condition state. This allows the calculation of the expected proportion (q_i) of the network of road segments in the ith condition state at the nth time period for a given preservation policy. Figure 4 shows the behavior of a network under the assumption of a Markovian process. The performance of the network can be evaluated in terms of these proportions. For example, desirable and undesirable condition states can

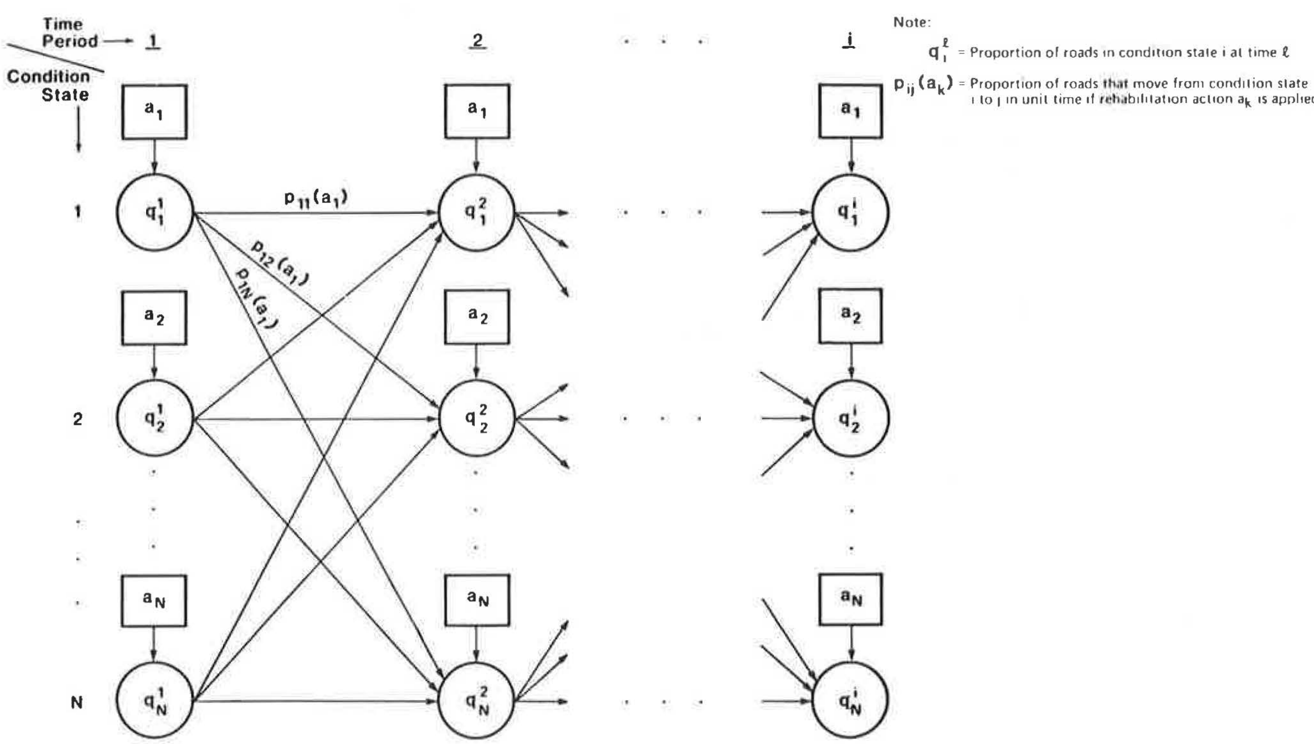

FIGURE 4 Behavior of a road network under a Markovian decision process.

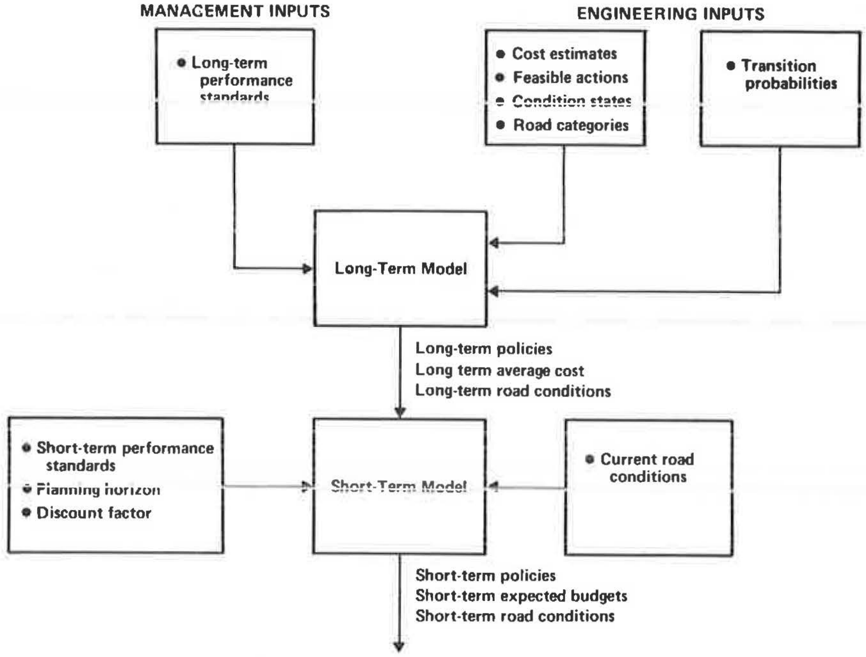

FIGURE 5 Determination of short- and long-term rehabilitation policies.

be defined, and the proportions of the network in these two categories can be plotted as a function of time. Also, whether the "health" of the network is improving or deteriorating can be assessed. A major objective of pavement management would be to find the preservation policy that would maintain desired performance standards over a long period of time at the lowest possible cost.

From a planning point of view, it is desirable that after some initial transition period (T) the network achieves a steady-state condition. A steady-state condition means that the proportion of road segments in each condition state remains constant over time. Mathematically, this implies that

$$q_i^T = q_i^{T+1} = q_i^{T+2} = \ldots, \text{ for all } i.$$

The advantages of reaching a steady-state condition is that the preservation policy will be stationary after time T (i.e., the selection of the preservation actions will be a function of condition state only and will not be affected by time). The expected budgetary requirements will also remain constant once a steady-state condition is obtained.

The user agency may desire to have control over the time (T) it would take for the network to reach the steady state. Depending on the initial conditions and the available budgets during the T time periods, short-term standards that are somewhat lower than the long-term standards may be acceptable. Optimal short-term policies (which may be different from the optimal long-term policy) can be determined to upgrade the network from its present condition to the long-term standards in time period T with minimum total expected cost while maintaining short-term standards during the first T time periods. Figure 5 shows the overall approach to determining the optimal long- and short-term preservation policies. Mathematical formulations are presented in Kulkarni et al. (3).

Advantages of Markovian Decision Process for Pavement Management

A Markovian decision process provides the capability to address two key questions of pavement management:

1. What are the minimum budget requirements to maintain desired performance standards for a network of pavements?
2. What maximum performance standards can be maintained for a fixed budget?

The first question is answered directly because for fixed performance standards, optimal policies and the corresponding minimum budget requirements are identified (see Figure 6). Note that both short- and long-term budget requirements are shown in Figure 6. The second question can be answered by varying performance standards until the minimum budget of the optimal policies matches the available budget.

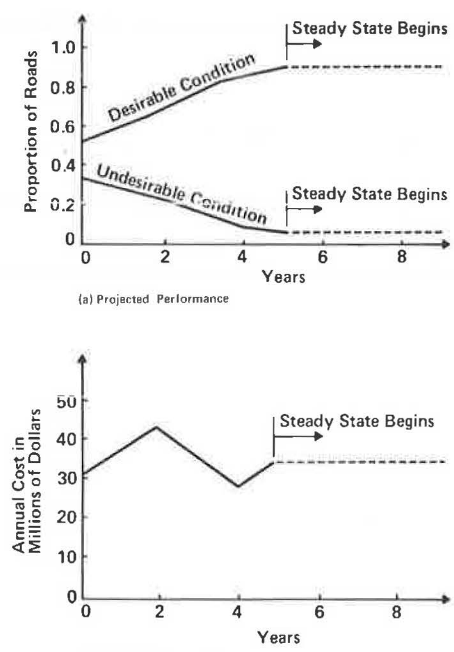

FIGURE 6 Projected performance and budgets under optimum policies.

The formulation of a PMS as a Markovian decision process offers certain distinct advantages.

1. Possibilities of pavement performance better or worse than expected are recognized and properly accounted for in the selection of preservation policies.

2. Future decisions of preservation actions for different roads are not fixed. They depend on how the pavements actually perform, and hence would be more realistic and cost effective.

3. The actions to be taken at the present time for different roads are uniquely identified. This is essential for planning purposes. In addition, the most likely actions to be taken during the next 2 to 3 years are also identified with a high degree of reliability.

4. Performance of a pavement following any given preservation action needs to be predicted only for one time period into the future. The prediction of pavement performance at succeeding time periods is conditional on how the pavement behaves and what action is taken. In contrast, a static decision model requires long-term predictions that are unconditional (i.e., independent of how the pavement may behave in the future). Such predictions are known to have poor reliability.

5. The success or failure of pavement management decisions can be evaluated by comparing the expected proportions of roads in desirable and undesirable condition states with the observed proportions of roads in those condition states. If the observed performance is significantly worse than expected, causes for this situation can be searched, identified, and corrected. Examples of such causes are poor quality control during construction, different materials, extreme environmental conditions, higher-than-expected traffic, and so forth.

6. A dynamic decision model has the potential for significant cost savings through the selection of less conservative preservation actions that still maintain the desired performance standards. Because a small proportion of a highway network can be accepted to be in poor condition at any given time, the model can consider actions for which there is some probability of pavement failure before reaching a prescribed design life. The probability of pavement failure along with the cost of repairing the deteriorated pavement are properly weighted to evaluate the options of substantial corrective actions when a pavement is in poor condition versus some moderate preventive actions before reaching poor condition.

ARIZONA'S PMS: A SUCCESSFUL APPLICATION OF THE MARKOVIAN DECISION PROCESS

The heart of the PMS in Arizona is an optimization model termed the network optimization system (NOS). It recommends pavement preservation policies that achieve long- and short-term standards for road conditions at the lowest possible cost. The NOS is based on formulating the problem as a constrained Markovian decision process that captures the dynamic and probabilistic aspects of the pavement management problem. Linear programming is used to find the optimal solution. The details of this system are provided in Kulkarni et al. (3) and Golabi et al. (4). The main steps involved in the development of the NOS were

1. Definition of condition states,
2. Selection of maintenance actions,
3. Development of transition probabilities,
4. Specification of performance standards, and
5. Development of computer software.

A brief description of each step is given in the following sections. The implementation of the system and its benefits are also summarized.

Condition States

The variables used to define the condition states were present roughness (three levels), present amount of cracking (three levels), change in amount of cracking during previous year (three levels), and index to the first crack (five levels). A total of 135 combinations of these variables are possible. However, 15 of these combinations are considered highly unlikely, which left 120 condition states.

Roughness represents the traveling public's perception of pavements in terms of comfort and the wear and tear on the vehicle caused by rough roads. It is measured by a Mays meter, which records deviations between the axle and the body of the car and adds up the number of inches of bumps per mile. Cracking is the highway engineers' rating of the structural adequacy of the pavement and its need for corrective maintenance. The road surface is compared with pictures showing different percentages of cracking.

Index to the first crack is a number that is linked to the last nonroutine maintenance action taken on the road. It is used to account for differences between the probabilities of deterioration of roads with no visible cracks, but with different last nonroutine actions. To understand the significance of the index, consider two road segments: A and B. The last nonroutine action on A has been resurfacing with 1 in. of asphalt and the last action on B has been resurfacing with 3 in. of asphalt. No cracks are visible on either road, and routine maintenance is planned for the current year. The two roads will have significantly different probabilities of developing cracks during the next year. Because the indices are different, the model assigns these roads to two different states with different probabilities of deterioration. However, once a road shows some cracks, the amount of future cracking depends only on the current cracking and on the rate of change in cracking; it is not important anymore to know the last nonroutine action taken or the time the action was taken. It is worthwhile to note that roads with the same age may behave differently because of other factors (for instance, subsurface moisture and deflection). The net effect of all these factors, including aging, is captured by the two condition variables: cracking and the rate of change in cracking.

To summarize, a state is defined by a vector (u, Δu, r, z), where u denotes the present amount of cracking, Δu the change in cracking during the previous year, r the roughness, and z the index to the first crack. The index z changes only if a nonroutine maintenance action is taken.

The statewide network was divided into nine road categories that were defined as combinations of average daily traffic and a regional environmental factor that depends on several climatic conditions; elevation and rainfall were the primary variables used to define the regional factor on a scale of 0 to 5. Because traffic density and the regional factor are independent of the preservation action, each pavement remains in one road category. This in effect made nine networks, each of which was characterized by a set of 120 condition states.

Maintenance Actions

A total of 17 alternate maintenance actions, ranging from routine maintenance to substantial corrective measures, were selected for asphalt concrete pavements. From this master list, a set of feasible actions was specified in the model for each state. The average number of feasible actions for each state was about six.

Transition Probabilities

The existing models for predicting road deterioration depend, for the most part, on empirical equations relating long-term deterioration to the structural properties of the pavement. Although these models are suitable for cases where adequate data do not exist, they were not appropriate for Arizona. Over the years Arizona had accumulated extensive data on its road conditions and the corrective actions taken on those roads. To obtain better predictions, regression equations were developed that concentrated on short-term deterioration, and Arizona's data base was used.

First a set of independent variables that are traditionally used for predicting deterioration were considered: deflection, spreadability, subgrade support, and so forth. However, the correlations obtained with these variables were rather poor. Second, it was argued that the influence of the engineering and environmental factors was captured by the observed pavement conditions. Hence the present values of the condition variables and the rate of change in these variables should reveal a strong correlation with future pavement condition, an assumption that was confirmed by the analysis of data (correlation coefficients for regression equations ranged from 0.81 to 0.95). This approach was consistent with the requirements of the optimization model, because it requires only what (condition) state the pavement would be in, and not why it would deteriorate to that state.

With this approach, the independent variables considered were present pavement condition (roughness or cracking), change in pavement condition during the previous year, maintenance actions, traffic densities, and the regional environmental factor. The dependent variables were changes in roughness and cracking in 1 year.

The (normal) continuous probability distributions of the dependent variables were discretized to give the probability of going from one level of roughness and cracking to another level in 1 year. It is reasonable to assume that roughness and cracking are probabilistically independent. Thus if the roughness associated with state i is denoted by r_i, the cracking by u_i, and the change in cracking in the previous year by Δu_i, then

$$p_{ij}(a) = P \text{ (moving from } r_i \text{ to } r_j \text{ in 1 year under action a) or}$$
$$= P \text{ (moving from } u_i \text{ and } \Delta u_i \text{ to } u_j \text{ and } \Delta u_j \text{ in 1 year under action a).}$$

As mentioned earlier, the index to the first crack for state j is the same as that of state i if a is routine maintenance, and is the index associated with a if a is nonroutine maintenance.

The data for the regression equations were derived from a randomly selected group of 270 road segments within the Arizona network. For each road segment, 2 or 3 years of data were available, leading to about 700 data points for each regression equation. To verify the accuracy of the predictions, an independent data set of 53 road segments not included in the initial development was selected at random from the Arizona Department of Transportation (ADOT) files. Verification was obtained by comparing predictions of roughness and cracking with actual measurements and observations for 5 years. The correlation coefficient between observed and predicted values was greater than 0.9 for the first year, and between 0.7 and 0.8 for the fifth year (the model needs only predictions from 1 year).

For every feasible action, a pavement in a given condition state can only go to three or four states. Thus, for feasible actions, only 3 percent of the elements in the transition probability matrix were nonzero. Because for each state 6 of the 17 actions are feasible, the number of nonzero $p_{ij}(a)$'s is about 2,600 (for each road category), or slightly more than 1 percent.

Specifying Performance Standards

To set performance standards, acceptable and unacceptable states were defined and ADOT's management specified the minimum proportion of roads required to be in acceptable states and the maximum proportion of roads permitted to be in unacceptable states. The performance standards may vary as a function of average daily traffic (see Table 1).

TABLE 1 Performance Standards for the NOS

Average Daily Traffic	Minimum Proportion of Roads with Acceptable Roughness	Maximum Proportion of Roads with Unacceptable Roughness	Minimum Proportion of Roads with Acceptable Cracking	Maximum Proportion of Roads with Unacceptable Cracking
0-2,000	0.50	0.25	0.60	0.25
2,001-10,000	0.50	0.15	0.70	0.20
>10,000	0.80	0.05	0.80	0.10

Note: Acceptable pavement condition is defined as roughness of less than 165 in./mile and cracking less than 10 percent. Unacceptable pavement condition would mean roughness of more than 256 in./mile or cracking of more than 30 percent.

Development of Computer Software

A coordinated set of computer programs was developed to accept the engineering and management inputs shown in Figure 4 and to generate matrices suitable for a linear programming (LP) software package. The output report after obtaining the optimal solutions summarizes the NOS actions and costs year by year for each mile of highway in the statewide network. The present condition of each mile and the last nonroutine action are used to determine the condition state, which is then matched to the NOS output file to determine the appropriate action for the current year. For subsequent years, the NOS predicts the most likely condition state of each mile, the corresponding action, and the estimated expected cost.

Implementation and Benefits

After extensive testing with real and hypothetical data, the NOS was fully implemented in the summer of 1980. A pavement management group comprising 11 people was formed at ADOT. The group is responsible for collecting data on road conditions, providing engineering inputs, eliciting management inputs to the system, reviewing inputs with district engineers, running the NOS, and recommending pavement preservation policies to management. The NOS is now routinely being used to prepare pavement preservation budgets and policies.

The PMS has changed the pavement management decision process in Arizona from a subjective, nonquantitative method to a modern system that integrates managerial policy decisions and engineering inputs through an optimization system. The significant benefits of the system to ADOT are as follows.

Cost Reductions

During the first year of implementation (fiscal year 1980-1981), the PMS saved $14 million of preservation funds. Because pavement condition data were available since 1974, it was possible to calculate the proportions of roads in acceptable and unacceptable conditions for past years. Those proportions have remained fairly stable. The amount budgeted by ADOT for 1980-1981 to keep the network at the same standards was $46 million. Using the PMS and following its recommended policies, ADOT was able to achieve the same standards with $32 million. The long-range standards used in the model (Table 1) were also the historical standards. The $14 million were subsequently spent on other highway-related projects. Because the NOS ties present actions and conditions to long-range performance standards, and large fluctuations in total annual expected costs are not allowed, the cost reduction in 1980-1981 was not at the expense of either poor future road conditions or costly measures in subsequent years. This is confirmed by the 1981-1982 preservation budget, which was only $28 million to keep the roads above acceptable standards.

There were two reasons for the cost reduction. First, traditionally, the roads have been allowed to deteriorate to a rather poor condition before any preservation action was taken. The roads then required substantial and costly corrective measures. The actions recommended by the PMS are mostly preventive measures; that is, it recommends less substantial measures before the road deteriorates to an extremely poor condition. Analysis indicates that less substantial but slightly more frequent measures not only keep the roads in good condition most of the time, but the measures are overall less costly; they prevent the road from reaching poor conditions that require much costlier corrective measures.

Second, in the past corrective actions were too conservative; it was common to resurface a road with 5 in. of asphalt concrete. The assumption was that the thicker the asphalt layer, the longer it would take for the road to deteriorate below acceptable standards. Although this assumption is correct, the time it takes for a road to deteriorate is not proportional to the asphalt layer. For example, the prediction model indicates that there is no significant difference between the rate of deterioration of a road resurfaced with 3 in. of asphalt concrete and a road resurfaced with 5 in. Therefore the policies recommended by the PMS are less conservative; for example, a recommendation of 3 in. of overlay is rather rare and is reserved for the worst conditions. It is important to note that the results of the prediction model are not sufficient for determining the optimal maintenance policy. Although the prediction model enhances highway engineers' understanding of the general effectiveness of actions, the final recommendation depends on considering the costs versus benefits of all actions in the context of short- and long-term standards and current road conditions. Given the size of the problem, this would only be possible through the use of a formal optimization model.

Sources of Funds

A major source of funds for highway maintenance is FHWA. These funds, called restoration, rehabilitation, resurfacing, and reconstruction (4R) funds, are based on factors such as miles of Interstate, the amount of land owned by the federal government in the state, and population. The estimated amount of 4R funds available to Arizona for preservation of the Interstate highway during the next 5 years is $167.5 million. By using the PMS, ADOT estimates that only $91.7 million is needed to maintain Interstate roads in acceptable conditions during the next 5 years. The surplus of $75.8 million will be allocated to other construction projects over the next 5 years (Table 2).

In addition to the 4R funds, the federal government provides Arizona with funds for maintaining and constructing primary and secondary roads [called primary-secondary construction funds (PSCF)], of which a minimum of 20 percent has to be spent on preservation. Traditionally, ADOT has allocated 50 percent of these funds for this purpose. By using the PMS, ADOT finds that only 20 percent of the PSCF is needed for preservation during the next 5 years. The difference of $25.6 million that would have been spent on preservation of secondary and primary roads will now be allocated to construction projects (Table 2).

Budgets

The PMS has provided a defensible procedure for preparing 1- and 5-year budgets for preservation of pavements. This has helped ADOT's management to justify the revenue requests before oversight legislative committees.

SUMMARY AND CONCLUSIONS

Pavements represent gradually deteriorating structures for which advance signs of impending failure can be observed. Most agencies collect pavement

TABLE 2 PMS Plan for Preservation Funds

	Funds ($000,000)						
Fiscal Year	Interstate Preservation Funds Needed	4R Funds Available	Surplus 4R Funds	Non-Interstate Primary/ Secondary Funds Needed	PSCF Funds Available	Surplus PSCF	Total Surplus
1982-1983	13.2	17.0	3.8	23.1	23.1	0.0	3.8
1983-1984	18.5	28.3	9.8	30.3	36.7	6.4	16.2
1984-1985	19.0	37.1	18.1	36.6	43.0	6.4	24.5
1985-1986	20.0	37.1	17.1	38.3	44.7	6.4	23.5
1986-1987	21.0	48.0	27.0	40.9	47.3	6.4	33.4
Total	91.7	167.5	75.8	169.2	194.8	25.6	101.4

Note: The data in this table give the funds needed to preserve present road and cracking conditions for the next 5 years (1982-1983 to 1986-1987), the funds available, and the resulting surplus.

condition data on a regular basis to identify such signs. However, neither the timing of occurrences of these signs nor the timing of actual failure following the signs can be predicted with certainty. Thus pavements designed and built the same way under the same traffic and environmental conditions reveal signs of distress at different times. Given this probabilistic behavior of pavements and the availability of periodic pavement condition data, a dynamic decision model, rather than a static decision model, is much more appropriate for such pavement management decisions as the selection of cost-effective preservation actions for pavements in different conditions and forecasting the future performance of a highway network.

In a dynamic decision model the choice of a future action depends on the pavement condition that would be observed before making the choice. Although future pavement condition would not be known with certainty at the present time, probabilities of different pavement conditions can be estimated based on the past performance of the pavement and factors such as traffic and environment. In contrast, in a static decision model future actions are fixed at the present time based on present information.

A special class of dynamic decision models--called a Markovian decision process--is described in this paper. This model is particularly suitable for pavement management decisions because it can incorporate multiple pavement condition variables, a large number of alternative actions, and a large-sized highway network. The model provides the capability to determine the minimum budget requirements to maintain desired performance standards for the highway network or, alternatively, to determine the maximum performance standards that can be maintained for a fixed budget. Among the significant advantages of a PMS using a Markovian decision process are reliable prediction of future performance of the network and identification of preservation actions that are generally less conservative (and less costly) than the traditional choices of actions and yet maintain the network performance at prescribed performance standards.

The development of a PMS for Arizona represents a successful application of the Markovian decision process to pavement management. Significant cost savings have resulted from the use of this system.

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Life-Cycle Costing of Paved Alaskan Highways

RAM B. KULKARNI

ABSTRACT

The development of a pavement design evaluation system (PDES), which provides a systematic, consistent, and efficient procedure to evaluate alternative initial designs for paved highways in Alaska on the basis of their total life-cycle costs, is described. The major cost components of PDES are initial cost of construction, cost of routine maintenance required to keep a pavement serviceable, possible salvage value, and user costs. PDES consists of four subsystems: pavement performance subsystem, cost subsystem, life-cycle cost procedure, and optimization subsystem. Mechanistic procedures tailored to Alaskan conditions and calibrated with empirical data and engineering judgments have been used to predict future physical characteristics of alternative pavement designs. The performance variables for which prediction models are developed are roughness caused by cumulative application of traffic loading, roughness caused by thaw settlement in permafrost regions, fatigue cracking, and major transverse cracking. Uncertainties associated with the prediction of future pavement performance are explicitly considered in PDES to calculate the total expected costs during a specified analysis period and to determine the minimum cost alternative that satisfies desired reliability constraints. As a tool for the designer and decision maker, PDES provides a means of documenting and justifying specific design selections for site-specific projects contemplated for construction in Alaska.

Recent developments in the field of pavement management indicate that the selection of an initial pavement design should consider not only the initial construction cost, but also costs incurred during a life-cycle period. Life-cycle costs should include user costs caused by increased surface roughness, routine maintenance costs for maintaining pavements in minimum acceptable condition, and inflation and interest factors.

Currently, the Alaskan road design process considers only the initial cost of the type of structure as determined by the provisions of the design manual. Alternative design choices are few and are usually a direct response to budget changes during the preconstruction period. The eventual effects of increasing or decreasing layer thickness cannot be rationalized because the trade-offs between increased initial costs and decreased life-cycle costs (user and maintenance costs) are not considered.

The primary objective of the investigation described in this paper was to develop a systematic procedure for the determination of life-cycle cost comparisons for alternative pavement designs contemplated for use in various climatic zones in Alaska. For purposes of this investigation, life cycle refers to serviceable life of original construction with provision for such maintenance activities as crack filling, seal coat, leveling, and thin overlays; however, thick overlays are not considered because they generally are not used in Alaska. Cost considerations include initial cost of construction, cost of routine maintenance required to keep the pavement serviceable, salvage value, and user costs.

To meet the objectives of the project, a pavement design evaluation system (PDES) was developed that provides a systematic, consistent, and efficient procedure to evaluate alternate designs and to select the optimum alternative for paved highways in Alaska.

The paper is organized into six major sections:

1. Research approach: An overview of the approach used in the investigation.
2. Pavement performance subsystems: The development of pavement prediction models used to estimate the future physical characteristics of alternative pavement designs.
3. Pavement cost subsystem: Cost models used to associate pavement costs with alternative design considerations.
4. Life-cycle cost calculations: Procedures and assumptions that are necessary to combine performance expectations with costs for alternative design considerations.
5. Optimization subsystem: Procedures used to determine the expected costs of feasible alternative designs.
6. Summary and conclusions: A review of results from the investigation with suggestions for implementation and periodic updating.

RESEARCH APPROACH

Figure 1 shows the basis for structuring the PDES. The major subsystems are given in this figure and show the general order and continuity of the proposed system. A brief description of each subsystem is provided in the following sections, which describe in detail the development of each subsystem.

Pavement Performance Subsystem

Two sets of pavement performance models are considered:

1. Statistical-mechanistic pavement performance prediction models used for the analysis of the normal structural pavement layers for surface environments (i.e., without considering the impact of permafrost conditions), and
2. Models that estimate the rate of development of pavement roughness for subsurface environment (i.e., roughness caused by thaw settlement for roads built over permafrost foundations).

The performance models are used to estimate the expected life cycle of the pavement (i.e., the time to reach a specified terminal condition) for two selected performance variables: roughness and fatigue cracking. In addition, an estimate of the dispersion around the expected life cycle is made

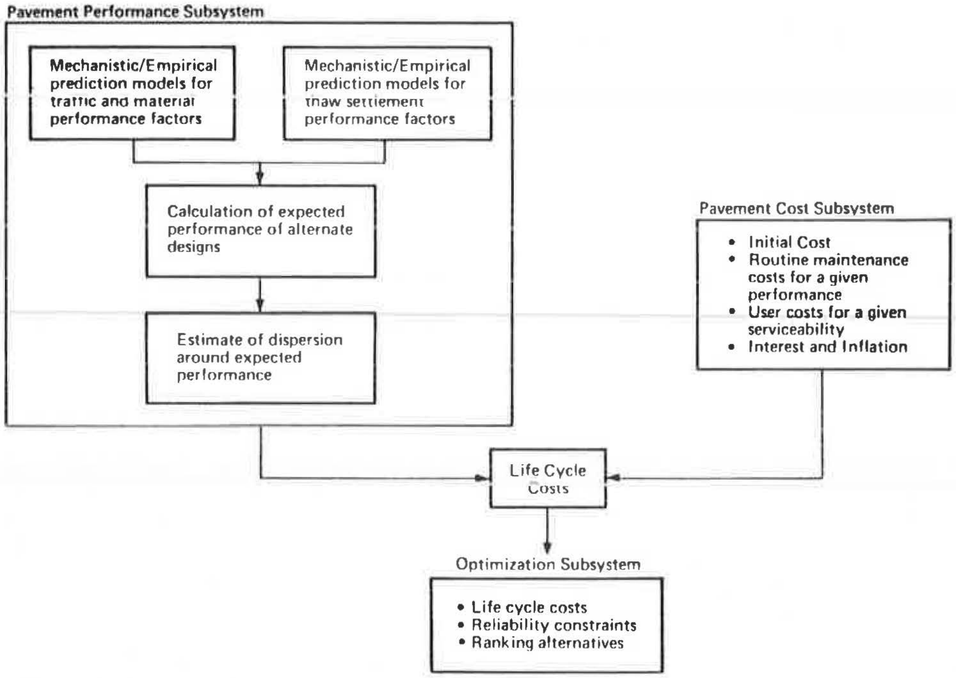

FIGURE 1 Pavement design evaluation system.

based on the analysis of available data and engineering judgment.

Cost Subsystem

The following cost components are included in the calculation of the total cost of a design alternative:

1. Initial construction and material costs (including possible salvage value);
2. Routine maintenance costs during a specified analysis period;
3. User costs, including vehicle operating costs and time delay costs; and
4. Inflation and interest factors.

Life-Cycle Cost Procedures

Computational procedures are developed to combine the performance models with the cost models in order to reflect maintenance policies and user cost considerations.

Optimization Subsystem

The total expected cost of each design alternative is calculated, and the alternative with the minimum total expected cost that satisfies specified reliability constraints is determined. A ranking of all design alternatives on the basis of their total expected costs is also produced.

PAVEMENT PERFORMANCE SUBSYSTEM

For purposes of this investigation, performance is characterized in terms of the following distresses: fatigue cracking (caused by traffic), major transverse cracks (caused by material properties and surface environment), and roughness (caused by traffic). In addition, for projects to be designed in permafrost locations, roughness caused by differential thaw settlement was also considered. A prediction model was developed to estimate the progression of each distress type with time as a function of the initial design, expected traffic, and surface and subsurface environmental conditions. Expected values of each distress, as well as the dispersion around the expected values, were characterized in the development of the performance prediction models.

Because of constraints on space, only the prediction model for fatigue cracking is described in this paper. Details regarding all of the prediction models are provided in Kulkarni et al. (1).

Prediction Model for Fatigue Cracking

Fatigue cracking is a result of cumulative damage produced by repetitive loadings applied to a pavement. Damage is believed to be associated with the deflection-induced strains that occur in the underside of the asphalt concrete layers. Fatigue cracks are usually referred to as alligator cracks because of the resemblance of the crack patterns to that of the skin of an alligator.

Fatigue cracking is influenced by a wide variety of factors, including pavement thickness, layer thicknesses, material properties, environment (temperature, rainfall, frost penetration), and traffic loadings (weight and frequency). Models to predict fatigue cracking should incorporate as many of the enumerated characteristics as appropriate.

Three general models were considered for use in predicting fatigue cracking:

1. A mechanistic-empirical model (PDMAP) developed for NCHRP (2);
2. An empirical model (OPAC) reported by Meyer et al. (3); and
3. Empirical relationships reported by McHattie et al. (4).

The PDMAP program was considered a prime candidate during the planning phases of the project; however, the model was eventually eliminated because of the lack of sufficient information relative to material properties and detailed performance information. A less sophisticated but adequate model was developed for this study based on damage models reported by McHattie et al. (4). It is pertinent to note that the PDMAP procedures can be incorporated in the PDES program once more information is available for damage models and material properties.

To take advantage of the information reported in McHattie et al. (4), two assumptions were required, as follows.

1. The majority of fatigue cracking occurs during the critical thaw weakening season (period). Observations of fatigue cracking at the AASHO Road Test (5) indicated that the majority of cracking occurred during the spring thaw period. Similar findings are reported for Alaskan highways (4).
2. Traffic during periods of thaw weakening is proportional to the total annual traffic (in terms of equivalent 18-kip single-axle loads) over a site-specific project. Consequently, the total traffic for each given project can be used as an independent variable in the regression analysis of fatigue cracking data on different projects. Because the distribution of the annual traffic by periods of the year will not be necessary, this will simplify the estimation of traffic data.

A total of 120 special study sections were available for developing a fatigue cracking prediction model (4). Only sections with fatigue cracking were included in the analysis because the timing of when fatigue cracks would develop could not be estimated for uncracked sections. Several alternative regression equations were tried with different independent variables and their combinations. The final equation selected for PDES was as follows:

$$\log (FC) = -19.05 + 5.67 \log (BB) + 2.09 \log (EAL) \quad (1)$$

where

FC = percentage of fatigue cracking in the section for both wheelpaths (ranges from 0 to 100 percent),
BB = surface deflection in 10^{-3}-in. units under 9-kip dual wheel load as measured with the Benkelman beam and represented by the mean deflection plus two standard deviations, and
EAL = annual equivalent 18-kip single-axle loads using AASHTO equivalency factors.

The square of multiple correlation coefficient for Equation 1 was 0.54.

Estimation of Inputs to Fatigue Cracking Prediction Model

In order for the designer to use the fatigue cracking prediction model, it will be necessary to estimate traffic and deflection for each design alternative. Traffic can be estimated based on available traffic count data for adjacent projects and the expected use of the new roadway. The deflection for each alternative design section is estimated by means of an elastic-layered structural analysis. The specific program incorporated into PDES is the N-LAYER program described by Schiffman (6).

The required inputs for the N-LAYER program to predict surface deflection under a standard 18-kip axle load are (a) elastic modulus of each layer of pavement, including foundation materials, during the critical period when most fatigue cracking occurs; and (b) thickness of each pavement layer. The data for the development of modular information were available from two sources: backup reports in McHattie et al. (4), as provided by Alaska Department of Transportation and Public Facilities (ADOTPF) staff; and information from Dynatest Consulting, Inc. (7).

Studies by ADOTPF research personnel have indicated that the occurrence of fatigue cracking is related to the percentage of fines in the aggregate. Information from Dynatest Consulting, Inc., provided data relative to the in situ moduli of asphalt concrete, aggregate base, and the supporting materials to a depth of 48 in. below the base. Thus an effort was made to predict the moduli of granular layers based on the percentage passing the No. 200 sieve. The moduli values used in developing the prediction model were the spring values reported by Dynatest Consulting, Inc.

The general form of the model was

$$MR = f(-200 \text{ in each layer}) \quad (2)$$

where MR is the resilient modulus, equivalent to modulus of elasticity; and -200 is the percentage of fines passing the No. 200 sieve.

Specific regression equations were developed for the base course (usually the first 6 in. below the surface layer) and the granular layers below the base course. The development of these equations is described in Kulkarni et al. (1).

The modulus of the asphalt concrete layer has been set at 1.1×10^6 psi, which is representative of values used by Dynatest Consulting, Inc., during field testing with the falling weight deflectometer. This value is included as a default value in the N-LAYER program.

PAVEMENT COST SUBSYSTEM

The principal elements in the cost subsystem include

1. Initial and stage construction (including possible salvage value),
2. Routine maintenance,
3. Excess road user costs, and
4. Considerations of interest and inflation.

Each of these elements is responsive to a combination of designer inputs and prediction model outputs.

Estimation of Initial Costs

The cost subsystem can accommodate the initial and stage construction costs for two general cases: a roadway section that traverses an area where no permafrost is present, and a section that traverses an area where permafrost is present. For cost comparisons, mass grading is excluded. It is assumed that mass grading will be essentially the same for all alternatives.

The roadway section that traverses a nonpermafrost subgrade would consist of a non- or low-frost-susceptible borrow on which the pavement section is constructed. For those sections that traverse a subgrade with permafrost, additional embankment would be constructed before the borrow layer. The installation of insulation and the construction of thermal berms could also be accommodated.

Designer inputs would include those items necessary to establish the geometry of the section such as paved width, roadway width, fill slopes, thickness of pavement layers (asphalt concrete, aggregate base, aggregate subbase), thickness of borrow, thickness of insulation, thickness of unclassified fill, and the dimensions of the thermal berms. Stage construction is assumed to be the construction of bituminous surface treatment (BST) on aggregate base before construction of the asphalt concrete surface. For the stage construction alternate, the width of the BST would be required.

The designer would also be required to input unit costs for the various materials of construction in the units included in the following table:

Item	Unit
Asphalt concrete	\$/ton
Aggregate base	\$/ton
BST	\$/yd^2
Aggregate subbase	\$/ton
Borrow	\$/ton
Insulation	\$/yd^2/2-in. thickness
Unclassified fill	\$/yd^3

If some salvage value is associated with certain materials for a specified design alternative at the end of a selected analysis period, the unit costs for initial construction should be reduced by the present worth of the salvage value.

Estimation of Routine Maintenance Costs

Maintenance cost records from 1977 through 1980 were provided by the Information Systems Division of ADOTPF in Juneau. These data consisted of the annual cost per mile for those activities associated with maintenance of the pavement surface. The information was provided for selected major paved routes in the state's highway system. The activities currently reported for surface maintenance include pothole repair (Activity 002), skin patching and thin overlays (Activity 004), crack sealing (Activity 011), and seal coats (Activity 012). The paved highway performance evaluation data for 1978, 1979, and 1980 were used to establish relationships between performance and routine maintenance costs.

Information on the suspected locations of permafrost was obtained by examining the raw data printouts from the Mays meter. There is a characteristic signature produced by the Mays meter graph that has been correlated with areas where permafrost is known to exist. The Mays meter records for all of the major routes were examined and the limits of suspected permafrost were identified. In almost all cases section lengths were less than 0.2 mile.

Although clear relationships between distress (as reported in the road inventory) and maintenance costs could not be identified, examination of the data did reveal general trends. By using these trends and engineering judgments, relationships were developed between fatigue cracking, traffic roughness, and thaw settlement roughness observed for a given year and routine maintenance costs for that year based on 1980 dollars. Only the relationship between fatigue cracking and routine maintenance costs is summarized in this paper. Details regarding all the relationships can be found in Kulkarni et al. (1).

For fatigue crack sealing (Figure 2), the first portion of the curve represents repair by crack filling, which might occur during the early stages of fatigue crack development. It was estimated that when more than 30 percent of the road section length has fatigue cracking, the choice would be the construction of a seal coat, which is represented by the linear portion of the curve.

Estimation of Excess Road User Costs

The roughness of a pavement can contribute to road user cost by increasing running time and operating cost. In considering excess road user cost (i.e., those road user cost differentials that are caused by pavement roughness only), it was decided to limit

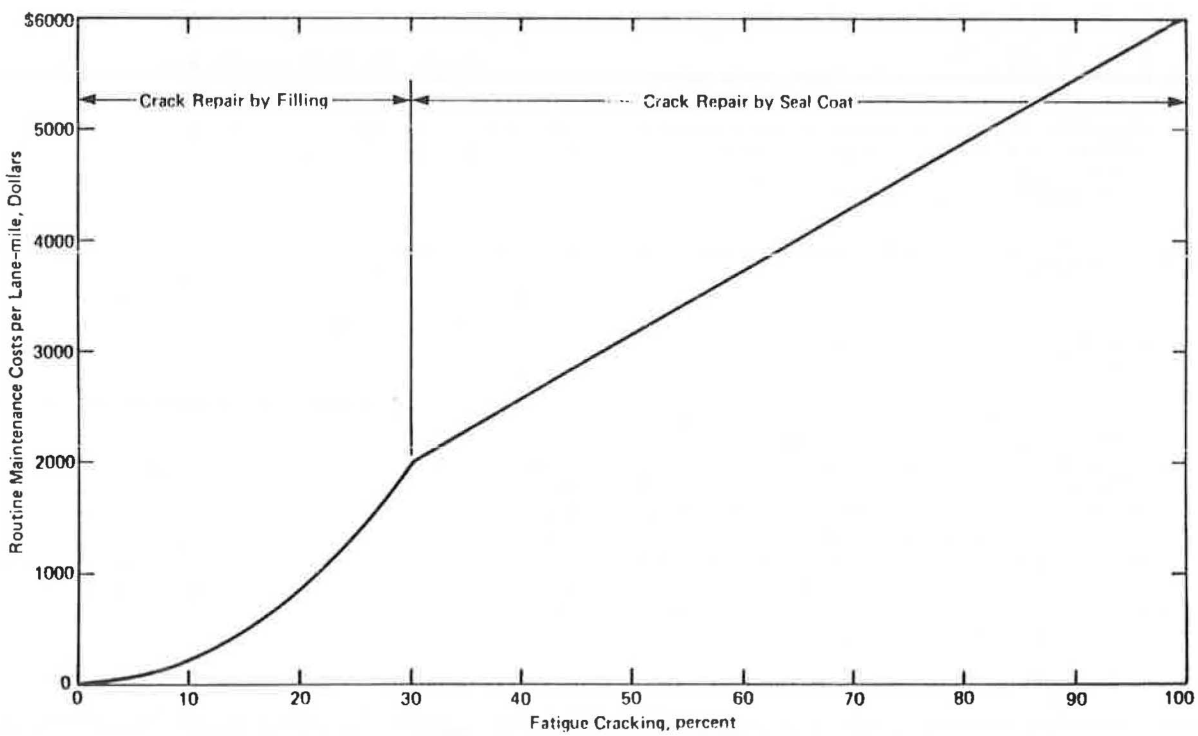

FIGURE 2 Relationship between routine maintenance cost and fatigue cracking.

the analysis to the estimation of excess cost experienced by commercial vehicles. Not included are excess road user costs associated with the operation of private vehicles. The differential operating costs are probably quite small, and noncommercial driver time may not have a significant dollar value.

A relationship was developed from information contained in the literature and from interviews with two trucking companies, which allowed the estimation of excess user costs as discussed in the following sections.

Differential Running Time

Zaniewski et al. (8) reported on a relationship that was developed between riding comfort and speed and includes such factors as volume/capacity ratio and speed limit. With riding comfort converted to present serviceability index (PSI) and speed to miles per hour, this relation becomes:

$$S = 2.404\,(PSI)^{0.0928}\,(v/c)^{-0.0275}\,(SL)^{0.704} \tag{3}$$

where

S = speed (mph),
v/c = volume/capacity ratio, and
SL = speed limit (mph).

The Mays meter used for roughness measurements on Alaskan highways has not been correlated to PSI as of this date. A method of correlating PSI with Mays meter readings is contained in Walker and Hudson (9). McHattie et al. (4) indicated that the average PSI for paved Alaskan highways was 2.2. The average Mays meter reading for the 1,200 miles of paved highway in the sample included in this study was 91 in 1980. Using this information, the following relationship was developed:

$$PSI = 5\exp[-(\ln M/4.7)^5] \tag{4}$$

where exp is the base of the natural logarithms, and M is the Mays meter readings (in./mile).

Combining Equations 3 and 4 results in the following relationship between speed and Mays meter reading:

$$S = 50\,(v/c)^{-0.0275}\exp[-(\ln M/4.7)^5] \times 0.0928 \tag{5}$$

This equation uses a speed limit of 60 mph. Although this is not the speed limit in the state, the use of 60 mph provides calculated average speeds that correspond more closely to the relationship between average speed and roughness obtained from interviews with trucking companies. For an average driver cost of \$33/hr, the average cost per Mays meter inch per mile over a wide range of roughness is \$0.00055.

Differential Operating Costs

The estimated average cost per mile for operating a four-axle tractor and multiaxle semitrailer on Alaskan highways is \$0.92. Included in this cost are fuel and oil, tires, depreciation, and maintenance and repair. In response to questions to trucking companies regarding an increase in costs when operating on extremely rough roads, it is estimated that the operating cost increase is \$0.00125 per Mays meter inch per mile.

Total Excess Road User Costs

Combining driver cost and operating cost, the total excess road user cost for large long-haul vehicles is estimated to be \$0.0018 per Mays meter inch per mile per truck. With the lowest Mays meter reading observed of about 30 in./mile, the excess road user cost then becomes \$0.0018 (M - 30).

A relationship was developed between equivalent axle loads (EALs) and excess road user cost. Using EAL constants developed for California and taking into consideration the greater axle load limit in Alaska, it is estimated that the heavy truck-trailer combination for which the excess road user costs were developed has an EAL equivalent of 3.25. The excess road user cost caused by roughness can be expressed as follows:

$$\begin{aligned}\text{Excess road user cost} &= \$0.0018/3.25 \\ &= \$0.00055\,(M - 30)\text{ per EAL per mile}\end{aligned} \tag{6}$$

Estimation of Interest and Inflation Factors

Epps and Wootan (10) recommend that the interest rate to be used in economic studies of this type should represent the real cost of capital. That is, it should be the actual rate of return on assets after inflation. They report that since 1966 the inflation-free interest rate has ranged from 3.7 to 4.4 percent. This represents approximately the difference between interest and inflation. It is interesting to note that although interest and inflation rates have varied considerably since 1966, the differential has remained nearly constant.

Because maintenance is labor intensive, the largest contributor to inflation of maintenance costs would be salary increases. It is reported that this has been at a rate of 8.5 percent during the past few years. The Planning and Programming Division of ADOTPF recommends an interest rate of 10.5 percent for studies of this type. Although this rate may be artificially low, it is used to provide uniformity throughout the state. An interest rate around 15 percent may be more realistic. Because the rates of inflation for construction costs, maintenance costs, and user costs may be different, PDES allows the user to input different rates of inflation for these cost components. A common interest rate is then used to convert the inflated costs incurred at different times into their present worth.

LIFE-CYCLE COST PROCEDURES

Figure 3 shows the logic used in calculating life-cycle costs. Input parameters shown in Figure 3 are used to develop prediction models for the relevant pavement performance variables. Various cost components are then estimated as a function of the performance variables.

The procedures used to incorporate uncertainties in pavement performance and to calculate different life-cycle cost components are described in the following sections.

Treatment of Uncertainties

Because of the uncertainties in predicting roughness and fatigue cracking, exact routine maintenance costs or user costs cannot be estimated for any one year. However, the probability that roughness or fatigue cracking would be equal to a specific value can be estimated. For given values of the two performance variables, costs could then be estimated.

The continuous probability distributions of roughness and fatigue cracking at any given time were discretized into 10 intervals, each with a probability of 0.10, and the median value for that interval was assumed to represent that interval. For a variable X, the 10 intervals and their representa-

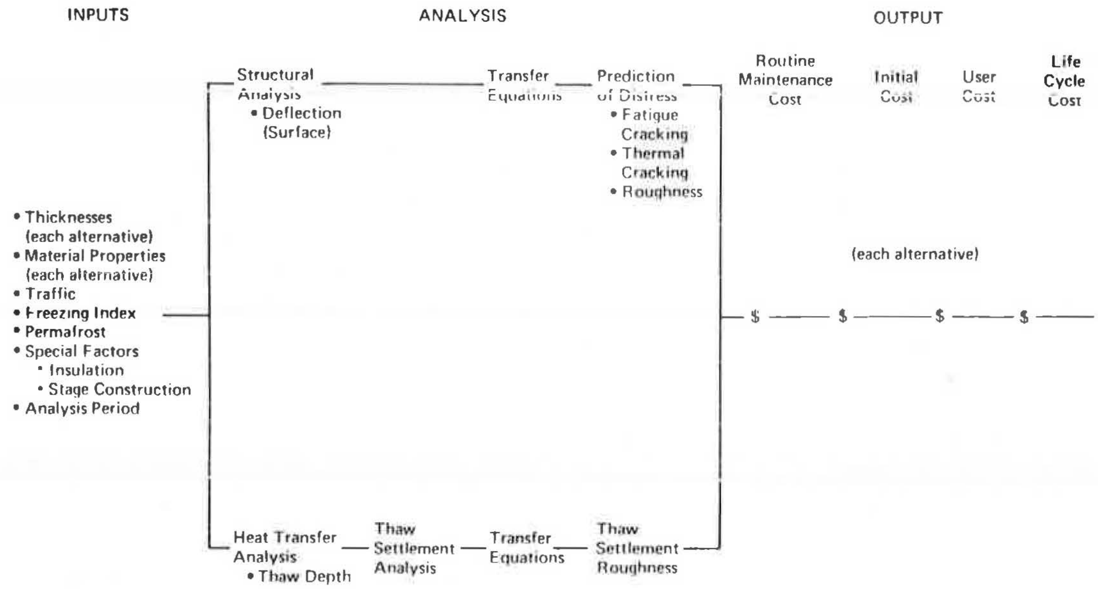

FIGURE 3 Life-cycle cost logic.

tive values were as follows (X_p denotes the value of X such that the probability of being less than or equal to X_p is p):

Interval	Representative Value
$X_{0.9} - X_{1.0}$	$X_{0.95}$
$X_{0.8} - X_{0.9}$	$X_{0.85}$
$X_{0.7} - X_{0.8}$	$X_{0.75}$
$X_{0.6} - X_{0.7}$	$X_{0.65}$
$X_{0.5} - X_{0.6}$	$X_{0.55}$
$X_{0.4} - X_{0.5}$	$X_{0.45}$
$X_{0.3} - X_{0.4}$	$X_{0.35}$
$X_{0.2} - X_{0.3}$	$X_{0.25}$
$X_{0.1} - X_{0.2}$	$X_{0.15}$
$X_0 - X_{0.1}$	$X_{0.05}$

If X is normally distributed with mean μ and standard deviation σ (or coefficient of variation $c = \sigma/\mu$), then X_p can be calculated from $X_p = \mu + k_p\sigma$, where k_p is a value from normal probability tables that corresponds to the cumulative probability of p.

It was assumed that if a pavement performs worse or better than the average at one time, it would continue to perform the same way at any other time. This is a reasonable assumption, because for a given project traffic and environmental conditions are fixed, and future pavement performance would depend on factors such as initial design and quality of construction that are determined at the time of construction. With this assumption, the performance values (X_p) at different time periods were connected to obtain a performance curve such that the probability of being less than or equal to the value on this curve at any given time would be p. Corresponding to 10 values of p, 10 different performance curves were thus defined for fatigue cracking and total roughness. This is shown schematically in Figure 4.

For each performance curve, maintenance cost and user cost were calculated by using the procedures described in the following sections. The expected costs at time t were then calculated by averaging the 10 values of the cost at that time. A standard deviation of the cost was also calculated by considering the deviations from the expected cost.

Initial Costs

The initial costs of each design alternative are calculated from the specification of the cross section of the design and properties of different layers (thickness, density, material, insulation), and unit construction costs. Volumes, weights, or areas of different quantities are calculated, multiplied by the appropriate unit costs, and summed to obtain cost per lane-mile.

For stage construction, the construction cost of the first stage is combined with the present worth cost of the second stage to obtain the total initial cost.

If some salvage value is appropriate to consider for a particular design alternative at the end of the analysis period, the unit costs should be reduced by the amount of the present worth of the salvage value of different materials (asphalt concrete, aggregate, and so forth).

Maintenance Costs for Fatigue Cracking

The maintenance policy used to estimate costs for fatigue cracking is shown in Figure 5. The basic fatigue cracking model estimates, with some uncertainty (not shown), that the amount of fatigue cracking will first reach 10 percent at year t_1. Maintenance will then be initiated that will correct the condition, bringing fatigue cracking to zero. Fatigue cracking will continue to develop according to the initial prediction curve. When the 10 percent cracking level is exceeded again at year t_2, maintenance will again be initiated to correct the condition. The maintenance cost is estimated as a function of the percentage of fatigue cracking in the year of maintenance. The process is repeated to time T for which the comparisons of alternate designs are to be made. Time T should be equal to or greater than the time required to develop 10 percent cracking in the most effective design. Alternatively, the user may specify an analysis period that is greater than the maximum design life of the strongest section. It is recommended that the user use an analysis period of 15 to 20 years. The program is currently limited to a maximum 25-year analysis period.

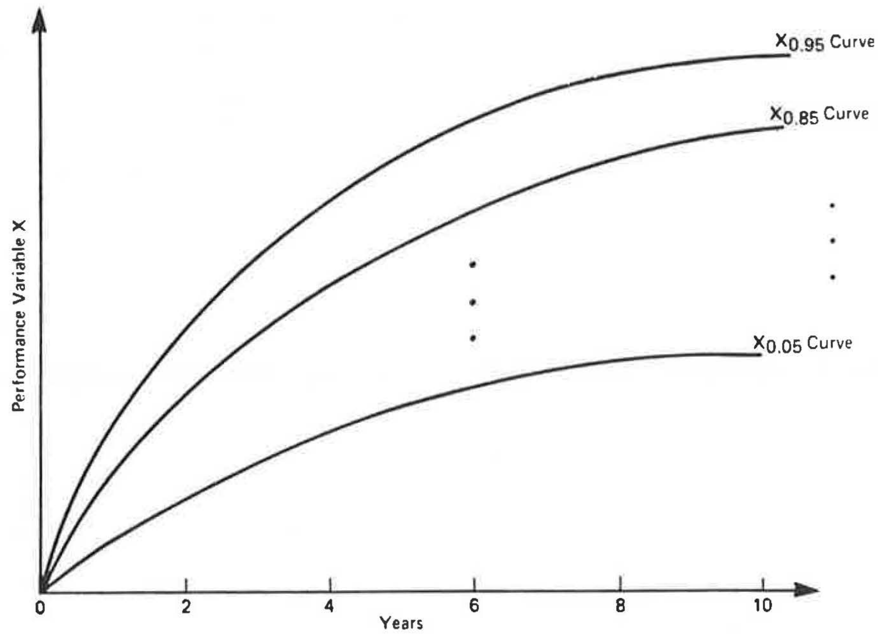

FIGURE 4 Treatment of uncertainties in performance prediction.

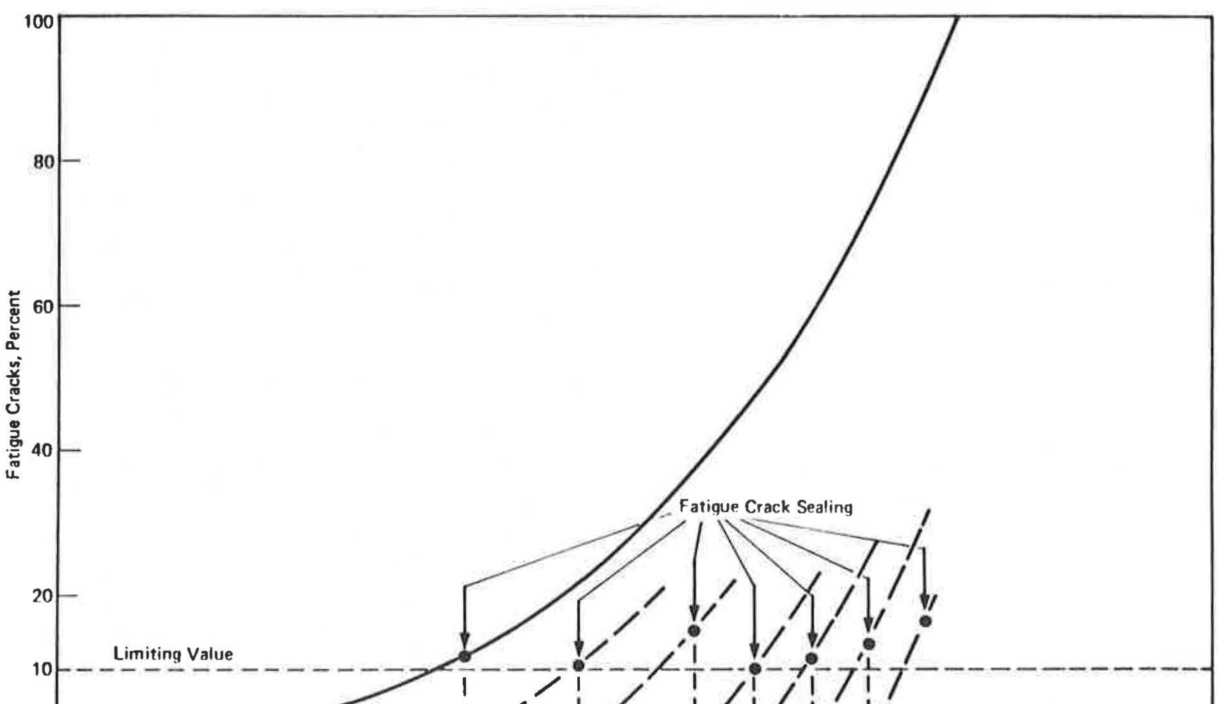

FIGURE 5 Fatigue crack sealing policy.

The expected maintenance cost for fatigue cracking in ith year [EMCF(i)] is given by

$$EMCF(i) = \Sigma\, MCF(i,FC_p)\, P(FC_p) \tag{7}$$

where $MCF(i,FC_p)$ is the maintenance cost for fatigue cracking in the ith year if the performance FC_p is followed, and $P(FC_p)$ is the probability of the performance curve FC_p.

Because 10 equiprobable performance curves are generated in the program, Equation 7 can be simplified to

$$EMCF(i) = (1/10)\, \Sigma\, MCF(i,FC_p) \tag{8}$$

The present worth of the total expected maintenance cost for fatigue cracking (TMCF) during an analysis period of T years is calculated from

$$TMCF = \sum_{i=1}^{T} \alpha_i\, EMCF(i) \qquad (9)$$

where

α_i = present worth factor = $[(1 + I_m)/(1 + I_d)]^i$,
I_m = inflation rate for maintenance activities, and
I_d = interest rate for discounting.

Maintenance Costs for Total Roughness

The process here is similar to the one described for fatigue cracking. The limiting value of roughness is assumed to be 160, and maintenance is assumed to reduce roughness to 130. Maintenance cost is a function of the percentage reduction in the roughness at the year in which maintenance takes place.

For stage construction, the second stage is assumed to reduce roughness to a mean value of 35 and a coefficient of variation equal to that assumed for roughness at any other time.

Equations for calculating the expected maintenance cost for roughness in ith year [EMCR(i)] and the present worth of the total expected maintenance cost for roughness (TMCR) during T years are similar to Equations 8 and 9, respectively. Thus,

$$EMCR(i) = (1/10)\ \Sigma\ MCR(i,R_p) \qquad (10)$$

and

$$TMCR = \sum_{i=1}^{T} \alpha_i\, EMCR(i) \qquad (11)$$

User Costs

Expected user costs for year i [EUC(i)] are calculated as a function of unit user cost in dollars per inch of roughness per EAL, total estimated roughness in a given year, and the number of EALs for that year. Thus

$$EUC(i) = (1/10)\ \Sigma\ UC(i,R_p) \times EAL(i) \qquad (12)$$

where $UC(i,R_p)$ is the user cost ($/EAL) in the ith year if the total roughness curve R_p is followed, and EAL(i) is the number of EALs during the ith year.

The present worth of the total expected user costs (TUC) for T years is obtained from

$$TUC = \sum_{i=1}^{T} \beta_i\, EUC(i) \qquad (13)$$

where

$$\beta_i = [(1 + I_u)/(1 + I_d)]^i \qquad (14)$$

and I_u is the inflation rate for user costs.

Total Cost

The present worth of the total expected cost (TEC) during an analysis period of T years is the sum of individual cost components during T years. Thus

$$TEC = I_o + TMCF + TMCR + TUC \qquad (15)$$

where I_o is the initial construction cost.

OPTIMIZATION SUBSYSTEM

The primary objective of PDES is to rank design alternatives on the basis of their minimum total expected costs. Because of the uncertainties in the prediction of pavement performance, consideration should be given to achieving some minimum reliability of satisfactory performance in addition to minimizing total expected cost. Reliability is defined here as the probability that a pavement would not reach a limiting condition within a specified time period. Mathematically, the reliability constraints can be stated as follows:

$$P\,[X > X^* \text{ in time } t^*] \leq \alpha \qquad (16)$$

This constraint states that the probability that the performance variable X exceeds a limiting value X* in time t* should be less than or equal to α. The reliability level associated with this specification will be $1 - \alpha$. The values of X*, t*, and α are provided by the user. This constraint is used both for fatigue cracking and total roughness in PDES. If a design alternative does not satisfy the reliability constraint for both fatigue cracking and total roughness, that alternative is considered infeasible and is not included further in the cost calculations. Only feasible design alternatives are ranked on the basis of their total expected cost.

SUMMARY AND CONCLUSIONS

The development of a PDES to rank alternative pavement designs in Alaska on the basis of life-cycle costs is described. Two main components of this system are pavement performance models and cost models. The performance variables for which prediction models were developed are roughness caused by cumulative application of traffic loading, roughness caused by thaw settlement in permafrost regions, fatigue cracking, and major transverse cracking. The major cost components of PDES are initial cost of construction, cost of routine maintenance, possible salvage value, and user costs.

Recommendations for future improvements in PDES include

1. Systematic and continuing collection of pavement performance data and adjustment of performance prediction models based on these data,
2. Accumulation of materials information necessary for mechanistic analysis of multilayered pavement systems and incorporation of more comprehensive mechanistic prediction models,
3. Development of improved modular values for the granular materials based on both field (falling weight deflectometer) and laboratory studies, and
4. Special studies (with somewhat limited scope) to obtain data on routine maintenance costs and excess user costs.

Although PDES is a stand-alone system for estimating life-cycle costs of alternative initial designs and selecting the minimum cost design, it can be expanded to fit into a broader pavement management system (PMS). This would involve the evaluation of combinations of initial designs and subsequent rehabilitation strategies such as overlays or possibly reconstruction. Most of the present structure of PDES, including the cost and performance prediction models, can be used in the development of a PMS suitable for Alaskan conditions.

In summary, PDES is a comprehensive procedure for ranking alternate pavement designs based on performance and cost expectations, while recognizing the uncertainty associated with each consideration. As a tool for the designer and decision maker, PDES will provide a means of documenting and justifying specific design selections for site-specific projects contemplated for construction in Alaska.

ACKNOWLEDGMENTS

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Testing the Delft University Pavement Management System

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ABSTRACT

Testing of the Delft University pavement management system on the secondary roadway network of the province of Zuid-Holland in the Netherlands is described. The application of network-level monitoring techniques, riding comfort measurements, skid resistance measurements, and visual condition surveys is discussed. The detailed visual condition surveys have proved to be especially useful. These surveys can be used to predict pavement performance, plan maintenance, and estimate the required maintenance budget levels, as well as determine the present status of the network. Techniques are presented for determining maintenance urgencies and leveling required budgets for a 3-year period. On the project level, the application of falling weight deflection measurements is described. These measurements are used to establish the structural condition of a roadway section. Overlays were designed for a section with a poor structural condition. The average of all individual overlays to be applied appeared to be equivalent to that estimated from the visual condition surveys.

The overall objective of a highway department is to keep its highway network in such a state that both safe traveling of all vehicles is guaranteed and sound structural pavement condition can be maintained without excessive costs. Criteria such as maximum levels of distress or levels of minimum serviceability determine the budget required to achieve this objective. Usually the budget will exceed the total available funds; therefore an approach is required in which an optimum balance between benefits and costs can be found. This approach or management system should incorporate the following components:

1. Procedures to determine visual condition, riding comfort, and skid resistance;
2. Criteria to identify highway sections with poor visual condition, riding comfort, or skid number;
3. Procedures to determine residual lives;
4. Procedures to determine the structural condition in a nondestructive way;
5. Criteria to determine when maintenance should be applied and procedures to determine which maintenance or rehabilitation strategy should be applied; and
6. A methodology to evaluate alternative maintenance options and to select the optimum strategy.

The method developed at the Delft University of Technology incorporates all these components (1). The main objectives of this method or system are to (a) evaluate the pavement condition, (b) estimate the maintenance and rehabilitation needs, (c) determine the budget level for each year in the programming period, and (d) determine budget allocations. These components should be structured in such a way that the total system can be implemented by user agencies with minimal difficulty.

Therefore, before proceeding with any implementation, it is recommended that the workability of the system be tested. In the test program errors and discrepancies can be eliminated and, if needed, adjustments to models and criteria can be applied to improve the management system and make it viable.

In this paper the results of the test program of the Delft University pavement management system (DUPMS) are described. Tests have been executed on (for Dutch circumstances) a relatively large secondary roadway network. In consultation with the Highway Administration of the province of Zuid-Holland, the secondary roadway network of that province was chosen as the testing area (see Figure 1). The network is 380 km long. The subsoil of the province of Zuid-Holland consists mainly of clay or a clay peat combination, except in the coastal region where a sand subsoil is found. Pavement construction varies from a 100-mm asphalt layer on a 350-mm blast furnace slag base layer to a 200-mm asphalt layer on a sand base. The majority of the roads are of the two-lane type, are 2 x 3.50 m wide, and are usually without a paved shoulder. The average daily traffic (ADT) on these roads can range from 4,000 to 20,000, with truck percentages of 15 to 20.

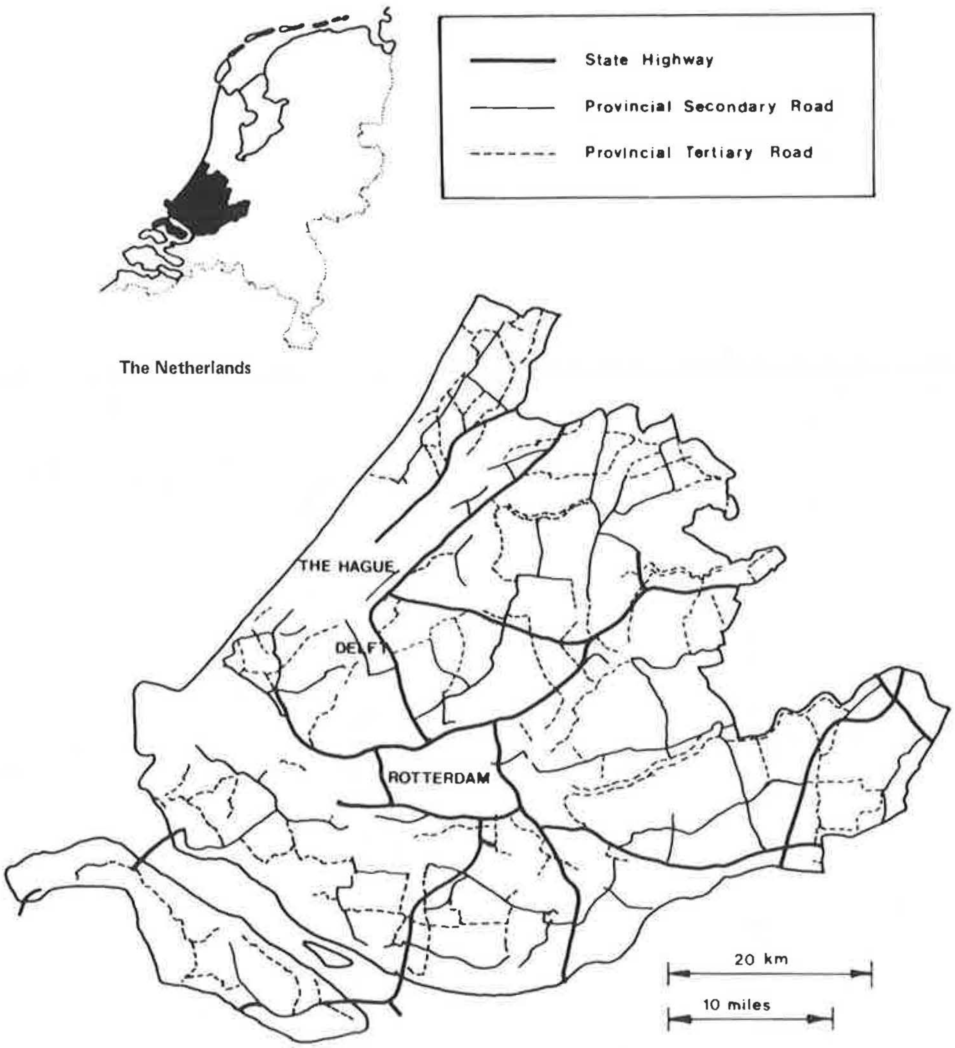
In the test program attempts have been made to find answers to the following issues:

1. Can the system be used on a large network without causing problems in the storage and retrieval of large amounts of data?
2. Can the data inventory, condition surveys, and deflection measurements be conducted within an acceptable time horizon?
3. Are the maintenance and rehabilitation proposals resulting from the survey data and deflection tests acceptable both from a theoretical point of view as well as from practical considerations, or do the various decision criteria have to be adjusted?
4. Does application of the management system lead to optimum budget allocation?

FRAMEWORK OF TEST PROGRAM

Like many currently used pavement management systems, DUPMS makes distinctions between monitoring on the network level and on the project level.

Network-level monitoring involves conducting inventories and several condition surveys to establish the current status of the roadway network. In an efficient system, these activities must be simple because they have to cover the complete network. By introducing criteria such as acceptable levels of serviceability or levels of maximum allowable dis-



Province of Zuid-Holland

FIGURE 1 Roadway network of province of Zuid-Holland.

tress, an indication of the location and extent of poor road sections in the network can be acquired. Also, a first estimate of maintenance budgets and planning of maintenance activities can be made.

Also, only selected road sections should be selected for project-level monitoring. This restriction is made to save monitoring costs and to speed up the monitoring procedure, because project-level techniques such as deflection testing and high-precision profile measurements are usually expensive and time-consuming. In some cases the interpretation of the survey results requires skilled personnel.

The structure of DUPMS, as tested on the secondary road network of the province of Zuid-Holland, consists of the following elements.

1. Network level
   a. Conduct a data inventory
   b. Determine the visual condition by general visual condition surveys, the objective being to obtain a first ranking of all sections according to their visual condition (based on this ranking a selection is made of roadway sections on which a detailed visual condition survey should be conducted)
   c. Determine the visual condition by detailed surveys to obtain data on the present visual status of the network (the results are used in the interpretation of deflection testings and in the assessment of residual pavement lives)
   d. Determine riding comfort
   e. Determine skid resistance
   f. Select roadway sections for project-level monitoring
   g. Estimate maintenance budget requirements (the results of the detailed visual condition survey can be used for a first estimate of the maintenance and rehabilitation budget needed for the next 3 years)
2. Project level
   a. Determine structural condition by deflection tests
   b. Conduct high-precision profile measurements to evaluate road roughness in a more detailed way (only on sections with poor riding comfort)
   c. Determine the texture depth on all sections with a low skid resistance
   d. Determine overlay design.

One element of DUPMS--the general visual condition survey--has not been executed in the test program. Usually this step, together with the data inventory, is the first step to be executed in the monitoring process. In this case this step has been omitted for the following reason.

Before starting the test program, the various districts of the province of Zuid-Holland, on behalf of the Provincial Highway Administration, had selected all roadway sections to which nonroutine maintenance or rehabilitation should be applied within the next 3-year period. For reasons of time, those selected sections were used in the continuation of the monitoring process because it was thought that the general visual condition survey would yield the same poor sections.

The total length of the selected sections was 110 km, or 250 lane-km, and covered around 30 percent of the secondary road network. A detailed description of the entire test program is presented elsewhere (2,3).

NETWORK-LEVEL MONITORING ACTIVITIES

Data Inventory

In a data inventory, usually data such as age, section boundary geometrics, surface type, and so forth are gathered. A delay in the execution of the monitoring process was caused by invalid, incomplete, and inefficient data storage. Therefore, it is stated emphatically that each district should frame its data bank in such a way that it can provide valid and complete data to a wide range of users.

Riding Comfort

Network-level evaluations of road roughness of the sections tested have been done in terms of riding comfort. These measurements are believed to be suitable for use on the network level because they provide a general impression of road roughness and can be conducted in a simple and fast way.

The Delft University ridemeter (4) was used in the test program. This ridemeter is a compact instrument (25 x 20 x 20 cm) that evaluates comfort based on the comfort criteria proposals of the International Standardization Organization (ISO). Vertical accelerations of the bottom of the car are measured by an external accelerometer and weighted by filters based on the ISO proposals. Subsequently, the average root-mean-square value of the signal is determined over a period of 15 sec, which is equivalent to a 200-m segment when traveling at 48 km/h, or to 333 m when traveling at 80 km/h. In the test program a traveling speed of 48 km/h was used for all sections. The obtained value is displayed on the counter of the ridemeter; it is called the ride index.

A high ride index indicates poor riding comfort. The magnitude of the ride index depends on road roughness, and also on the velocity of the car and car characteristics such as mass, springs, shock absorbers, tire pressure, and so forth.

In the test program only one type of car was used during the testing--a Mercedes Benz 508D with a gross mass of 3570 kg. The car characteristics were assumed to be constant during the 3-day testing period.

Figure 2 shows a histogram of all ride indices obtained on the Zuid-Holland roadway network. In a previous paper (1) levels of acceptable riding comfort, based on present serviceability index (PSI), have been presented for an Opel Kadett (the U.S. equivalent for this car type is the Chevrolet Chevette). These levels had to be adjusted to fit into the conditions used in the test program. Changes in the car and the level of sensitivity of the ridemeter necessitated multiplication of the levels of acceptable riding comfort by a factor of 4. For this study a ride index of 80 was considered to provide poor riding comfort, whereas at a level of 120 or higher application of maintenance strategies due to lack of riding comfort was considered to be inevitable.

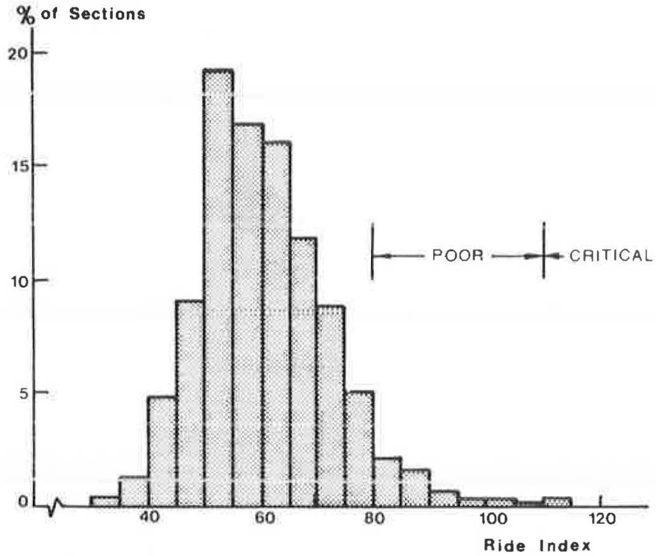


FIGURE 2 Histogram for ride index distribution.

The histogram in Figure 2 shows that only 6 percent of the surveyed sections had poor riding comfort and that only a small number did not meet the minimum level of 120. Consequently, the majority of the network provides fair to good riding comfort. No extra profile measurements were performed on the very poor sections because these sections had poor visual conditions as well. This in turn resulted in deflection testings to determine all feasible maintenance strategies. Because an improvement in the structural condition will result in an improvement in the riding comfort, the expensive profile measurements could be omitted.

Figure 3 shows the relationship between the ride index and the number of deduct points as obtained by the detailed visual condition survey. This figure shows that riding comfort measurements can be used

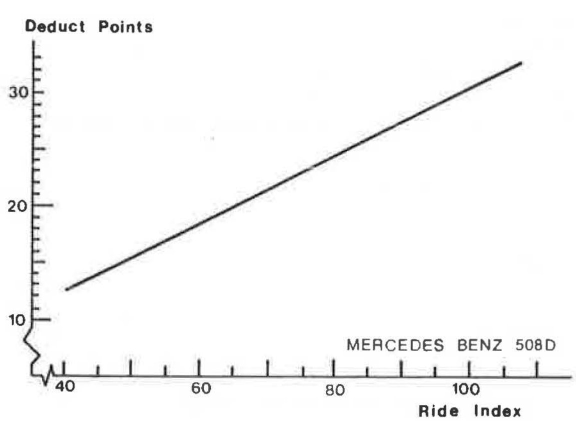


FIGURE 3 Relationship between ride index and deduct points obtained in the detailed visual condition survey.

for the selection and ranking of highway sections in terms of visual condition.

Skid Resistance

An essential requirement of all roads is that vehicles should be able to travel safely. All roadways should therefore have a surfacing with adequate resistance to skidding. In this test program the determination of skid resistance is based on the friction coefficient between the vehicle tire and the pavement surface. The actual tests were performed by the Road Engineering Division of the National Public Works Department. To measure skid resistance they used a trailer with a rolling wheel in an 86 percent slip mode, mounted with a smooth Permanent International Association of Road Congresses (PIARC) tire, towed at a traveling speed of 50 km/h. A wet skid resistance of 0.38 was considered to be the utmost minimum to be permitted on arterial roadway systems, according to proposals of Working Group R1 of the Dutch Study Centre for Road Construction (5). To avoid allowing the surface condition to deteriorate to that level, a warning level was set at 0.45.

No problem occurred in the test program on this condition aspect because a wet skid resistance value of 0.45 or more was measured on all sections. Therefore, because of sufficient resistance to skidding, additional texture depth measurements could be omitted.

Visual Condition

In DUPMS surface distress is monitored by visual condition surveys. The objective of these surveys is to establish the present status of the pavement condition by identifying the type, degree, and extent of distress. By rating these distress identifications and by setting selection criteria, the following characteristics are obtained:

1. Maintenance and rehabilitation volume,
2. Budget level,
3. Additional monitoring actions (e.g., deflection tests), and
4. Location of poor roadway sections.

For reasons of efficiency, extensive visual condition surveys should only be conducted on highway sections where the extent and degree of distress give cause to these detailed surveys. Therefore, a general visual condition survey is recommended as a first action to evaluate the present status of pavement condition in a quick and simple way. Quantity in this phase is more important than quality. Based on the status of the pavement condition, a selection of the sections where detailed surveys should be conducted is made.

General Visual Condition Survey

In DUPMS five general survey distress-type combinations are rated [see Figure 4 (6)]. These ratings express both extent and severity; they range from 1 to 5, where 1 means that no visual distress or only slight distress of limited extent can be observed, and 5 indicates that either moderate distress of large extent or severe distress is present. Usually detailed surveys are recommended when a rating of 3 or higher is assigned to one of the distress types of texture, roughness, or soundness (see Figure 4).

As previously mentioned, this general visual condition survey was omitted in the test program because the Provincial Highway Administration of the test province already provided a list with roadway sections to which some type of nonroutine maintenance should be applied within a 3-year time horizon. It was assumed that the roadway sections from the visual condition survey would be almost similar to that resulting from the general survey.

Detailed Visual Condition Survey

The system developed by Texas A&M University (7) was used for the detailed visual condition survey in DUPMS. Only slight modifications had to be applied to adjust it to Dutch circumstances. In this system the type, extent, and degree of distress can be identified, and each combination is rated according to a deduct point table.

To achieve consistency, the surveys are conducted using distress catalogs with photographs and detailed descriptions of each distress type. This consistency can be enhanced by using sheets on which the exact location and severity of cracks can be drawn (see Figure 5). Introduction of these sheets, along with the standard notation sheets (Figure 6), resulted in a remarkable improvement in consistency of the survey results from each individual survey team (8). An additional benefit of these sheets is that they can be used in the interpretation of the results of deflection tests. These sheets indicate where irregularities in the deflection basin might be explained by the occurrence of cracking.

In the test program 110 km (250 lane-km) was selected by the Provincial Highway Administration. Because of manpower and time constraints, the detailed survey could not be conducted on each section. Instead randomly selected 100-m segments of each section were chosen in such a way that of each section kilometer at least 50 percent was surveyed. It was believed that the difference in deduct points, obtained when each section was surveyed completely, and those obtained in the 50 percent mode, would be neglectable.

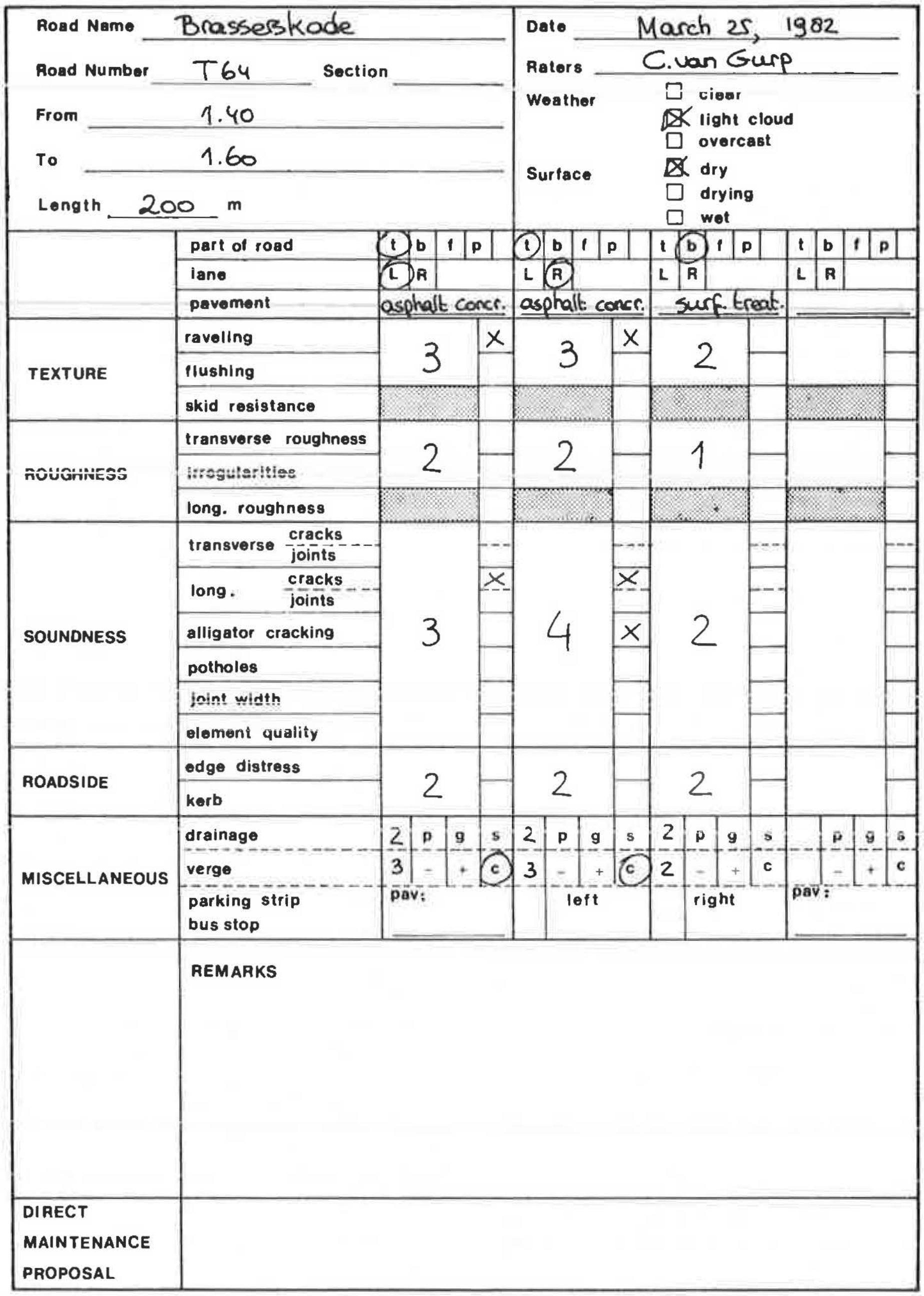
Figure 7 presents a histogram of the deduct points. It shows that only 4.3 percent of the surveyed 100-m segments has more than 40 deduct points. From previous research it is noted that not more than 40 deduct points should be admitted to avoid the risk of excessive damage caused by severe winters (9).

From the conducted surveys it could be concluded that surface defects (e.g., potholes) and rutting are of minor importance. Only a limited number of sections have these distress types. The prevailing distress type is raveling.

All survey data were processed by the computer program WBP-3 (10). This program converts the keypunched survey notations into deduct points and has options to provide graphical displays of deduct point subtotals and to select poor segments. But only the present status of the pavement condition is considered, and no indication of the degree of increase in deduct points is provided. The next section deals with this issue.

Visual Condition Performance

If the present status of a section or a roadway network is unacceptable, maintenance and rehabilitation actions can be programmed to restore the status of the network and to keep it at an acceptable level. Usually these activities have to be planned at least 1 year before actual application. Therefore, besides indications on the present status, data on the de-


Road Name Brasserskade
Road Number T64 Section
From 1.40
To 1.60
Length 200 m
Date March 25, 1982
Raters C.van Gurp
Weather ☐ clear ☒ light cloud ☐ overcast
Surface ☒ dry ☐ drying ☐ wet

	part of road	t b f p		t b f p		t b f p		t b f p	
	lane	L R		L R		L R		L R	
	pavement	asphalt concr.		asphalt concr.		surf. treat.			
TEXTURE	raveling	3	X	3	X	2			
	flushing								
	skid resistance								
ROUGHNESS	transverse roughness	2		2		1			
	irregularities								
	long. roughness								
SOUNDNESS	transverse cracks / joints								
	long. cracks / joints		X		X				
	alligator cracking	3		4	X	2			
	potholes								
	joint width								
	element quality								
ROADSIDE	edge distress	2		2		2			
	kerb								
MISCELLANEOUS	drainage	2 p g s		2 p g s		2 p g s		p g s	
	verge	3 - + c		3 - + c		2 - + c		- + c	
	parking strip / bus stop	pav:		left		right		pav:	
	REMARKS								
DIRECT MAINTENANCE PROPOSAL									

FIGURE 4 General visual condition survey sheet *(6)*.

gree of deterioration of the roadway network cannot be omitted in a pavement management system. This degree of deterioration in DUPMS is assessed by visual condition performance models. Periodic surveys provide the data for these models. Policy variables such as levels of maximum acceptable number of deduct points decide when a roadway section will enter the less-acceptable condition phase. The actual date of transition into this lower phase is a useful tool in planning maintenance activities and estimating maintenance costs. A short abstract of the model is presented in the following paragraphs. A complete description is provided elsewhere (11,12).

Visual Condition Performance Models

To compare distress types or combinations of distress with each other, the visual condition index has been introduced. This index links the current number of deduct points to its corresponding maximum:

$$P_v = 1 - (dp/dp_{max}) \quad (1)$$

where

P_v = visual condition index,
dp = number of deduct points, and
dp_{max} = maximum number of deduct points.

From periodic surveys, the decline of P_v with time could be derived:

$$P_v = 1 - \exp\{\alpha[(t/T) - 1]\} \quad (2)$$

where

t = years since construction,

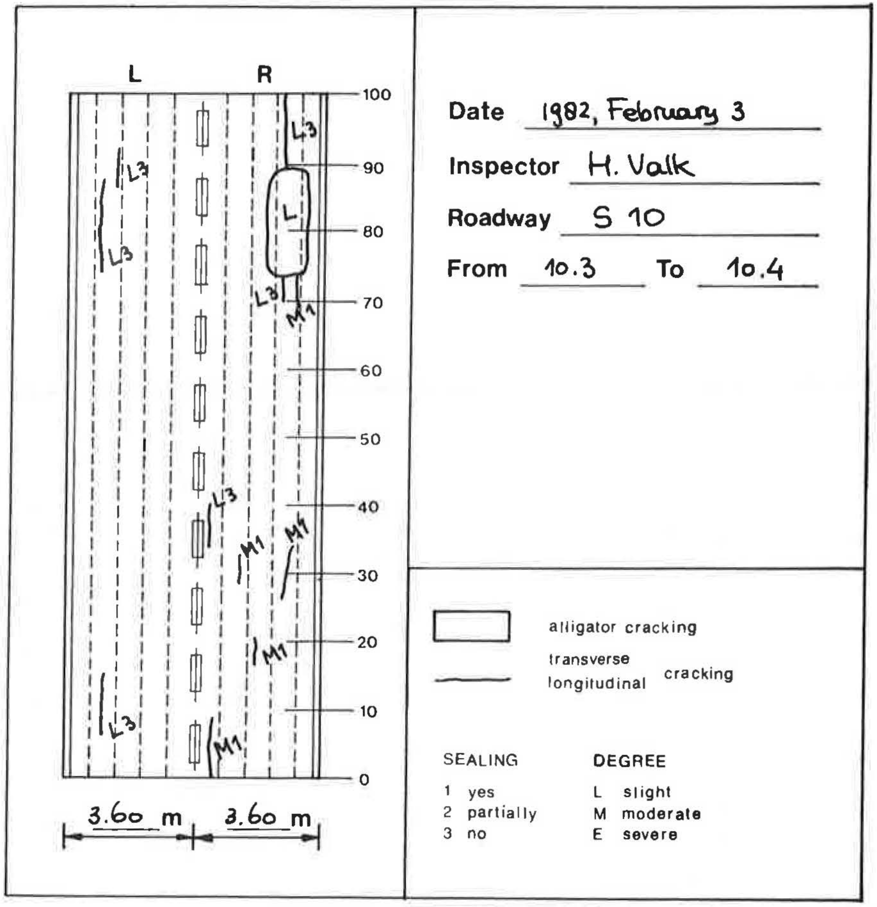


FIGURE 5 Cracking notation sheet.

General			Surface Texture												Cracking										Deformations						Verge			
Febr. 3, 1982 Date: Road S10 Number:			Raveling			Flushing			Patching			number of Core Holes	Potholes	number of Failures	Transverse Cracking			Longitudinal Cracking			Alligator Cracking			Crack Sealing 1 yes 2 partial 3 no	Rutting			Corrugations			0 not present 1-2 good 3-4 fair 5-6 poor			
			% area (1) 1-15 (2) 16-30 (3) >30			% area (1) 1-15 (2) 16-30 (3) >30			% area (1) 1-15 (2) 16-30 (3) >30				m^2 (1) 0.01-1 (2) 1-2 (3) >2		number per 100 m (1) 1-7 (2) 8-15 (3) >15			m per m^2 (1) 0.1-1 (2) 1-2 (3) >2			% area (1) 1-5 (2) 6-25 (3) >25				% area (1) 1-15 (2) 16-30 (3) >30			% area (1) 1-15 (2) 16-30 (3) >30						
	from km to km	Lane	slight	moderate	severe	slight	moderate	severe	good	fair	poor				slight	moderate	severe	slight	moderate	severe	slight	moderate	severe		slight	moderate	severe	slight	moderate	severe	Pavement Edge Condition	Shoulder Condition	Shoulder Edge Condition	Verge
1	10.1-10.2	L	1			1												2						2										
2	10.2-10.3	L		1												1		2			1			2										
3	10.3-10.4	L	1															1						3										
4	10.1-10.2	R		2		1												2			1			3										
5	10.2-10.3	R		2					1							1								3										
6	10.3-10.4	R	2			1																												
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FIGURE 6 Detailed visual condition survey sheet.

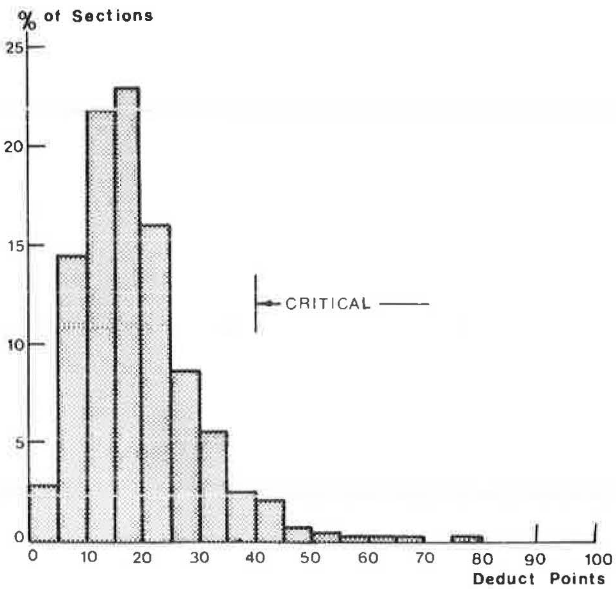


FIGURE 7 Histogram for deduct point distribution.

T = pavement life (P_v = 0), and
α = construction parameter.

Figure 8 shows the performance curve of the visual condition index. In this graph the ratio t/T is called the life index. The magnitude of the construction parameter (α) determined the shape of the performance curve. A high α value involves a steep decline of the visual condition index when the pavement life expires, whereas a low α value causes a more gradual, predictable deterioration. Usually asphalt pavements with rigid bases have high α values (α = 7 to 8), whereas constructions with unbound bases have low α values (α = 3 to 4).

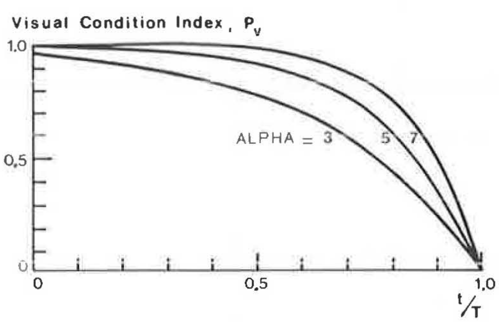


FIGURE 8 Visual condition performance curves.

By periodic surveying, P_v - t combinations enable the determination of α and T by using linear-regression techniques. For a first survey, the construction parameter (α) can be assessed by using the data in Table 1 to determine T. When more survey data are available, a more appropriate α value can be determined.

Condition Phase and Minimum Level

If P_v = 0, the maximum number of deduct points has been assigned and consequently the roadway section involved has reached its pavement life for the distress type considered. When P_v = 0, the section has failed already, and reconstruction is inevitable. To avoid large expenditures and to provide an acceptable level of serviceability, the visual condition index should not drop to its bottom value.

TABLE 1 Construction Dependency of α Value

Distress Type	Raveling	Cracking	Overall Condition
Construction type			
Asphalt layers on cement-bound base	5	7.6	7.4
Asphalt layers on base of blast furnace slags showing cementation	5	5–8.5	6–8.7
Asphalt layers on unbound base	5	3–3.5	4.3
Bituminous construction	5	5.5	5.5

Maintenance should be applied when the degree of deterioration is only limited. Figure 9 shows the process of deterioration for a number of roadway sections. It can be seen that the lower the minimum acceptable level chosen, the more deferral in maintenance will be accepted. From data of highway authorities it appears that maintenance activities should be started when the visual condition index has dropped to 0.7 for cracking and 0.6 for the overall condition. Figure 9 shows that not every roadway section has the same degree of deterioration. It was thought necessary to take this degree into account. This has been done by the maintenance urgency range; that is, the period required for the visual condition index to drop from its minimum level to its ultimate minimum level. This level is set to 0.5 for cracking and 0.4 for the overall condition. The shorter the urgency, the less maintenance can be deferred.

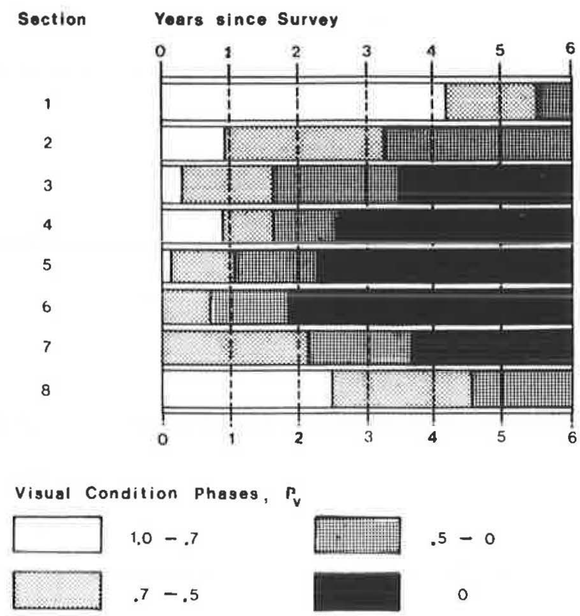


FIGURE 9 Visual condition phases.

Confidence Levels

The procedure previously described can be run for each 100-m survey segment. In the test program, however, only roadway sections selected by the districts were used. For each of these sections, the mean and standard deviation of the number of deduct points of the corresponding survey segments were calculated. The mean and standard deviation of the visual condition index could be derived from these data.

When this mean visual condition index is entered in the performance model, a mean life index and mean condition phase lives will be obtained. A mean value indicates that there is the probability that 50 percent of the section will have a lower visual condition index, and subsequently a shorter pavement life. This probability can be diminished by entering a lower visual condition index. If the combined mean minus standard deviation is entered, the probability that parts of the roadway section will have a shorter pavement life has already been reduced to 15 percent. Figure 10 shows the influence of the choice of confidence level on the condition phase length of Section 1 from Figure 9.

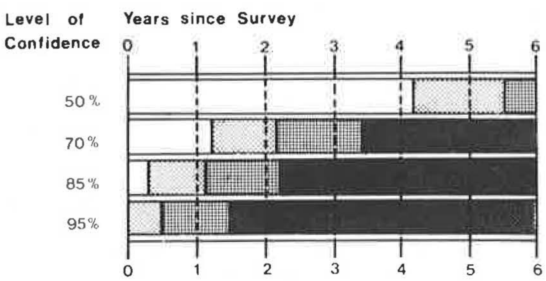


FIGURE 10 Relationship between level of confidence and visual condition phases.

Based on confidence levels, condition phases, and minimum levels, plans for maintenance activities, maintenance costs, and reinspection and deflection tests are made. For determining visual condition indices, residual lives, and condition phases, the computer program PLAIN has been developed (12).

Estimating Volume and Costs of Maintenance

In the previous section it was noted that, based on levels of the minimum acceptable visual condition index, roadway sections with high maintenance urgency can be selected. By using the visual performance models, a ranking of those sections can be made according to their maintenance urgency, which in turn can be used in planning maintenance and rehabilitation and in estimating maintenance costs. Although the total surface area to be improved is known and the data on pavement condition in terms of deduct points or visual condition index are gathered, no exact programming of maintenance or an exact allocation of funds to individual projects can be made. The network-level data are not accurate enough and do not yield information on the structural condition that is detailed enough. Therefore deflection measurements are recommended. But for a general indication of the budget required, visual condition survey results are satisfactory.

Estimating Maintenance Volume

An overlay design is usually based on the present structural condition, the structural condition at the end of the design period, and the length of the design period. The sections on Structural Condition and Overlays (presented later in this paper) will deal with this issue in more detail. They also will demonstrate how the structural condition index can be assessed from the visual condition index. This means that, based on data from visual condition surveys, an indication of the overlay required can be obtained. This overlay design will of course be rough because the conversion from the visual condition index to the structural condition index will be subjected to inaccuracies. But based on this rough overlay design, an estimate of maintenance volume and maintenance costs can be made. Evaluation of the overlay designs based on deflection measurements indicated that the visual condition data yielded an overestimated overlay thickness in some cases, whereas in other cases the overlay thickness was underestimated. However, the average overlay thickness derived from visual condition survey data overestimated the average overlay thickness, as determined by using deflection test data, by only 4 percent. Therefore this method can be used on the network level to indicate the maintenance volume and corresponding costs. For an exact allocation of funds, more accurate and detailed data are needed.

This method of determining the maintenance volume on the network level was used in the Zuid-Holland test program. A study was made of how the distribution of the total surface area of the surveyed sections to be maintained would change with changing minimum acceptable visual condition index levels and levels of confidence.

A previous section indicated what should be the minimum acceptable level and what should be the ultimate minimum level for the visual condition. Between these values, the minimum allowable visual condition indices have been varied to test the maintenance volume dependency. Three levels have been used:

1. Level A: Minimum visual condition index for cracking = 0.7, and minimum visual condition index for overall condition = 0.6;
2. Level B: Level A - 0.1; and
3. Level C: Level A - 0.2.

The determination of the maintenance volume has been performed for four levels of confidence (i.e., 50, 70, 85, and 95 percent). Figure 11 shows the relationship between the maintenance volume (expressed in surface area of the surveyed sections) and the minimum acceptable levels and confidence levels. This figure shows that there is a shift in surface area to be maintained to the first year, when a high minimum level of acceptable visual condition and a high level of confidence are chosen. This peak diminishes when one or both on these levels are lowered. Over a 3- to 4-year period the maintenance volume is less dependent on these levels. Therefore, for reasons of saving on maintenance volume, there is no need to let the pavement deteriorate to a low acceptable level.

Estimating Maintenance Costs

Deferral of maintenance leads to an increase in the maintenance budget. If the visual condition index has dropped to zero, the construction will be in such a poor state that only expensive rehabilitation can restore the pavement condition to an acceptable level. Deterioration of the condition involves crack propagation through the construction. At a visual condition index of $P_V = 0$, cracking has propagated through the entire construction, whereas at a level of $P_V = 0.7$, cracking has only propagated for 60 percent. Based on this and other results of crack growth analyses (13), it could be calculated that dropping the minimum allowable visual condition index for cracking from 0.7 to 0.6 or 0.5 will involve an overlay thickness of 1.3; that is, 1.75

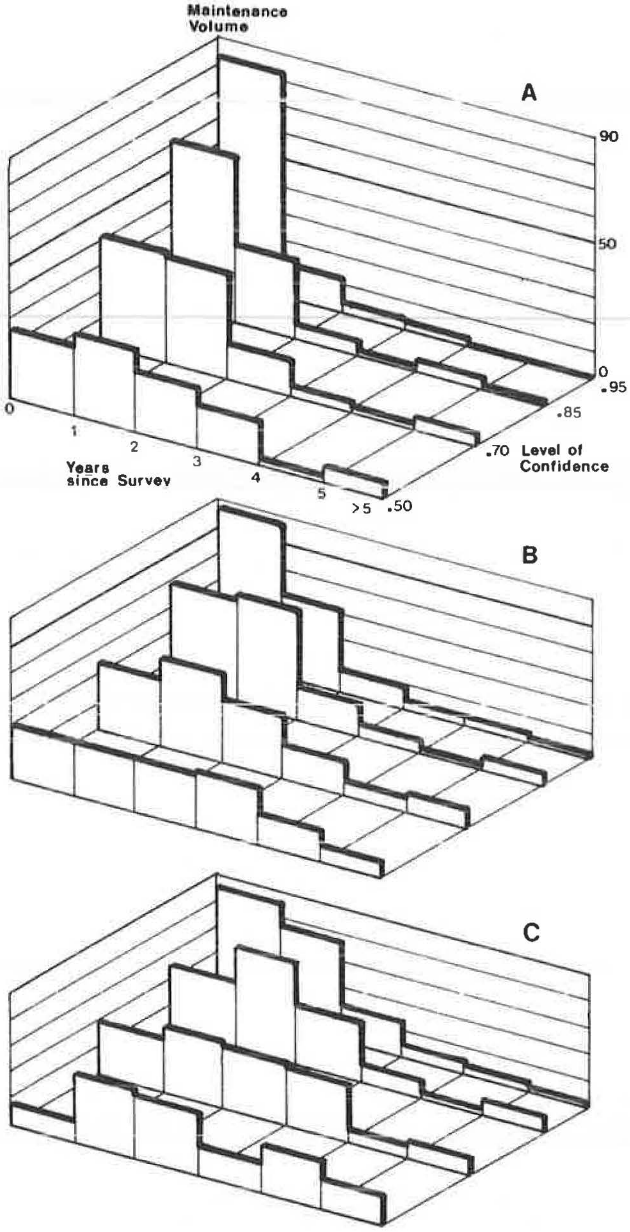


FIGURE 11 Histogram for maintenance volume distribution for three minimum levels for the visual condition index.

TABLE 2 Additional Maintenance Cost Indices

Level of Confidence (%)	Variation Coefficient of Visual Condition by Level of Minimum Acceptance Visual Condition[a]								
	0.1			0.2			0.3		
	A	B	C	A	B	C	A	B	C
50	1.02	1.19	1.53	1.11	1.29	1.58	1.22	1.34	1.59
70	1.02	1.10	1.39	1.04	1.14	1.36	1.09	1.20	1.36
85	1.00	1.05	1.27	1.02	1.06	1.20	1.04	1.09	1.20
95	1.00	1.02	1.15	1.00	1.02	1.07	1.01	1.03	1.07

[a]Level A: minimum visual condition index for cracking = 0.7, and minimum visual condition index for overall condition = 0.6; Level B: Level A – 0.1; and Level C: Level A – 0.2.

thicker than would be needed if the overlay was applied when P_V was 0.7. These factors are directives and are based on crack growth characteristics of common Dutch asphalt mixes.

In estimating the budget required, the magnitude of the variation in the visual condition index of the occurring distress is of importance. If this variation is large, then for any level of confidence there will be more sections with a poor index than in the case of a small variation. This indicates that, in the case of a large variation, each unit of surface area to be maintained requires more budget than would be needed for this same area in the case of a small variation. The data in Table 2 give the magnitude of this effect as a function of level of confidence, minimum acceptable visual condition index, and coefficient of variation in the visual condition. Figure 11 shows that in year 1, for Level A and a confidence level of 85 percent, 61.0 percent of the surveyed area will need maintenance, whereas for Level C 24.7 percent will need maintenance. When a variation in condition within the sections of 0.2 is assumed, then in the first case 61.0 x 1.02 = 61.2 unit costs and in the second case 24.7 x 1.20 = 29.6 unit costs are required.

Figure 12 shows the result of application of the data in Table 2 to Figure 11. Table 2 unit costs have been calculated for a variation coefficient of 0.2 and are corrected for inflation and rates of discount. The chosen rate of inflation is 6 percent and the rate of discount is 10 percent. The data in Table 3 give the cumulative unit costs over a 4-year period. An estimate of the actual maintenance costs can be made based on these costs.

Planning Maintenance

The data in Table 3 indicate that for the combination P = 70 percent, Level A is the combination with the lowest costs. However, 85 percent of the maintenance budget is concentrated in the first 2 years of the 4-year analysis period. For the combination P = 70 percent, Level B indicates that for an additional 2.6 percent, a more equilized distribution of the budget required will be obtained.

If no acceptable distribution can be found, the distribution can be adjusted by shifting maintenance projects forward or backward in time. The choice of which project or section should be shifted can be based on the visual performance of the project or section considered. Figure 9 shows the degree of deterioration of a number of sections. Sections 2 and 4 in this figure will enter the condition phase 0.7 - 0.5 at the same date. Section 4, however, will complete this phase more quickly and should have a higher maintenance urgency than Section 2. Therefore, if one of these sections should be dropped from the maintenance program temporarily, it would be preferable to select Section 2.

PROJECT-LEVEL MONITORING ACTIVITIES

In the previous sections of this paper all network-level monitoring activities, as performed in the test program, have been described. By introducing decision criteria, the surveyed sections can be categorized, maintenance can be planned, and the necessary project-level monitoring activities can be selected.

As already mentioned in the sections on Riding Comfort and Skid Resistance, there was no need to evaluate roughness and skid resistance in a more detailed way. The number of sections with insufficient riding comfort or skid resistance was so small that only attention will be paid to the diagnostic survey of the structural condition.

It was also noted that prediction models of pavement performance in terms of visual condition

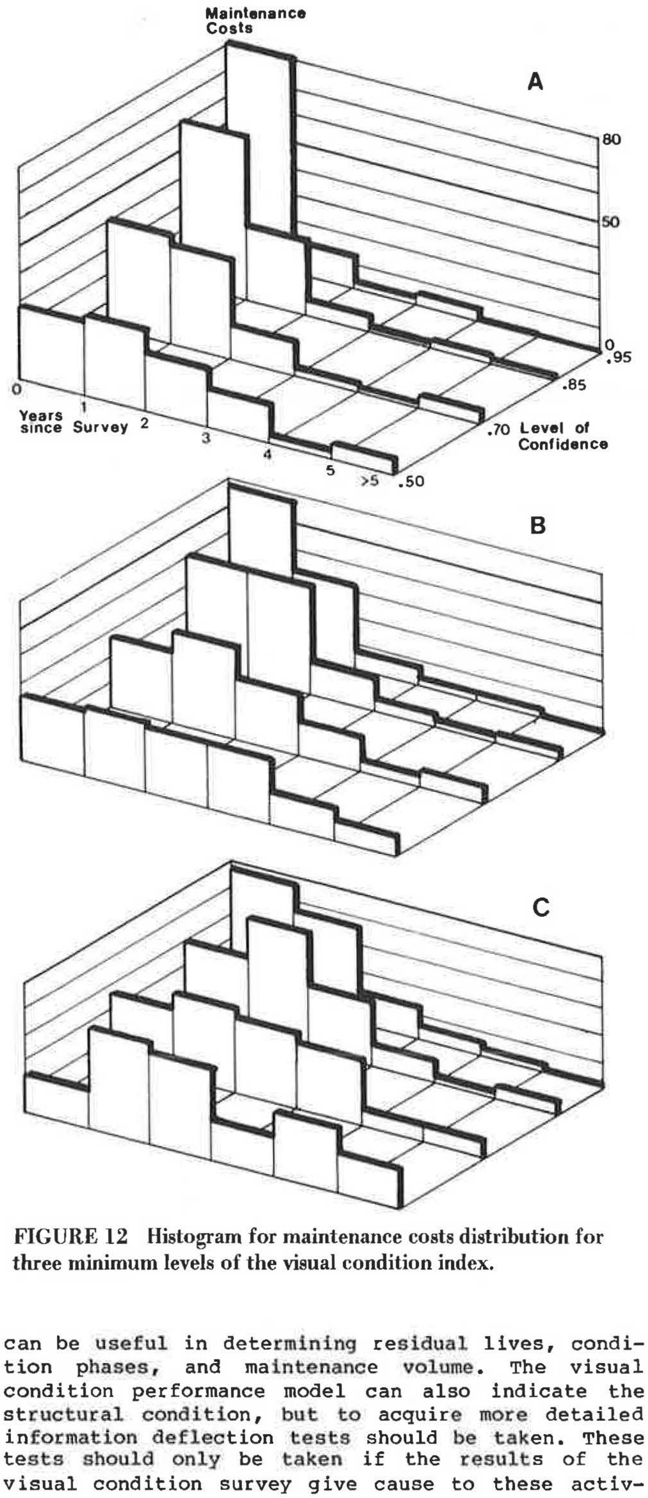


FIGURE 12 Histogram for maintenance costs distribution for three minimum levels of the visual condition index.

can be useful in determining residual lives, condition phases, and maintenance volume. The visual condition performance model can also indicate the structural condition, but to acquire more detailed information deflection tests should be taken. These tests should only be taken if the results of the visual condition survey give cause to these activities. It would take too much time, personnel, and cost if each section was monitored in this way. How these sections were selected is presented in the next sections.

Distinction is made between indicative falling weight deflection measurements (one data point every 100 m) and diagnostic falling weight deflection measurements (five data points every 100 m).

TABLE 3 Maintenance Unit Costs Over a 4-Year Period

	Level of Minimum Acceptable Visual Condition[a]		
Level of Confidence (%)	A	B	C
50	94.5	96.4	100.6
70	91.7	94.1	110.3
85	92.9	94.1	103.7
95	94.8	93.8	96.4

Note: Data are for the Zuid-Holland secondary road network, 1982.

[a]Level A: minimum visual condition index for cracking = 0.7, and minimum visual condition index for overall condition = 0.6; Level B: Level A − 0.1; and Level C: Level A − 0.2.

Indicative Deflection Measurements

Indicative deflection measurements were thought to be necessary if the visual condition index for cracking in asphalt constructions with rigid bases dropped below 0.8. These pavement structures can show a steep increase in the extent of cracking when the pavement life expires. Therefore these indicative measurements leave a larger margin in terms of time to plan maintenance strategies. Indicative measurements should also be taken if no more than 3 years have passed since construction, while the visual condition index for cracking has already dropped below 0.8. If such new construction already shows that extent of distress, the structural condition should be monitored periodically to take precautions to avoid rapid deterioration.

Diagnostic Deflection Measurements

In general, diagnostic deflection measurements should be taken if the visual condition index for cracking is 0.7 or lower. In this case the structural condition has deteriorated to such a degree that detailed information to determine feasible maintenance strategies is required.

Sometimes cracking will not be the prevailing distress type. In this case diagnostic measurements are recommended if the visual condition index for the overall condition drops below 0.7. No measurements are required if the visual condition index for cracking is still 0.8 or more. If the visual condition index for cracking is 0.8 or lower and the residual life is less than 2 years for constructions older than 3 years, diagnostic measurements must be taken to determine the most feasible strategy.

Structural Condition

In DUPMS monitoring the structural condition on the project level is done by falling weight deflection measurements. The objective of these tests is to obtain detailed information on the load-carrying capacity and the degree of deterioration. The load-carrying capacity is characterized by the equivalent layer thickness calculated to Odemark's theory (14):

$$he = 0.9 \sum_{i=1}^{n-1} h_i \sqrt[3]{E_i/E_n} \tag{3}$$

where

he = equivalent layer thickness,
h_i = thickness of layer i,
E_i = stiffness of layer i,
E_n = stiffness of layer n = subgrade modulus, and
n = number of layers.

By using this relationship, which was derived between the equivalent layer thickness and the deflection basin, the equivalent layer thickness can be estimated from deflection measurements. Because loading magnitude and temperature influence the deflections and consequently the equivalent layer thickness, adjustments have to be applied to correct for these factors. Besides these factors there is a dependency of the equivalent layer thickness on the subgrade modulus. In order to compare different constructions with each other, adjustments are applied in such a way that two identical equivalent layer thicknesses will relate to identical performances (1,13).

A maximum value of the equivalent layer thickness is determined at the moment of construction. A minimum value of the pavement condition is considered where a crack has propagated through all bound pavement layers and where no transfer of the load caused by aggregate interlock across the crack takes place. This condition of a fully developed crack coincides with a visual condition index of $P_v = 0$.

The degree of structural deterioration of a pavement structure is represented by the structural condition index:

$$P_d = he_n/he_o \quad (4)$$

where

P_d = structural condition index,
he_n = equivalent layer thickness after n load applications, and
he_o = initial equivalent layer thickness.

Figure 13 shows the decrease of the structural condition index as a function of the number of load applications and the dispersion in the layer thicknesses and layer moduli (= $S_{\log N}$). This $S_{\log N}$ value can be determined by deflection measurements (13). With use of the previously mentioned definition of the minimum equivalent layer thickness, calculations indicated that this minimum corresponds to a minimum P_d = 0.65 if the surface curvature index is smaller than 140 μm, and 0.75 if the surface curvature index is greater than 200 μm. For intermediate curvature indices, intermediate minimum structural condition indices can be used (13). This minimum index, and the corresponding number of load applications, is called N.

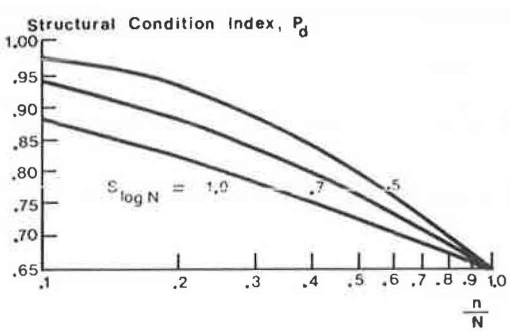
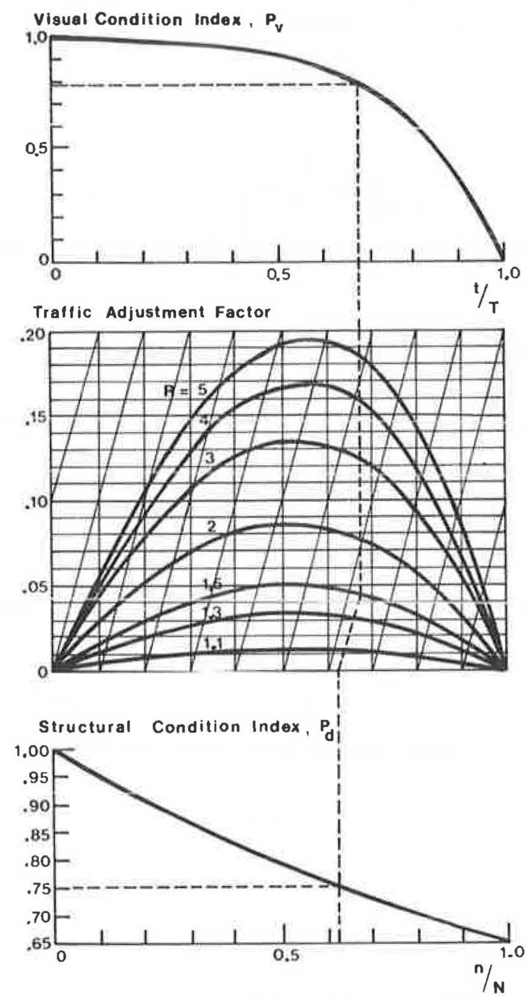


FIGURE 13 Relationship among structural condition index, allowable number of axle loads, and $S_{\log N}$ value.

For an accurate determination of the structural condition index, the equivalent layer thickness should be measured just after completion of the construction. Unfortunately, most deflection tests are only conducted on roadway sections with a poor condition. In those situations it is recommended to conduct deflection tests on locations not subjected to traffic loading to obtain candidate he_o values. In the Zuid-Holland test program, the area between the wheelpaths was used to estimate he_o. The he_n value was obtained from deflections measured in the wheelpaths.

In some cases, however, a structural condition index larger than 1 was obtained. This behavior is probably caused by extension of cracking from the wheelpaths to the area between the wheelpaths. Other reasons may be postcompaction of the granular base caused by traffic loads, which will cause a stiffening of the base in the wheelpaths, or differences in asphalt thickness in and between the wheelpaths caused by maintenance activities. In those cases the structural condition index can be assessed from the visual condition index (see Figure 14). The surface curvature index measured sets the minimum structural condition index. From the visual condition surveys, a life index t/T, which is based on years, is determined. This index can be converted to an index based on traffic intensities or axle load repetitions when the ratio of traffic intensity in year t=T over traffic intensity in year t=0 is known. The parameter $S_{\log N}$, as determined by deflection testings, determines the exact shape of the structural performance curve. But to avoid these conversions and to determine a proper value for the structural condition index, deflection measurements should be taken just before a new pavement construction opens to traffic.



Note: R = ratio of traffic intensity in year t = T over traffic intensity in year t = 0.

FIGURE 14 Conversion from visual condition index into structural condition index.

Overlays

In the Zuid-Holland Test program only overlays were taken into account as maintenance activities. The main objective for applying this type of maintenance was to reduce stresses and strains in the existing pavement structure and to reduce crack propagation through both the existing pavement structure and the overlay. The reduction required is a function of the structural condition required at the end of the design period, the amount of traffic to be carried, and the probability that the overlay design will be successful.

The overlay design method used in the Zuid-Holland test program has been outlined elsewhere (13,15). Here the most important aspects are summarized. The number of axle load applications (N_i) that the pavement can sustain for a probability of survival (P_i) is calculated from

$$\log N_i = a_o + a_1 b_o + a_1 b_1 \log he_i - u_i S_{\log N} \quad (5)$$

where

he_i = equivalent layer thickness,
u_i = standardized normal deviate associated with a probability P_i,
$S_{\log N}$ = standard deviation of the logarithm of the number of load repetitions to failure,
a_o, a_1 = constants from the relation $\log N = a_o + a_1 \log \varepsilon$, and
b_o, b_1 = constants from the relation $\log \varepsilon = b_o + b_1 \log he$.

If pavement life has to be extended, the required equivalent layer thickness can be calculated by using Equation 5. If N_1 is the number of allowable load applications for an equivalent layer thickness (he_1) and a probability (P_1), and if N_2 is the sum of the number of axle loads to be carried in the design period and for N_1, than for a probability of survival (P_2) the required equivalent layer thickness (he_2) can be calculated as follows:

$$\log (N_1/N_2) = a_1 b_1 \log (he_1/he_2) - u_1 S_{\log N} + u_2 S_{\log N} \quad (6)$$

if

$$J_i = 10^{u_i S_{\log N}} \quad (7)$$

Then Equation 6 can be rewritten into

$$he_2 = he_1 \sqrt[a_1 b_1]{N_2 J_2 / N_1 J_1} \quad (8)$$

The overlay thickness can be calculated by using Odemark's (14) theory:

$$h_o = (he_2 - he_1)/0.9\sqrt[3]{E_o/E_s} \quad (9)$$

where

h_o = overlay thickness (m),
E_o = overlay stiffness (MPa), and
E_s = subgrade modulus (MPa).

Note that only information on the amount of traffic is needed in terms of a ratio for past traffic (see ratio N_2/N_1 is Equation 8). Exact knowledge of the axle load spectrum is not strictly necessary. Furthermore, note that an exact asphalt fatigue relation does not need to be entered, but that it can be confined to data on the slope of this relation.

The chosen overlay design life was for a 10-year period. Valk (16) indicated that an overlay design period of 10 to 15 years will result in the lowest costs. The structural condition index required at the end of the design period was set at 0.85, which involves only a slightly cracked pavement. The probability of survival required was set at 85 percent. The data in Table 4 give the overlay thicknesses, calculated in this way, for all the selected sections. Also given are data on what overlay thickness would be necessary if the determination was only based on construction data and visual condition surveys. Although for some sections the difference in overlay thickness can be up to 40 mm, the average thickness required equals that determined by using structural condition data. This proves that, on the network level, maintenance volume and costs can be estimated by visual condition surveys, but that for an accurate allocation of the budget to individual projects, structural condition data, as determined by deflection measurements, are necessary.

TABLE 4 Overlay Thickness for 10-Year Design Period

	Overlay Thickness (mm)		
Roadway Section[a]	Based on Deflection Measurements	Based on Visual Condition Surveys	Difference in Overlay Thickness (mm)
S 7 - L 4.6 - 5.0	87	79	−8
S 7 - R 4.6 - 5.0	86	79	−7
S 7 - L 6.3 - 6.9	71	81	+10
S 7 - R 6.3 - 6.9	73	81	+8
S 7 - L 11.8 - 12.3	31	52	+21
S 7 - R 11.8 - 12.3	81	52	−29
S15 - L 1.6 - 2.0	31	48	+17
S15 - R 1.6 - 2.0	20	41	+21
S15 - L 4.4 - 5.4	82	64	−18
S15 - L 5.4 - 6.0	67	64	−3
S15 - R 4.4 - 6.0	71	64	−7
S22 - L 0.9 - 2.0	39	79	+40
S22 - R 0.9 - 2.0	60	79	+19
S22 - L 3.4 - 5.1	71	83	+12
S22 - R 3.4 - 3.8	47	83	+36
S22 - L 23.3 - 23.4	66	43	−23
S22A - R 3.1 - 3.5	24	21	−3
S29 - L 9.0 - 10.0	25	19	−6
S29 - R 9.0 - 10.0	19	19	0
S30 - L 6.2 - 7.2	70	51	−19
S30 - R 6.2 - 7.2	68	51	−17
S36 - L 16.8 - 18.0	11	23	+12
S36 - R 16.8 - 18.0	16	23	+7
S40 - L 3.9 - 4.8	16	35	+19
S40 - R 3.9 - 4.8	23	35	+12
S47 - L 30.9 - 31.9	68	53	−17
S47 - R 30.9 - 31.9	68	53	−17
Avg	52	54	+2

Note: Data are for the Zuid-Holland secondary road network, 1982.
[a]Roadway sections selected by visual condition surveys.

CONCLUSIONS AND RECOMMENDATIONS

The results of tests of the DUPMS to a provincial roadway network have been presented. Some of the key issues involved in the test program are summarized as follows.

1. For a reliable assessment of pavement lives and a well-funded determination of maintenance or rehabilitation, an efficient structured accessible data bank is a prerequisite.
2. For the selection of maintenance projects, general visual condition surveys are recommended. Periodic surveys provide information on the performance of the visual condition.
3. A good assessment of the condition of roadway sections can be obtained by detailed visual surveys. To acquire uniform and consistent data, attention should be paid to the training of the inspectors.

Using sheets where the exact location of cracking can be drawn has proved to be valuable.

4. Visual condition surveys can be used for planning maintenance activities and estimating maintenance costs.

5. In the determination of the structural condition index, deflection tests should taken just after finishing the construction or application of a major maintenance strategy.

6. If the data mentioned in item 5 are not available, a comparative structural condition index can be obtained by conducting deflection tests both in and between the wheelpaths.

7. In roadway sections with a large extent of cracking, visual condition surveys have proved to be useful in the interpretation of the deflection basin.

8. Determining the overlay thickness (on the project level) by using only visual condition survey data and construction data is not recommended. On the network level, however, a reliable estimate of the average required overlay thickness can be determined in this way. On the project level, deflection tests should be used for an accurate determination of the structural pavement condition and the overlay thickness.

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Developing Pavement Management Systems at County and City Levels

WES WELLS

ABSTRACT

The experience of San Francisco Bay Area cities and counties in collectively attempting to improve pavement maintenance practices by use of a pavement management system (PMS) is discussed. The development of this tool is viewed as a major factor that is necessary to assist in securing additional road maintenance revenues and in improving performance in an environment of limited revenues. The findings of the Metropolitan Transportation Commission, the agency that served as the catalyst for this effort, are summarized. These findings should have relevance and broad applicability as many other cities and counties begin developing or upgrading their pavement maintenance capabilities. Better understanding about what a PMS is, what it can do, and what should be considered before such a system is implemented are addressed. A user's manual to help guide the implementation of PMSs will be developed as these efforts continue.

The San Francisco Bay Area includes 94 cities and 9 counties. There are 1,400 miles of freeways maintained by the state. However, the focus of this report relates to the 11,000 miles of streets maintained by the cities and the 6,000 miles of roads maintained by the counties. These 17,000 miles of local roads were budgeted at $18 billion in replacement costs, making it the single biggest public investment in the Bay Area.

The Metropolitan Transportation Commission (MTC) is the metropolitan planning organization (MPO) for the region. It was created by state legislation in 1970. Historically, most of its staff have concentrated on administering federal and state transit grants and preparing and updating a regional transportation plan and a transportation improvement program. However, in early 1981 a group of public works directors came to MTC seeking help in documenting local street and road maintenance needs.

An ensuing study, taking 18 months to complete (1,2), documented numerous maintenance shortfalls, which have since fostered substantial follow-up activities. Three recommendations from that study were as follows: (a) significant new additional revenues must be found (in aggregate about $170 million was being spent, whereas it was estimated that roughly $310 million was needed); (b) local officials and the general public were largely unaware of the problem, and a large-scale public information program was needed; and (c) significant cost savings could be achieved through improved maintenance practices. The subject of this paper deals largely with the latter recommendation.

BACKGROUND

Reviewing Bay Area Maintenance Practices

In initially estimating maintenance needs, surveys of street pavement conditions were required. It became evident in seeking condition data that few cities or counties had systematically documented their information. Moreover, even fewer jurisdictions built road maintenance budgets or planned annual or multiyear maintenance programs on an automated pavement management system (PMS) basis. The definition of PMS used in this paper is as follows: An integrated set of systematic procedures designed to assist engineers and managers in making consistent and cost-effective decisions related to the design, maintenance, and restoration of pavements. Arguably, it appeared prudent to try to establish some sort of prototype PMS.

Apparently, others had the same idea. In early 1982 a proposal was submitted to MTC to develop such a system. The proposal was to take 5 years and cost roughly $1.5 million. Ten to fifteen cities and several counties were to jointly participate in the development of such a system. Public works directors from several major Bay Area cities and counties met with MTC during the course of several months to evaluate the proposal. The group concluded that 10 jurisdictions could probably come up with $30,000 per year for 5 years, but this was not the direction to go at that time. Instead, it was recommended that a pavement management evaluation committee (PMEC) be formed, made up of pavement experts from local jurisdictions who had been working in this general area. This group was both to review their collective experiences with PMSs and formulate next-step recommendations that MTC, acting as a catalyst and facilitator, would help implement.

Representatives from 15 jurisdictions met over the course of six monthly meetings in the latter half of 1982. Most of the time was spent reviewing the positive and negative experiences that four jurisdictions had encountered in working with PMSs. Four consultants with PMS experience also presented their systems and made various suggestions on what to do and what not to do. The collective recommendations of this PMEC resulted in MTC securing the services of a consultant to assist the jurisdictions in improving their pavement management practices.

The Consultant Effort

The effort of using a consultant to help MTC act as a catalyst marked a significant departure from the prototype PMS development work mentioned earlier. It was a recognition that a step backwards was necessary (i.e., even more reconnaissance was needed). There was strong sentiment from city and county jurisdictions that they did not want to "reinvent the wheel." There was also a strong indication that there were issues beyond merely developing a PMS that had to be addressed. [These issues will be discussed in greater depth later; but several of them are as follows: there was a need for stronger management support and emphasis in this area (pavement maintenance is generally not at the top of the public works director's list), there was a need to gain better public understanding and support, and it was important to recognize the different and sometimes competing needs within a public works department from budgeting, engineering, maintenance, and administration.]

To be able to go beyond the development of a PMS and to grasp some of the issues just mentioned, the expertise of a consultant was required, which proved difficult to find. Many consultants in this field were well skilled in developing maintenance management systems, but this dealt more with work flow, scheduling, and tracking, which was not the management system desired. Several consultants had developed excellent PMSs but could not see beyond the need to sell their own systems.

What was desired by MTC, beyond the capability to provide a useful PMS, was to have such a capability grow and increase in utility in subsequent years. Therefore an understanding of the public works milieu was required. The ability to communicate with the technician, the administrator, the engineer, top management, the public, and elected officials was needed. Other desirable skills of no less importance included the ability to (a) conduct training classes and lead seminars, (b) write clearly and succinctly at several levels, and (c) have a national perspective on pavement maintenance experience.

In the remainder of this paper the major findings acquired from the 18-month study of maintenance needs, the reviews of pavement management practices with local public works personnel, and a questionnaire that surveyed local perceptions of PMSs and other maintenance problems and needs are discussed. It is primarily findings from these three areas that have helped define the scope of work for the consulting contract that is currently under way. It is believed that these findings will have broad applicability as many other cities and counties begin to develop or upgrade their pavement maintenance capabilities. In the concluding portion of the paper the basic orientation that has evolved in the Bay Area because of the findings is described. The basic products and activities that will be produced as a result of this contract are also described.

FINDINGS

Findings from the 18-Month Study of Bay Area Local Road Maintenance Needs

The overwhelming conclusion drawn from this study was that the Bay Area's 17,000-mile local road system was not being adequately maintained. It was noted that local jurisdictions were actually falling further behind. That is, roads were deteriorating at a rate faster than they were being repaired. Roughly $170 million was being spent annually for road maintenance purposes:

Maintenance Category	Expenditure ($000,000s)
Preventive	41
Routine	31
Nonpavement	
Street lighting	29
Traffic safety	18
Street cleaning	10
Landscaping	11
Miscellaneous	11
Other	
Special programs	5
Administrative, engineering	14
Total	170

The evaluation indicated that $270 million (or a shortfall of about $100 million) should have been spent. (More important, 65 percent of this shortfall was for preventive maintenance.) The money was spent only for ongoing maintenance, or maintenance that was required to keep roads in adequate condition. A backlog of $400 million was also documented. This would have been the amount required to bring roads that had deteriorated because of deferred maintenance back to adequate condition. Converting this shortfall to a 10-year capital improvement program would have meant the total required maintenance expenditure would have to be $310 million, or roughly 80 percent more than the current level (3).

Moreover, the revenue shortfall was found to have been part of a gradual revenue reduction that had been taking place for more than a decade. This bleak revenue situation underscored deeper seated problems associated with staff resources, morale, and methods of planning, programming, and budgeting. For example, a typical city's or county's overall approach at budget time was reduced to one of picking which line items in the prior year's budget were to absorb the new cuts. In some cases long-term revenue decline had fostered complacency and a sense of lack of urgency from top management.

The shortfall was determined by conducting a windshield survey of visual pavement distress in 11 jurisdictions representing about 10 percent of the Bay Area's road mileage. In aggregate, 55 percent of the pavements were noted to be in adequate condition and the remainder required some sort of corrective maintenance. Twenty-five percent required seals, 15 percent required overlays, and 5 percent had deteriorated to the extent that complete restoration was required.

It was also found that wide variations existed among jurisdictions. There were significant differences in the conditions of pavements depending on not only the quality of past maintenance activities, but also on such factors as the commitment of the council, the age of the streets, and the growth in traffic. There were variations in how maintenance was performed and the types of treatments used. For example, some jurisdictions tried to direct limited funds to higher-order streets (arterials), whereas others merely repaired on a complaint basis with no overall strategy. There was also wide variation in the amount of surface preparation done before the application of seals and overlays. In summary, there was diversity in pavement condition, maintenance strategies, and maintenance treatments and their application. Most of the problems that were found across jurisdictions could be substantially improved by the establishment of or improvements in PMSs.

However, the conditions previously described underscore the futility of merely upgrading PMS capabilities if those other issues are not addressed. If pavement maintenance practices were expected to be improved, the first need had to be additional revenues, but this was only the first step. Issues such as public, council, and even management priority and support; staff adequacy as well as morale and motivation; maintenance strategies that seek ways to hold the line instead of constructing what-if scenarios that do not speak to reality; as well as a host of other issues must all be dealt with head-on.

Findings from Working with Local Public Works Personnel

In the initial efforts to develop a large-scale prototype PMS, six public works directors were asked to review the 5-year, $1.5 million proposal. They concluded that a common mistake was to be too ambitious initially, particularly in terms of what was actually needed. They also indicated that PMSs were often oversold, and worse, were developed beyond the jurisdictions' resources so that the resultant system was only partial, or not usable in an ongoing sense, particularly by maintenance personnel. The orientation was often theoretical and did not fit the needs of the jurisdictions by considering the public works milieu.

The public works directors also pointed out that PMS was such a broad concept that it could mean all things to all people. Therefore it was difficult and confusing to talk about a PMS. For example, a public works director may see a PMS as a tool that primarily helps to build a road budget. A road chief may want to have a system to schedule treatments and allocate personnel. Engineers may want a system to test alternative maintenance strategies. The maintenance researcher may want to build a pavement performance model for a maintenance optimization process. Finally there is the programmer whose primary interest is the logic systems that organize, manipulate, and display the data. Because of these numerous confusions and varying perceptions, it would appear necessary, particularly during the initial steps, to clearly define the major users and what level of effort would be required to develop specific PMS elements for specific needs.

The public works directors' advice was to not reinvent the wheel, but rather to build on the substantial body of experience that existed within local departments, and build incrementally. Different jurisdictions were at different steps in developing a PMS; what was needed was to use the best parts of existing systems. Other jurisdictions already had the necessary information, but they lacked the knowledge of what to do with it or the computer expertise to process it. They therefore concluded that MTC should take personnel from cities or counties who had some experience and use this group's experience in charting a new course or direction for PMS development.

Therefore PMEC was formed, and their collective experience was reviewed over the course of six monthly meetings. They made five major recommendations.

1. Develop guidelines to help jurisdictions improve maintenance practices: This would be a user's guide that contained sections pertinent to understanding, promoting, and implementing a PMS.

2. Develop techniques to promote standardization: Because many cities and counties were considering the development of a PMS, it was thought that an opportunity to achieve some standardization could provide real cost savings in three ways: uniformity in measuring pavement distress (common elements would be gathered because individual jurisdictions might not use all measures or may weigh measures differently), uniformity in treatment options, and potential uniformity through centralized computer, equipment, and inventory procedures and personnel.

3. Improve communication: The key to this recommendation was the recognition that clear information and education was needed at several levels, such as information and visual aids for officials and the public, general information for management, and materials and techniques for training. The following improvements were needed: information on the effectiveness of new maintenance treatments, educational materials for elected and administrative officials on the potential benefits from PMSs, increased sharing of mutual problems and practices, and more relevant training for engineering and road maintenance personnel.

4. Develop three basic elements of a PMS--pavement condition index, maintenance treatment options, and a technique to match the index to the options: The development of these three elements is viewed as occurring incrementally, beginning with manual systems and moving toward full automation. Current thinking has these elements being developed at three different levels depending on the varying sizes and needs of each jurisdiction.

5. Gather more comprehensive information to better define maintenance problems and needs by jurisdiction size: Given the variations in experiences among jurisdictions, PMEC suggested that a questionnaire be prepared that surveyed all cities and counties. In this way information could be stratified by grouping common responses to such things as experience in implementing PMSs, perception of maintenance problems and needs, securing views on potential facilitator and catalytic functions, and other needs or deficiencies that might have been overlooked.

Findings from the Questionnaire

All Bay Area jurisdictions were sent a questionnaire in June 1983. To date, 8 of 9 counties and 40 of 94 cities have responded. This represents jurisdictions responsible for 75 percent of the 17,000 miles of local streets and roads.

The following maintenance problems, in priority order, were listed as the most serious:

1. Lack of resources (both revenues and staff),
2. The ability to design an overall maintenance strategy,
3. Cost-benefit information on various maintenance strategies, and
4. Knowledge of road conditions.

Two other problems frequently cited were decisions on maintenance versus construction and lack of council or board support.

In an attempt to more accurately measure the extent of PMS development, city and county personnel were asked if they had PMSs that were implemented and functional. Eight said yes, 10 said they were in the implementation stage, and of the remainder 8 were at the developmental stage and all but 4 said they were interested in implementing a PMS. Of the four that indicated no interest, three said they were too small and lacked the necessary funds, and the fourth indicated they did not know what a PMS was.

A PMS can be developed at various levels of sophistication and at various levels of data gathering. Cities and counties can also implement a few basic elements or all possible elements of a PMS. To better understand this issue, additional information on desirable levels of data, sophistication, and elements of a PMS were gathered. PMS information cited as being of greatest utility, in priority order, were pavement condition, maintenance history, design and construction data, structural capacity, average daily traffic, and functional class. Most respondents assigned much lower priorities to ride quality and skid resistance. PMS elements cited as being of greatest utility were identification of street conditions, identification of required maintenance treatments, and budget data on needs. Elements rated as slightly lower in priority included projection of future pavement condition, economic analysis of alternatives, and determination of cause of deterioration.

These answers confirm the finding that most jurisdictions initially want the three basic elements of a PMS: (a) a process to measure pavement condition, (b) a list of the most cost-effective maintenance treatments to correct the problems identified in the pavement condition measurement, and (c) a means of matching treatments to problems by street segment so that priorities can be established. Two underlying issues relate to varying responses based on jurisdiction size and stage of PMS development.

Additional analysis indicated that larger jurisdictions, and those jurisdictions with more developed PMSs, consider the more advanced PMS elements (projections of future conditions, alternative network analysis) to also be of high utility.

Correcting maintenance practices goes beyond just the development of PMS. Jurisdictions were asked to indicate what types of information sharing would be most useful. Roughly 85 percent of the responding jurisdictions indicated that forums for periodic information exchange, training on pavement inspection and other aspects of pavement maintenance, seminars on maintenance options and treatments, and seminars on PMS experience would be useful.

Only 30 to 40 percent indicated that monthly bulletins on bid prices or joint purchases would be useful. This information indicates a high level of interest in both developing and expanding PMSs, and improving other areas of pavement maintenance practices.

RECOMMENDATIONS

Orientation

It is worth summarizing the major overall orientation gained from synthesizing the experiences and findings from the Bay Area. Before considering the development of a PMS, jurisdictions should be aware of the factors presented in the following list. These factors represent the major findings that ought to be directly addressed before developing a PMS. A jurisdiction can easily spend in excess of $100,000 for a system and discover later that spending more time on deciding what was needed and phasing that process in incrementally could save dollars and increase utility.

The major factors to be aware of before implementing a PMS are as follows:

1. Most cities and counties already have PMSs. It is only a matter of how complete, automated, and sophisticated these systems have become.
2. There appear to be great opportunities for several jurisdictions to pool efforts in developing PMSs: (a) standardization of basic PMS elements can promote cost savings, centralization of some data and computer functions, and a better basis for ultimately comparing effectiveness of treatment options; (b) information sharing about positive and negative PMS experiences becomes increasingly important as more are developed (it also helps to alleviate the "black box" syndrome); and (c) technology transfer does not occur readily, and therefore seminars on new maintenance treatments, materials, and techniques are needed.
3. Two major needs are readily apparent in most cities and counties: What are the most cost-effective maintenance treatments for similar pavement conditions? What overall maintenance strategies are most cost effective given certain budget levels? (It is critical to be able to determine whether current budget levels are gaining or losing ground in terms of keeping streets in adequate condition.)
4. Different size cities have different needs and capabilities. In the Bay Area roughly one-third of the cities have less than 50 miles, one-third have 50 to 150 miles, and the other third have more than 150 miles. Probably two-thirds of the cities (those having up to 150 miles of streets) will have little need to develop PMSs beyond the three basic elements. (Only nine jurisdictions have systems greater than 500 miles.)
5. Locating a PMS in a public works department is no minor matter. All of the following disciplines can play a vital role: engineering, maintenance, planning, and budgeting. Locating a PMS on the periphery where interrelationships are weak can limit or doom its utility.

Many of the variabilities across jurisdictions that were discussed earlier argue convincingly for a great deal of tailoring at the front end of PMS developmental efforts. However, the experience of actually getting 11 jurisdictions to conduct pavement condition surveys, and completing this effort in several months, also argues convincingly that there are great opportunities for standardization, sharing, and centralizing some functions as well.

What follows are several overall orientations that should be considered and emphasized. First, a PMS is merely a tool. For that tool to work effectively it must be supported, understood, and used by the management and personnel in each jurisdiction. The latter point is key. In smaller public works departments where the public works director, the budget analyst, the engineer, the maintenance supervisor, and the data processor may be the same person or only a few, the fragmentation may not be critical. But in larger departments, for example, if the engineers implement a PMS, but it is not really used to develop budgets, secure funds, and implement a maintenance program, the investment might as well not have been made. This suggests that all of these groups must not only be involved but must be addressed in terms of organization, education, training, and required performance.

A related but more implicit part of this view concerns the broader context into which a PMS is developed. That is, effective road maintenance can only be achieved if practices above the public works department level are addressed (e.g., citywide budgeting and priority setting processes). As stated earlier, in the Bay Area the process of estimating what it would take to keep roads adequately maintained appears to have been replaced by trying to hold the line against further cuts. When maintaining the status quo or retrenchments become institutionalized, strategies developed across broad fronts must be addressed if there is to be any hope of improving pavement maintenance practices.

A second overall orientation is in a sense directly related to the first. It is important to keep in mind that in preparing reports, different requirements in the political, financial, technical, and management fields must all be addressed. This means that to have a PMS generate 30 different reports on different maintenance priorities is not adequate. Different audiences (elected officials, the public, engineers, administrators, and so forth) all require different information presented at different levels. Technical manuals, slide shows, simply illustrated summary reports, and the like are all necessary.

A third overall orientation relates to PMSs at several levels. The development of a PMS is given in the following outline. The process assumes starting with a simple framework and using existing information as much as possible. The system should be developed at the network level first, with a strong link to the budget development process. The system should provide a list of segments that require maintenance by maintenance options, alternative strategies to achieve required maintenance with costs by year, and identification of the match or mismatch of required costs with anticipated revenues. The work flow framework is as follows:

- I. Preliminaries
  - A. Decide what is wanted based on needs and resources
  - B. Define initial required data elements to define pavement condition, including survey process
  - C. Define road system by segment and functional class
  - D. Develop a data management package (start with a simple system and automate incrementally)
  - E. Set up an agreed on institutional and organizational structure with time lines and management reviews
- II. Basic elements
  - A. Construct pavement condition scales
  - B. Formulate cost-effective treatment options
  - C. Develop a logic system to match street segments to treatment options
- III. Enhancements
  - A. Develop prediction models of pavement performance as pavement condition surveys are repeated
  - B. Develop maintenance strategies using optimization techniques

The previous outline gives the basic elements of a PMS, but the elements are not monolithic in either sophistication or in when they are implemented. Although a great deal of individual tailoring is necessary, a prototype PMS is possible. It is only that different jurisdictions start at different points in the process timewise. Different jurisdictions may also only choose to implement the three basic elements in a manual process. Others may stay with the three basic elements but move to more automated processes. A few may have needs and the necessary resources to add all the enhancements and fully automate on a large frame data base manager.

In addition to the overall orientation, it is useful to also emphasize and consider important points that would apply within individual elements of a PMS or in the design of the implementation process.

1. Do not underestimate the data management and computer aspects. Included here would be the importance of clarifying what data are gathered, when, and at what levels. The complete street segment inventory versus sampling, as well as updating, are also not trivial matters.

2. There are many ways to measure pavement distress, but relying on a single dimension index is probably an oversimplification. For example, to meaningfully relate treatments to conditions, the condition should discriminate among underlying causes such as subgrade drainage failures or excessive heavy traffic. This suggests differing levels of measurement at different stages (e.g., at the network level visual distress may be sufficient, but at the project level, once a class of segments has been highlighted as deficient, more detailed measurements including deflection and core samples may be required).

3. In developing PMS elements, it is important to remember not only the building-block approach, but also that elements should be modular. That is, it should be possible to upgrade one PMS element to a more comprehensive option without having to change any other elements.

Consultant Work Tasks

The major focus of the consultant effort will be to produce a report composed of stand-alone modules, which will act as a user's guide to developing PMSs (see Table 1). Some of these modules will be technical in orientation, others will be simple summaries for lay audiences, whereas others may actually be packaged slide presentations stressing the importance of developing systematic pavement maintenance strategies.

Most of the modules highlighted in Table 1 are self-explanatory at this stage in their development. However, module 2 does require some additional explanation, particularly because the economic benefits of PMSs have seldom been clearly documented. Economic benefits at two levels will be illustrated: (a) different combinations of maintenance techniques on different distress conditions, and (b) a structured PMS.

TABLE 1 User's Guide to Developing PMSs

No.	Module	Purpose
1	Evaluation of successes and failures in PMS efforts	Through case studies and actual experiences with PMSs, document major causes of failures and requirements for success
2	Economic analysis of elements of PMS	Such an analysis will help illustrate to decision makers the economic benefits of (a) different combinations of maintenance techniques on different distress conditions, and (b) a structured PMS
3	Questions concerning PMS	Major questions voiced by public works directors, managers, and engineers will be compiled and addressed
4	Define, describe, document, and chart PMS elements	Major elements will be summarized, different implementation levels will be described, and a step-by-step procedure for moving up the ladder will be provided
5	Potential catalytic and facilitator functions to promote PMS	Utility of following functions will be analyzed: training, seminars, centralized data structures and computer facilities, standardized PMS elements, standard output reports, and so forth
6	Packaged technical and public presentations	On the technical side, a training course would be produced that combines the basic concepts of a PMS with state-of-the-art pavement rehabilitation techniques; on the public side, the cost-effectiveness of a structured approach to pavement management will be provided

For the first level, a range of typical pavement conditions and treatments will be described for a sample of street segments. Next, the different maintenance techniques will be applied both individually and in logical combinations to the different pavement conditions. For each application, the cost of construction and future maintenance over the extended life of the street segment will be estimated. Extended life will of course not be precise, but by assuming a range of possible lives, the economic effects over this range can be calculated. Through the application of this approach, the relative cost-effectiveness of each maintenance technique and combination of techniques can be illustrated.

For the second level--economic benefits of a structured PMS--a more involved approach will be necessary. First, a small pavement network of 20 to 30 street sections will be described in terms of design, conditions, traffic, age, and so forth. Life prediction models will be developed based on previous research and information from Bay Area experiences. Two different PMS concepts will be analyzed: one will be a nonstructured approach typical in most cities and counties, and the other will be a structured PMS. The nonstructured system will reflect the typical policy of letting pavements fail before performing maintenance work, then overlaying or reconstructing the pavements. The system will have no formal procedures for selecting segments for maintenance treatments. The structured system will have the basic components of a PMS, including a pavement condition rating procedure, a procedure for setting priorities, and a reasonable procedure for selecting maintenance treatments based on cost-effectiveness.

The two approaches will be applied to the network over a period of 40 to 60 years. All of the costs of maintenance will be accumulated, and the condition of the pavement sections will be recorded over the analysis period. A direct comparison of the pavement network costs and overall network condition will be made for each of the systems. The results should graphically show the economic benefits of the structured system, particularly if some measure of user cost increases, caused by allowing pavements to deteriorate below acceptable standards, can be factored in.

The other major focus of the consultant effort will be to develop the three basic elements of a PMS as provided in the previous outline. The objective of this effort is to go beyond the description of the framework necessary for establishing PMSs. The three basic elements will be described at a level of detail sufficient for individual jurisdictions to pursue actual PMS development. In this way actual implementation problems can be experienced, and opportunities for standardization can be tested. The ongoing interest in improving Bay Area PMSs and maintenance practices can continue to be explored.

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A Stable, Consistent, and Transferable Roughness Scale for Worldwide Standardization

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ABSTRACT

Since the AASHO Road Test there has been great interest in the measurement of road roughness for evaluation of serviceability as defined by Carey and Irick, and, perhaps more broadly and importantly, for evaluation of road roughness as it affects vehicle operating costs and road maintenance, particularly in developing countries. In this paper work done in the United States, Brazil, Canada, Bolivia, Nigeria, Panama, and elsewhere with respect to the selection of a uniform method for calibrating road roughness devices is reviewed. Because most roughness measurements are made with response-type roughness measuring instruments, there needs to be a calibration technique for such instruments that can be easily used by any country. It is essential that the method be based on characteristics of the road surface and not on characteristics of any individual vehicle or measuring velocity of the response-type roughness meter. A specific calculation algorithm is also needed. A calibration technique is recommended that is based on a true profile of the roadway surface analyzed with wavebound analysis to determine root-mean-square vertical acceleration for several applicable waveband statistics that are combined to produce the calibration factor. The development of the methodology is presented.

Since the AASHO Road Test, where the concept of pavement serviceability was developed by Carey and Irick (1), increasing importance has been given to user-related pavement evaluation. This type of evaluation is concerned primarily with the overall function of the pavement; that is, how well it serves traffic or the riding public.

The serviceability of a pavement is largely a function of its roughness (2), and several models can be found in the literature to estimate serviceability as a function of roughness alone (3,4). Moreover, it has been demonstrated that roughness is the principal measurement of pavement condition directly related to vehicle operating costs (5,6).

Roughness is normally measured with response-type measuring systems, which are relatively fast and inexpensive; however, the output of these systems is not stable over relatively long periods of time. Consequently, it is necessary to establish a stable roughness scale against which response-type roughness measuring systems can be calibrated.

In this paper a roughness scale is presented that can serve as a universal standard. The scale is stable and consistent and allows transferability over time and space. The roughness scale is derived from the quarter-car index (QI) scale. It was originally defined in the Brazil costs study (7). It was based on simulating a quarter-car's response to a road profile as measured by a Surface Dynamics profilometer (SD or GMR profilometer). The simulation was designed to duplicate the response of the old Bureau of Public Roads roughometer. How to obtain the QI scale from an analysis of a rod-and-level-generated road profile is discussed in this paper.

It is expected that rod and level profile summary statistics put forth in this work can be used to characterize pavement roughness over a wide range of wavelengths in a more reliable manner than other existing profilometer systems.

USE OF PROFILE SUMMARY STATISTICS TO QUANTIFY PAVEMENT ROUGHNESS

The motion of a vehicle on a pavement results from a dynamic system where the vehicle is excited by the vertical displacements of the pavement profile. If the parameters that define the dynamic system as well as the roadway profile are known, vibration theory can be used to determine the vehicle vertical movement at a given speed (8,9).

Most vehicle parameters (tires, suspension, body mounts, seats, and so forth) are relatively similar. Moreover, on any particular road, most cars will be driven at similar speeds. Therefore, the excitations into the car, and thus the riding characteristics, become primarily a function of the road profile (2).

To determine the QI roughness scale from rod and level measurements of pavement profiles, four different summary statistics were tested that the published literature indicated might be useful: (a) wave amplitude, which was originally shown by Williamson et al. (4) to be highly correlated with ratings of riding quality; (b) root-mean-square vertical acceleration, which has been used as a basis for Mays meter calibration (10); (c) mean absolute vertical acceleration, which has been suggested for Mays meter calibration (11); and (d) slope variance, which was found to be highly correlated with serviceability rating at the AASHO Road Test (12).

QI ROUGHNESS SCALE

The Surface Dynamics profilometer used in Brazil and Texas studies consisted of a light delivery vehicle that houses a profile computer, analog tape recorder, quarter-car simulator, a road-following wheel in each wheelpath, potentiometers, and accelerometers. A potentiometer is connected between each road-following wheel and the vehicle body to measure the relative movement between the test wheel and the body (Figure 1). Two accelerometers are secured on the vehicle body directly over the road-following wheels to sense the movement of the body. The potentiometer and accelerometer signals are then electronically combined to remove car body movement and obtain a stable roughness measurement (13).

The profile computer is a special-purpose electronic system that processes the potentiometer signals and the accelerometer signals to obtain the road profile. An analog tape recorder is used to record the profile data so it can be processed after the recording. The quarter-car simulator (QCS) is a special-purpose analog computer that simulates the motion of a single tire mass system over the road profile as it is generated or from the analog tape. The system consists of a body mass, one tire, shock absorber, and springs; the response measured is a summation of the body movement relative to the wheel axle over a fixed distance (Figure 2). The parameter values incorporated into the QCS are for a Bureau of Public Roads (BPR) type roughometer, as reported in the manufacturer's instruction manual.

The roughness output from the QCS, termed the QI, can be accepted as a standard measure of roughness. QI_p has units of deformation per unit length trav-

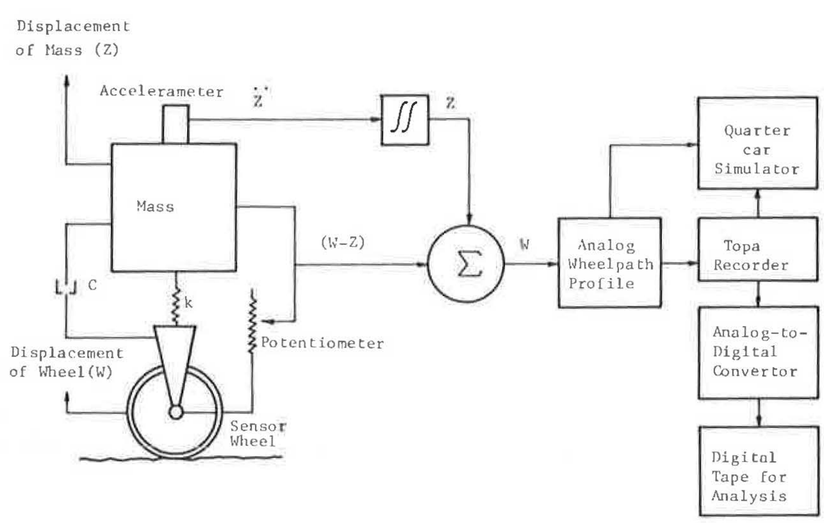

FIGURE 1 Simplified block diagram of the SD profilometer measurement system.

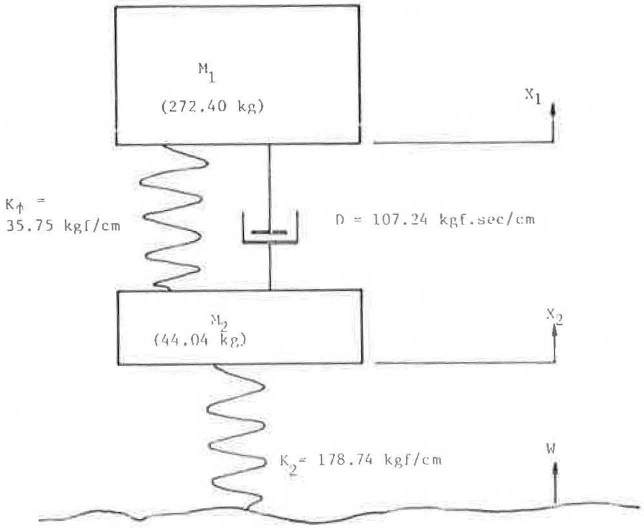

FIGURE 2 QCS schematic.

eled, but to avoid confusion with other roughness measures, the units were designated counts per kilometer. Referring to Figure 2, QI is defined by

$$QI = 1/21 \int |X_1' - X_2'| dL \quad (1)$$

where

X_1 = ordinate of sprung mass ($X_1' = dX_1/dL$),
X_2 = ordinate of unsprung mass ($X_2' = dX_2/dL$), and
L = distance along the road.

Application of Newton's second law to M_1 and M_2 in Figure 2 gives the following set of second-order differential equations:

$$-K_1 (X_1 - X_2) - D(\dot{X}_1 - \dot{X}_2) = M_1 \ddot{X}_1 \quad (2)$$

$$K_1(X_1 - X_2) + D(\dot{X}_1 - \dot{X}_2) - K_2 (X_2 - W) = M_2 \ddot{X}_2 \quad (3)$$

The solution of these equations is required for the evaluation of QI. The electronic circuits in the profilometer QCS were especially designed to give an analog solution to the equations, thus providing the QI. Note that the solution can also be obtained through digital computers, when the pavement profile is known, by using numerical integration. To indicate the range of QI, values of less than 30 counts/km have been observed on new paved roads after construction in Brazil, whereas pavements that require an overlay normally have values greater than 60 counts/km.

ROD AND LEVEL MEASUREMENTS OF PAVEMENT PROFILE

The leveling method is slow and requires considerable care and labor; therefore, this method is not feasible for regular use in measuring long road segments. Thus in this paper the rod and level measurements are examined solely for use in calibrating roughness measuring devices. Use of the leveling method is feasible where an expensive profilometer is not available. The shortest practical distance between successive profile readings or measurement points using rod and level procedures was considered to be 100 mm. The implication of longer intervals between measured points is addressed later. In spite of the continuous nature of a road profile, discrete measurements are not detrimental because the profile must in any event be expressed in discrete terms to be analyzed digitally.

A three-person team consisting of one surveyor and two assistants performed the profile measurements while traffic was continually controlled by flagmen or police. Typically, a maximum of 120 to 130 m of road were surveyed per day on points marked in each wheelpath.

A standard survey level and a rod readable in millimeters were used; elevations were recorded in millimeters. Specially designed code forms were used in the field to minimize transcription errors, and the data were double-checked after input on the computer by using an edit program and by plotting each data point of each profile. Errors are detected and corrected through this procedure. The use of profile plots is particularly appealing because it provides visual identification of errors, so that only reliable data are analyzed.

ROUGHNESS MEASUREMENT SECTIONS

Twenty paved road sections varying from smooth to rough were selected to compare relationships between rod and level measurements of pavement profiles and the Surface Dynamics profilometer. The objective was to correlate QI with some other profile summary statistics so that a convenient standard to calibrate Mays meters (or other response-type roughness measuring devices) could be available in the absence of an SD profilometer. Response-type roughness measuring systems such as the Mays meter must be continually calibrated and checked because their characteristics change as the tires, shock absorbers, and springs on the vehicle wear or as adjustments to the sensors are made.

The sections selected for this study included asphaltic concrete and double surface treatment surfacings. To ensure that profilometer measurements would properly reflect section roughness at the time of the survey, each section was measured with the profilometer a week before, during, and after the measurements with rod and level. From these runs a QI value was established for each wheelpath of each section. The results are given in Table 1.

A total of 3,200 and 6,400 data points was obtained to describe the profile of a short and long section, respectively. Short sections (160 m) were used only when a uniform 320-m section was not found at the required roughness level. In addition, three long sections that had low, medium, and high roughness levels were surveyed twice to provide replicate data for a repeatability study. Thus a total of 131,200 data points was obtained with rod and level for this analysis.

As stated, wave amplitude, root-mean-square vertical acceleration (RMSVA), mean absolute vertical acceleration (MAVA), and slope variance were tested to estimate QI as a universal standard. The mathematical details are presented elsewhere (13,14). Only the details of RMSVA are reproduced here.

USE OF RMSVA TO ESTIMATE QI

RMSVA is a relatively simple profile statistic (10). RMSVA can be defined as the root-mean-square difference between adjacent profile slopes, where each slope is the ratio of elevation change to the corresponding horizontal distance interval selected. This horizontal distance is the base length, and RMSVA can be computed for several base lengths.

TABLE 1 Profilometer Results (QI) on Roughness Correlation Sections

Section	Length (m)	Surface	Profilometer QI		Survey Date
			Right Path	Left Path	
M05	320	AC	62	68	05/79
M06	160	AC	48	40	05/79
M07	160	AC	99	92	05/79
M08	160	AC	68	60	05/79
M09	160	AC	137	105	10/79
M13	320	DST	77	61	10/79
M14	320	DST	62	60	11/79
M15	320	DST	59	74	10/79
M22	320	AC	77	68	08/79
M23	320	AC	27	23	08/79
M26	320	AC	58	57	08 /79
M27	320	AC	48	41	08/79
M28	320	AC	58	53	08/79
M29	320	AC	76	67	10/79
M30	320	AC	87	67	10/79
M31	320	AC	66	70	10/79
M32	320	AC	37	36	08/79
M38	160	AC	105	97	11/79
A16	320	DST	62	94	06/80
A17	320	DST	72	76	06/80
M23(R)	320	AC	22	23	03/80
M28(R)	320	AC	57	55	03/80
M30(R)	320	AC	86	68	03/80

AC = asphaltic concrete
DST = double surface treatment
R = replication

RMSVA is obtained from elevations $Y_1, Y_2 \ldots, Y_N$ of equally spaced points along one wheelpath by

$$VAb = \left[\sum_{i=k+1}^{N-k} (SB_i)^2/(N-2k)\right] \quad (4)$$

where

VAb = RMSVA corresponding to base length b,
b = ks (i.e., the base length),
k = arbitrary integer used to define b as a multiple of s,
s = sampling interval (i.e., the horizontal distance between adjacent points), and
SB_i = an estimate of the second derivative of Y at point i given by

$$SB_i = \left\{[(Y_{i+k} - Y_i)/ks] - [(Y_i - Y_{i-k})/ks]\right\}/ks$$

or

$$SB_i = (Y_{i+k} - 2Y_i + Y_{i-k})/(ks)^2$$

A simple computer program was developed to perform RMSVA computations (14). The least-squares method and ridge analysis were used to develop a model to predict profilometer QI from rod and level profile RMSVA. The following equation was found to best fit the data:

$$QIrmsva = -8.54 + 6.17\ VA10 + 19.38\ VA25 \quad (5)$$

R squared = 0.95, standard error = 5.65, and CI = QIrmsva ± 11.68

where

QIrmsva = QI estimate from RMSVA;
VA10, VA25 = RMSVA corresponding to base lengths of 10 and 25 decimeters, respectively (mm/m^2); and
CI = approximate 95 percent confidence interval.

A visual presentation of how well Equation 5 predicts QI is shown in Figure 3, where profilometer QI is plotted against QIrmsva.

COMPARISON BETWEEN ROD AND LEVEL ANALYSIS PROCEDURES

The data in the previous sections demonstrate that it is possible to compute summary statistics from rod and level profiles that correlate well with the SD profilometer QI. This is true in varying degrees for the statistics used to summarize rod and level profile data, namely (a) wave amplitude, (b) RMSVA, (c) MAVA, and (d) slope variance. The best predictor was RMSVA.

From consideration of standard error for residuals, multiple correlation coefficient, and stability of regression coefficients, it can be concluded that wave amplitude, RMSVA, and MAVA predict QI to about the same degree of accuracy and represent a better estimate than slope variance. From a computational point of view, the vertical acceleration procedures (i.e., RMSVA and MAVA) are preferable to wave amplitude, whose computation is more detailed. Because RMSVA predicts QI slightly better than MAVA, it appears reasonable to recommend the use of Equa-

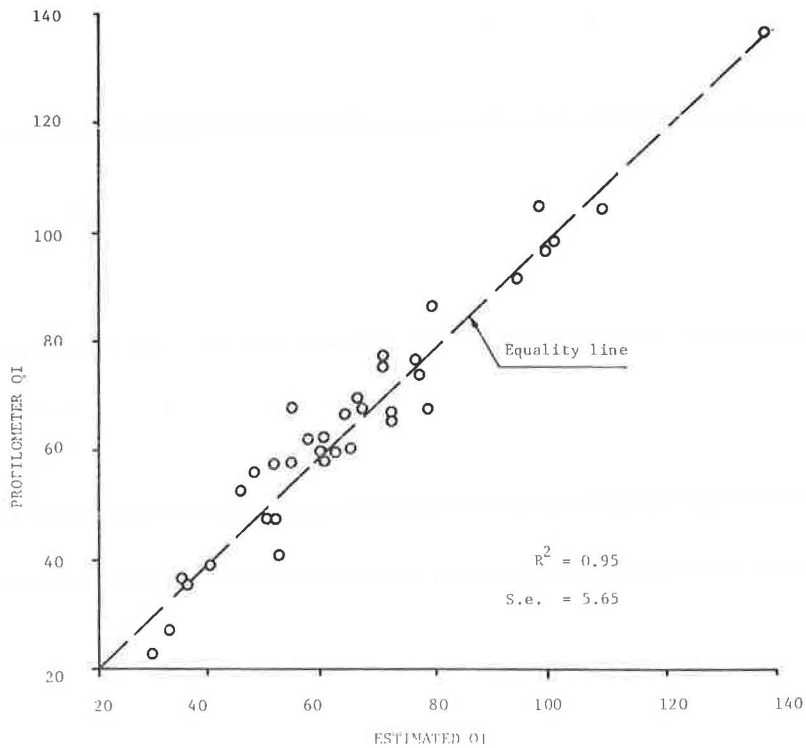

FIGURE 3 Relationship between SD profilometer QI and QI estimated from RMSVA.

tion 5 for estimating QI from rod and level measurements of pavement profile. For further applications, the QI estimate from RMSVA (i.e., QIrmsva) will be represented simply QIR.

REPEATABILITY OF ROD AND LEVEL ROUGHNESS MEASUREMENTS

The repeatability of an instrument refers to the degree to which the repeated measurements made with the instrument agree with each other (15). When the profile of a test section is measured twice, the results are not expected to be exactly the same because of variations in the exact profile line surveyed and random measurement error.

Three test sections with low, medium, and high roughness were selected for studying the repeatability of rod and level roughness measurements. The measurements on these sections were replicated about 6 months after the initial measurements.

The Walsh test was used to compare the means of the QI statistics obtained from the replicate pavement profile surveys. This nonparametric test was selected because of its power and usefulness for small samples (16). The results indicated that the rod and level measurements of pavement roughness in both surveys were not significantly different at the 10 percent confidence level. Therefore, the data analyzed indicate that the rod and level procedure has good repeatability.

USE OF QI FOR CALIBRATING ROUGHNESS MEASURING SYSTEMS

Roughness measuring systems such as the Mays meter, bump integrator, and roughometer have in common the fact that their roughness output for the same road section can vary with time as changes in machine condition (e.g., tires, springs, shock absorbers, mass) occur. Roughness measuring instruments of this type are classified as a response-type road roughness measure system (RTRRMS) in contrast to systems that measure the longitudinal profile characteristic directly (17). Rod and level measurements of pavement profile fall in the second category.

In general, RTRRMSs have the advantage of relatively low cost, simple operation, and high measuring speed. However, because of their susceptibility to changes, RTRRMSs require periodic calibration against a stable measuring system to provide consistent and useful measures of pavement roughness.

The kind of calibration problem of concern here can be described as follows (18): there are two related quantities X and Y, such that X is relatively easy to measure and Y is relatively difficult and requires more effort or expense; furthermore, the error in a measurement of Y is negligible compared with that for X. In this context X can be interpreted as an RTRRMS output and Y is some pavement profile summary statistic obtained, for example, from rod and level measurements. The problem consists of estimating unknown values of Y, corresponding to measurements of X, through a calibration equation established from simultaneous X and Y measurements on a number of sections [i.e., the calibration equation is of the form $Y = f(X)$].

From the foregoing discussion it can be concluded that a roughness measure Y, to be useful as a roughness standard, has to be repeatable and highly correlated with the roughness outputs from the devices whose calibration is desired. The good correlation between the rod and level summary statistic QI and several roughness measuring devices will be discussed later. Rod and level repeatability was noted to be good; therefore, QI obtained from rod and level measurements of pavement profile represents an acceptable means to calibrate response-type roughness measure systems.

For calibrating RTRRMSs against rod and level summary statistics (e.g., QIR), the same method developed by Walker and Hudson (19), which uses the SD profilometer as standard, is recommended. The method requires that about 20 paved sections covering the roughness range of interest be selected. Test section length should be a multiple of the roughness device output intervals and, preferably, on the order of 300 m or longer. Depending on the pavement structure and traffic loads on these calibration sections, rod and level measurements of both wheelpaths should be conducted about twice a year or even at shorter time intervals if seasonal effects are suspected to be a significant factor in ride quality.

In summary, the calibration procedure recommended for use remains the same whether rod and level or the SD profilometer is used as the standard. The roughness device to be calibrated is operated over a number of test sections whose wheelpath profiles have been measured with rod and level; the output from the roughness device for each section is then correlated against the profile summary statistic QI. Thus a calibration equation is obtained that permits the pavement roughness, in terms of QIR, to be estimated from measurements with the other roughness device.

ANALYSIS OF SAMPLING RATE EFFECT ON ACCURACY OF QI ESTIMATES

As stated previously, a 100-mm sampling interval was chosen for the rod and level measurements of pavement profile in this study because it represents the minimum interval feasible to be implemented in the field. Subsequently, it was demonstrated that rod and level summary statistics obtained with this sampling interval constitute an accurate means to estimate QI. In this section the possibility of adopting longer sampling intervals, which would expedite not only the field work but also data processing, is examined.

By eliminating intermediate data points, different sampling intervals were simulated for this analysis. A maximum sampling interval of 500 mm was selected because it is necessary for computing VA10 and VA25, which are independent variables in Equation 5.

Differences between mean QI obtained from the 500-mm sampling interval and the basic QI (i.e., at 100-mm intervals) were analyzed by a test for correlated samples (20). The results indicate that the hypothesis of equal QI means from the two sampling intervals used cannot be rejected at the 10 percent level of significance. The good agreement obtained between QI values calculated from 100- and 500-mm sampling intervals is shown in Figure 4. Therefore, a sampling interval of 500 mm is recommended for use in future applications.

An investigation of the influence of sampling interval on the QIwa and QImava indices was also carried out. The wave amplitudes computed from a 200-mm sampling interval are significantly different from the ones obtained when the original 100 mm is used. Therefore, it is considered that only 100-mm sampling intervals or less can yield accurate wave amplitude values and, consequently, accurate QI estimates when this approach is used. The influence of sampling intervals on MAVA was found to be similar to the influence on RMSVA.

ADEQUACY OF QI SUMMARY STATISTIC OF ROADWAY ROUGHNESS

Several statistics have been proposed to summarize measurements of roadway roughness as reviewed by Gillespie et al. (17). In this section the suitability of QI as one of these statistics is examined.

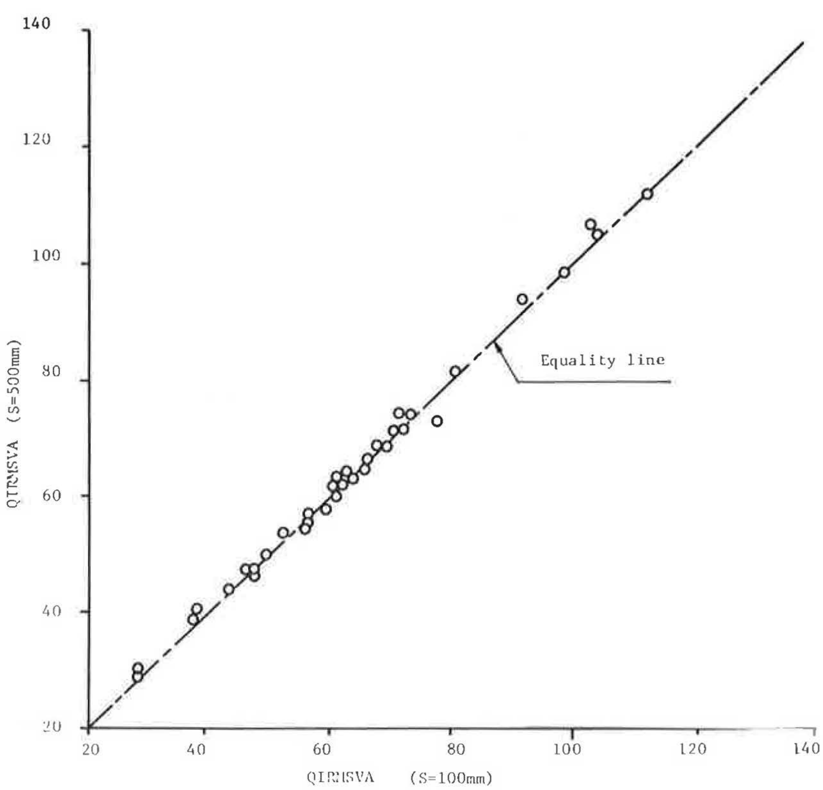

FIGURE 4 Comparison between QI values obtained from 100- and 500-m sampling intervals.

It has been demonstrated in the Brazil study (21) that QI is an extremely useful measure of roadway roughness because it is one of the most significant independent variables in the equations developed to predict road user costs. Bump integrator measurements of pavement roughness, which are highly correlated with QI, were also demonstrated to be an important predictor of vehicle operating costs in the Kenya study (5). Therefore, insofar as road user costs are concerned, QI can be considered as a good summary statistic of roadway roughness.

It has been stated that a good roughness index should correlate well with human panel ratings of riding quality (4). The evaluation of 40 test sections, selected on the paved and unpaved highway network in the vicinity of Brasilia by a panel of 52 raters, yielded the following correlation equation (22):

$$SI = 4.66 \exp(-0.00534QI) \quad (6)$$

R squared = 0.83

where SI is the present serviceability index (i.e., an estimate of the mean panel rating) and QI is the quarter-car index (counts/km).

This equation indicates that QI correlates well with serviceability rating. Because QI also is an important explanatory variable in road user cost prediction equations, it appears reasonable to recommend QI as a roadway roughness summary statistic for general use. Furthermore, studies of road deterioration in Brazil have provided equations to predict roughness using QI units, for both paved (14) and unpaved (23) roads, as a function of variables such as material characteristics, traffic loads, and volumes. These relationships, together with road user cost equations, provide an essential tool for the economic analysis of highway investments.

CORRELATION BETWEEN QI AND ROUGHNESS MEASURING SYSTEMS

An International Road Roughness Experiment (IRRE) conducted in Brazil in May and June 1982 examined the correlations between QI (and other roughness scales) and different road roughness measurement equipment in use throughout the world (24). A total of 49 sections, each 320 m long, were evaluated for roughness on a wide range of paved and unpaved roads. The roughness was measured at a number of speeds by seven RTRRMSs, including three Mays meter systems, a car-mounted bump-integrator unit from the Transport and Road Research Laboratory (TRRL), a National Association of Australian State Road Authorities (NAASRA) roughness meter from the Australian Road Research Board (ARRB), a TRRL bump-integrator trailer, and a BPR-type roughometer from the Federal University of Rio de Janeiro.

A summary of the correlations between QI and the RTRRMSs included in the IRRE is given in Table 2 for all road surface types studied (i.e., asphalt concrete, surface treatment, gravel, and earth). The overall correlation is good, but the highest correlation coefficients are obtained when the RTRRMSs run at 50 km/h. Therefore, it is recommended that this speed be used for calibrating an RTRRMS against the QI scale. If other speeds are selected for roughness measurements (e.g., 32 or 80 km/h), regression equations should be developed to convert RTRRMS readings at these speeds to the readings that would be obtained at 50 km/h.

TABLE 2 Summary of Correlations Between QI and RTRRMS (R-Squared Values) from the IRRE

RTRRMS	RTRRMS SPEED (KM/H)		
	32	50	80
MM01	.89	.95	.72
MM02	.94	.94	.77
MM03	.88	.91	.67
BI-CAR	.92	.92	.84
NAASRA	.93	.94	.92
BI-TRL	.92	.94	----
BPR	.85	----	----

CORRELATION BETWEEN QI AND RARV

A roughness scale--reference average rectified velocity (RARV)--was defined as part of an NCHRP project (17). The RARV roughness scale depends on the simulated speed and sampling interval used to measure the road profile. In the IRRE, a 500-mm sampling interval was used to measure the road profiles with rod and level; therefore, RARV based on this sampling interval was obtained for all of the 49 road sections studied. Figure 5 shows the relationship between QI and RARV for a simulated speed of 50 km/h. Similar scatters were obtained for simulated speeds of 32 and 80 km/h, where the computed R-squared values were 0.92 and 0.97, respectively. The good correlation between QI and RARV has two simple explanations. First, both QI and RARV originated from a linear simulation of a quarter of a car; their values, however, are not the same because the model parameters used in the simulation (e.g., spring constants, sprung mass, and nonsprung mass) are different. Second, RARV values obtained from different simulated speeds are intercorrelated.

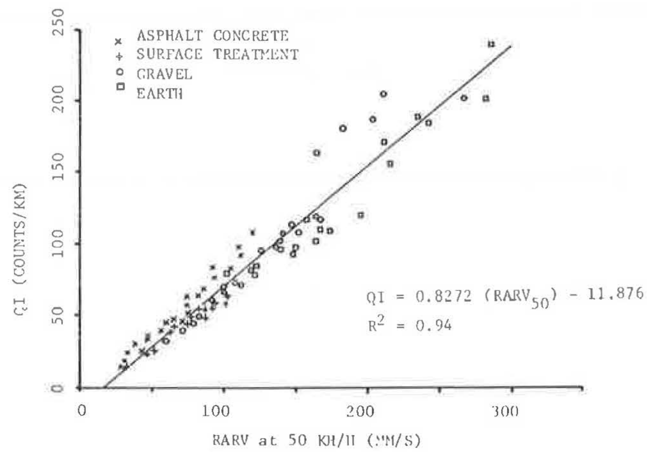

FIGURE 5 Relationship between QI and RARV at 50 km/h.

For practical purposes, QI and RARV are interchangeable when road sections of the same length are used; however, some recent work by Visser indicates that the RARV computation may present problems associated with variable section length. Because QI is easier to compute and the QI scale has been implemented in various countries (e.g., Brazil, Bolivia, South Africa, Nigeria, Panama, and at least one state in the United States), its use is recommended for worldwide standardization.

INTERNATIONAL VALIDATION OF THE QI ROUGHNESS SCALE

Two important research projects have contributed to the international validation of the QI scale: (a) the IRRE conducted in Brazil in May and June 1982, which has been discussed earlier in this paper, and (b) a correlation study of roughness measurements with QI carried out in South Africa (25). A brief summary of the South African experiment is presented here.

The aims of the study were to extend and evaluate the results from Brazil to a wider range of road sections and measuring instruments. The specific objectives were to

1. Run a correlation experiment so that the estimated QI values on paved and unpaved sections could be related to the outputs obtained with the different roughness measuring instruments used in South Africa,
2. Check the repeatability of the estimated QI values obtained from measurements at different times on a rough and smooth section, and
3. Evaluate the influence of distance between adjacent measured points of the profile on the estimated QI of gravel roads.

Several response-type roughness measuring devices are in use in South Africa and were used in the correlation experiment. These include (25) a modified Portland Cement Association (PCA) meter, a linear displacement integrator (LDI), a BPR roughometer, and a photologger roughness output.

In the original study a 100-mm spacing between adjacent rod and level elevation points was used, but it was found that on paved roads the spacing could be increased to 500 mm without affecting the roughness statistic (13). For the South African study, the 500-mm spacing was used on the paved roads, whereas a 100-mm spacing was used on unpaved roads because of uncertainties about the influence of corrugations with a 1-m period on the summary statistic.

A trained team of one surveyor, one assistant for noting the readings, and one assistant skilled in using the staff could complete a 200-m paved test section in a day. This includes traveling to the site, marking out the section, and placing traffic control devices. Normally, two assistants were employed for traffic control, except on roads carrying heavy traffic where the aid of traffic police was required. A section length of 200 m was selected because the roughness instruments measure in multiples of 100 m and because difficulty was encountered in finding a longer homogeneous length of road, especially in the rougher range.

An automatic self-leveling instrument with a vernier attachment was used for most of the measurements. The rod appropriate to this instrument is called a half-decimeter rod (i.e., a half meter is divided into 100 divisions). Therefore, in conjunction with the vernier, the precision is 0.05 mm. This precision is unnecessary for the present purpose, and results were only recorded to 0.5-mm accuracy. Specially designed code forms were used in the field to minimize transcription errors. After keypunching, the data were checked by an editing routine, and the remaining errors became obvious from plotting the profile. On the unpaved sections the same procedure as for the paved sections was used, except that elevations were measured every 100 mm instead of every 500 mm. A rough and a smooth paved section were also measured by using a standard level with a split-bubble, and a staff graduated in centimeters.

The repeatability of the rod and level procedure was checked on two paved sections, one smooth (section 25) and one rough (section 26). Note that the section numbers relate to sections on the standard calibration route. The measurements were first made in the beginning of October 1981 and repeated at the end of November 1981. This time span was long enough to ensure that no marks from the first measurement were visible, but it was also short enough to prevent any major changes in roughness on the sections. Results of the two measuring sessions are given in Table 3. The differences between the means of the two sessions are 0.7 and 1.0 for sections 25 and 26, respectively. These differences are not meaningful.

TABLE 3 Repeatability of Rod and Level Measurements in South Africa

Date	QI_R outer wheel	QI_R inner wheel	QI_R mean
Section 25			
2 Oct 1981	19.1	18.3	18.7
27 Nov 1981	21.3	17.6	19.4
24 Nov 1981 (Cm rod)	21.5	18.7	20.1
Section 26			
14 Oct 1981	67.6	75.6	71.6
30 Nov 1981	68.3	77.0	72.6
25 Nov 1981 (Cm rod)	69.7	76.1	72.9

In Brazil a standard survey level and a staff graduated in centimeters were used. To test the influence of the survey instrument, two different level instruments and staffs were used in this comparison. One instrument was an automatic self-leveling instrument with vernier attachment and a half-decimeter rod, and the other instrument was an ordinary surveyor's level with split-bubble and an ordinary staff graduated in centimeters. Sections 25 and 26 were again measured; the results are also given in Table 3. For the ordinary level the computed QI values are slightly higher than for the automatic level, but the difference is less than one unit of QI when compared with the mean of the values

TABLE 4 Computed QI Values on Unpaved Road Profiles Measured at 100- and 500-m Intervals

Section	Interval (mm)	QI outer wheel	QI inner wheel	QI mean
G1	100	67.5	152.9	110.2
	500	69.0	157.4	113.2
		68.9	156.1	112.5
		65.9	155.7	110.8
		68.5	148.0	108.2
		65.0	147.3	106.2
G2	100	131.7	135.2	133.4
	500	131.8	128.4	130.1
		135.5	131.4	133.5
		129.7	142.6	136.1
		130.3	138.0	134.1
		130.7	134.1	132.4

TABLE 5 Statistics Related to the Linear Regressions Between QI and Roughness Outputs of Different Instruments in the South African Study

Dependent Variable	Independent Variable	R-squared	Standard error of residuals	Sample size	Intercept	Slope	
						Cooefficient	t-value
QI	LDI	0.98	3.40	18	-4.60	22.46	27.1
QI	Photologger	0.96	4.45	18	6.74	0.1898	20.5
QI	ln PSI	0.97	3.78	18	92.63	-56.39	11.9
QI outer wheelpath	BPR	0.90	7.73	18	-16.98	0.6866	-24.3

obtained with the automatic level. The centimeter rod, which was less precise than the half-decimeter rod, would yield rounding errors, and this is reflected by the slightly higher QI value. However, the difference is not meaningful, and any accurate surveyor's level could be used in generating QI measurements.

Unpaved roads normally exhibit corrugations or deformations that have a greater amplitude than those found on paved roads, and concern existed about the QI generated from profile measurements taken at 500-mm intervals. For this reason measurements were taken at 100-mm intervals on the two unpaved sections, both of which exhibited corrugations. The data collected permitted an evaluation of whether the 500-mm spacing of readings has a meaningful influence on the result. The data in Table 4 give the QIs completed and the variations are not significant.

The statistics related to the correlations between QIR and the PCA roadmeter, LDI, photologger, and BPR roughometer are given in Table 5. From these statistics it can be noted that, based on the standard error of estimate and the R-squared, the decreasing order of best correlation with QI is the LDI, PCA roadmeter, photologger, and BPR roughometer. In fact, the correlation with the BPR roughometer is considerably poorer than for the other instruments. The relatively poor performance of the BPR roughometer is attributed to its advanced age and poor condition. The correlation between QI and the LDI, which is similar in characteristics to the Mays meter, is similar to the values obtained in Brazil.

OTHER VALIDATIONS

The rod-and-level-based QI scale calibration procedure has been successfully used to control roughness measurements in a number of countries. For the Ministry of Transport in Panama, Hudson et al. (26) established a roughness measuring capability to assist in determining priorities for pavement rehabilitation and maintenance for Panama's highways. The National Highway Service of Bolivia first used a TRRL pipe course and then, under Butler's direction, replaced it with the rod-and-level-based QI calibration procedures to control roughness measurement taken with two Mays meters (27). Bolivia maintains a network-wide roughness inventory on its paved roads and has studied maintenance service levels for aggregate road grading frequencies based on roughness. Hudson is also using the rod-and-level-based QI calibration procedure in Nigeria for inventorying roads and establishing a country-wide pavement evaluation and management system.

SUMMARY AND CONCLUSIONS

It has been demonstrated in this paper that rod and level measurements of pavement profile, using short sampling intervals, represent a feasible and accurate means for establishing a stable roughness scale (QI). Estimates of QI were developed from four different profile summary statistics found in the literature: wave amplitude, RMSVA, MAVA, and slope variance. From a computational point of view, the vertical acceleration procedures (RMSVA and MAVA) are superior.

When a 500-mm sampling interval is used to collect pavement profile data with rod and level, QI can be estimated more precisely from RMSVA than from MAVA; therefore, Equation 5 using RMSVA is recommended for obtaining QI.

The rod and level QI scale is particularly appealing for developing countries, where the costs of such procedures may be significantly less than the costs of other procedures, depending on sophisticated imported profilometers.

A number of alternatives for transferring a roughness standard from one region to another have been presented in the technical literature, including the rod and level survey method. Taking into account the inherent limitations of some of these alternatives and the analysis conducted in this study, and considering simplicity, reliability, and costs as important factors, it is reasonable to conclude that, with the current state of the art, the QI scale is a suitable worldwide roughness standard, and its adoption is recommended.

ACKNOWLEDGMENTS

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Serviceability and Distress Methodology for Predicting Pavement Performance

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ABSTRACT

In this paper the fundamental aspects in the development and application of a methodology for predicting pavement performance are summarized in terms of three indices: (a) present serviceability index, (b) distress area index, and (c) distress severity index. A statistical procedure used for estimating the parameters of the performance relationships guarantees that the goodness-of-fit between predicted and observed data is maximized. The most salient feature of the proposed methodology is the use of an S-shaped curve that recognizes a change in the rate of deterioration of a pavement as the traffic level accumulates until rehabilitation is needed. Serviceability ratings, based on data obtained from 164 pavement test sections, are used to predict the performance of black-base, hot-mix, and overlay pavements. In addition, the proposed method can be used when pavement performance is ascertained in terms of area and severity distress ratings for several types of pavement distress such as rutting, flushing, raveling, alligator cracking, transverse cracking, longitudinal cracking, and patching.

The purpose of this paper is to summarize recent developments and actual applications of a pavement performance equation that predicts the loss of serviceability or deterioration caused by various types of distress. The proposed model represents an improvement over the original AASHO Road Test performance equation in that it predicts more realistic long-term behavior. This is achieved through the use of a sigmoidal or S-shaped curve that recognizes the ability of a pavement to reduce its rate of deterioration as the traffic level approaches the end of the service life of the pavement. This behavior, for example, is typical of pavements that have received adequate routine maintenance in the past. To evaluate the parameters in the performance model, a least-squares curve fit technique is employed using field measurements from the data base for flexible pavements available at the Texas Transportation Institute (1). The types of pavements considered along with the number of test sections evaluated are as follows: black base, 51 sections; hot-mix asphaltic concrete, 36 sections; and overlays, 77 sections.

The data for each test section consisted of values of the present serviceability index as a function of the number of 18-kip equivalent axle loads. In addition, the structural performance of the pavement was evaluated in terms of distress severity and area for the following distress types (in each case the primary variable correlated with the distress type is shown in parentheses): rutting (N-18), alligator cracking (N-18), patching (N-18), flushing (ADT), raveling (ADT), longitudinal cracks (time), and transverse cracks (time). For the primary variables, N-18 and ADT represent the number of 18-kip single-axle loads and the average daily traffic, respectively; in addition, time represents the number of months since initial construction.

The paper is divided as follows. First background information that pertains to the development of the AASHO highway performance equation is presented. Second, the development and characteristics of the proposed sigmoidal or S-shaped curve are described. Third, the procedure for determining the design constants for the curve, using present serviceability index data, is presented. Fourth, the prediction of pavement distress using the proposed methodology is discussed. Finally, an actual application of the method to predict the functional and structural performance of Texas pavements is presented.

GENERAL BACKGROUND ON PERFORMANCE EQUATIONS

Types of Performance

In the 20 years since the AASHO Road Test began, the idea of performance has been accepted and broadened to accord with the measures of service that the pavement provides. Because of this, it is now possible to define roughly three types of performance: functional, structural, and survival (2).

1. Functional performance: This is the measure that was adopted by the AASHO Road Test; that is, the present serviceability index, which measures the quality of riding conditions from the point of view of the traveling public.
2. Structural performance: The deterioration of structural performance is measured by the appearance of various forms of distress and their relative importance in triggering decisions to maintain or to rehabilitate a pavement. These measures include roughness, cracking (several types), rutting, flushing, raveling, failures (potholes), and patches in flexible pavements. The measures for rigid pavements include spalling, cracking, and joint problems such as pumping, failures, and faulting. Because structural performance is visible or measurable, whereas functional performance is primarily subjective, there have been several attempts to relate the two.
3. Survival: The survival of a pavement is determined by the amount of time that it lasts before major maintenance or rehabilitation must be performed. Survival is measured by the probability that a given pavement is still in service a number of years after its construction. Historical records may be used to develop such survivor curves, which are important in projecting budget levels for maintenance and rehabilitation work.

Each of these kinds of performance has its own use in serving the public. The latter two are of principal importance to the agency that is responsible for keeping a roadway network in good operating condition.

The form of the AASHO equation is

$$g = (W/\rho)^{\beta} \quad (1)$$

where

- g = damage function, which is a normalized variable that ranges from 0 to 1 as distress increases or as functional performance or survival probability decreases;
- W = quantity of normalized load or climatic cycles, or the total elapsed time to reach a given level of g;
- ρ = quantity of normalized load or climatic cycles, or the total elapsed time until g reaches a value of 1 (it is usually assumed to be a function of the structural variables); and
- β = a power that dictates the degree of curvature of the curve relating g to the ratio of W/ρ; a high value of β (greater than 1) indicates that g remains low over the majority of the life of the pavement, whereas a low value of β (less than 1) indicates a high value of g over the life of the pavement.

The damage function in the AASHO design equation is defined as a serviceability index ratio:

$$g = (P_o - P)/(P_o - P_f) \quad (2)$$

where P_o is the initial serviceability index, and P_f is the minimum serviceability index (in the AASHO design equation this value is equal to 1.5). Combining Equations 1 and 2, the AASHO design equation can be rewritten as

$$P = P_o - (P_o - P_f)(W/\rho)^{\beta} \quad (3)$$

A graphical representation of Equation 3 is shown in Figure 1.

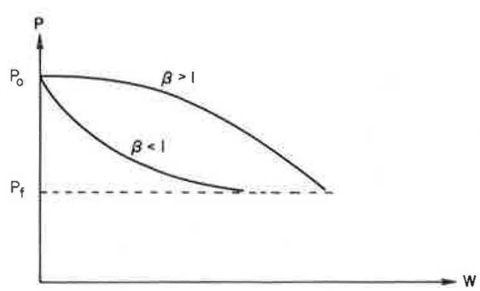

FIGURE 1 AASHO performance curve.

Alternative Forms of Functional Performance Equations

The shape that a functional performance curve should take can be deduced from the boundary conditions placed on the serviceability index scale as well as from long-term observations of field data. The serviceability rating scale ranges between 0 and 5 and, as it is defined, can be neither greater than 5 nor less than 0. As pavement roughness increases, the serviceability rating will decrease and will approach, but not drop below, a serviceability rating of 0 no matter how much traffic passes over the pavement. Thus the performance curve starts out horizontally bounded from above by a rating of 5. As load repetitions increase, the curve is bounded from below by a rating of 0, a value that it approaches as an asymptote. These boundary conditions imply that a functional performance curve should be S-shaped.

The form of the AASHO design equation (Equation 1) assumed that the serviceability-index-versus-traffic curve never reverses its curvature, as shown in Figure 1. By way of contrast to this assumed form of equation, a number of observed serviceability-index-versus-traffic relations have shown a reversal of curvature (see Figure 2).

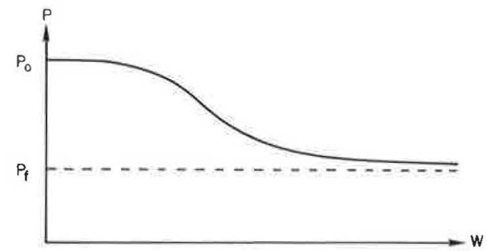

FIGURE 2 S-shaped performance curve.

The S-shaped feature of the curve shown in Figure 2 requires an equation of the form

$$(P_o - P)/(P_o - P_f) = \exp[-(\rho/W)^{\beta}] \quad (4)$$

which can be rewritten as

$$P = P_o - (P_o - P_f)\exp[-(\rho/W)^{\beta}] \quad (5)$$

In this paper the S-shaped performance function defined in Equation 5 has been considered. Obviously, there are many choices for this function; the following list of considerations is helpful in deciding what particular choice to use.

1. The function must have a maximum (minimum) value at traffic level (or time) equal to zero and must be strictly decreasing (increasing) as the traffic level increases.
2. The function cannot have negative values; indeed, if the performance value is standardized to be between 0 and 1, the particular choice of the function cannot have values outside this range as traffic or time increases.
3. The function must have at least one parameter so that a family of pavements may be represented for different values of the parameter or combinations of parameter values in the case of several parameters.
4. The structure of the performance function must be suitable for an efficient estimation procedure of the parameters on the basis of observed data.

It is easy to verify that all of these conditions are satisfied by Equation 5. This equation has been investigated and validated by using an extensive data base for flexible pavements available at the Texas Transportation Institute. Previous studies have also demonstrated the validity of Equation 5 in predicting pavement performance (1,3-6).

PROCEDURE FOR DETERMINING DESIGN CONSTANTS

Assuming that P_o is known, the purpose of this section is to develop a statistical procedure to determine the constants P_f, ρ, and β on the basis of observed performance data for a given type of pavement.

The performance relationship (Equation 5) can be expressed as

$$P_o - P = \alpha \exp[-(\rho/W)^{\beta}] \quad (6)$$

where

$$\alpha = P_o - P_f \quad (6a)$$

Taking the natural logarithm of Equation 6 yields

$$\mathrm{Ln}(P_o - P) = \mathrm{Ln}(\alpha) - (\rho/W)^\beta \quad (7)$$

which can also be written as

$$\mathrm{Ln}(P_o - P) = \mathrm{Ln}(\alpha) - \rho^\beta(1/W)^\beta \quad (8)$$

Using the transformation $e^\tau = 1/W$, Equation 8 becomes

$$\mathrm{Ln}(P_o - P) = \mathrm{Ln}(\alpha) - \rho^\beta(e^\beta)^\tau \quad (9)$$

which is equivalent to

$$z = a - bc^\tau \quad (10a)$$

where the variables of substitution are

$$z = \mathrm{Ln}(P_o - P) \quad (10b)$$

$$a = \mathrm{Ln}(\alpha) \quad (10c)$$

$$b = \rho^\beta \quad (10d)$$

$$c = e^\beta \quad (10e)$$

Given a collection of m data points (P_i, W_i), where P_i is the serviceability index corresponding to a traffic level W_i, and i = 1, 2, ..., m, the remaining portion of this section deals with the development of a statistical procedure to find a, b, and c on the basis of observed data.

Specifically, the data can be computed as follows:

1. Find $z_i = \mathrm{Ln}(P_o - P_i)$ for i = 1, 2, ..., m, and
2. Find $\tau_i = \mathrm{Ln}(1/W_i)$ for i = 1, 2, ..., m.

Therefore, the observed values of P_i and W_i are transformed into values of z_i and τ_i, respectively. The statistical model to be used is defined as

$$z_i = a - bc^{\tau_i} + \epsilon_i \quad (11)$$

where ϵ_i is the random error corresponding to the value z_i associated with τ_i.

The basic procedure to estimate the parameters a, b, and c is the well-known least-squares method. This method computes a, b, and c in such a way that the quantity $\sum_{i=1}^{m} \epsilon_i^2$ is minimized. This quantity can be obtained from Equation 11 as

$$\sum_{i=1}^{m} \epsilon_i^2 = \sum_{i=1}^{m} (z_i - a + bc^{\tau_i})^2 \quad (12)$$

The necessary (and in this case sufficient) conditions for a minimum are given by

$$\partial\left(\sum_{i=1}^{m} \epsilon_i^2\right)/\partial a = 0 \quad (13a)$$

$$\partial\left(\sum_{i=1}^{m} \epsilon_i^2\right)/\partial b = 0 \quad (13b)$$

$$\partial\left(\sum_{i=1}^{m} \epsilon_i^2\right)/\partial c = 0 \quad (13c)$$

These conditions can be shown to be equivalent to

$$\sum_{i=1}^{m} (z_i - a + bc^{\tau_i}) = 0 \quad (14)$$

$$\sum_{i=1}^{m} (z_i - a + bc^{\tau_i})c^{\tau_i} = 0 \quad (15)$$

$$\sum_{i=1}^{m} (z_i - a + bc^{\tau_i})\tau_i c^{\tau_i - 1} = 0 \quad (16)$$

It is noted that Equations 14 and 15 are linear in a and b; therefore both parameters can be obtained in terms of z_i, τ_i, and c. The corresponding results are as follows:

$$a = \left[\left(\sum_{i=1}^{m} C^{2\tau_i}\right)\left(\sum_{i=1}^{m} Z_i\right) - \left(\sum_{i=1}^{m} C^{\tau_i}\right)\left(\sum_{i=1}^{m} Z_i C^{\tau_i}\right)\right] \div \left[m \cdot \left(\sum_{i=1}^{m} C^{2\tau_i}\right) - \left(\sum_{i=1}^{m} C^{\tau_i}\right)\left(\sum_{i=1}^{m} C^{\tau_i}\right)\right] \quad (17)$$

$$b = \left[-m \cdot \left(\sum_{i=1}^{m} Z_i C^{\tau_i}\right) + \left(\sum_{i=1}^{m} C^{\tau_i}\right)\left(\sum_{i=1}^{m} Z_i\right)\right] \div \left[m \cdot \left(\sum_{i=1}^{m} C^{2\tau_i}\right) - \left(\sum_{i=1}^{m} C^{\tau_i}\right)\left(\sum_{j=1}^{m} C^{\tau_i}\right)\right] \quad (18)$$

The values of a and b given by Equations 17 and 18 can be substituted into Equation 16 to obtain the following final result:

$$\sum_{i=1}^{m} Z_i \tau_i C^{\tau_i} - \left\{\left[\left(\sum_{i=1}^{m} C^{2\tau_i}\right)\left(\sum_{i=1}^{m} Z_i\right) - \left(\sum_{i=1}^{m} C^{\tau_i}\right)\left(\sum_{i=1}^{m} Z_i C^{\tau_i}\right)\right] \div \left[m \cdot \left(\sum_{i=1}^{m} C^{2\tau_i}\right) - \left(\sum_{i=1}^{m} C^{\tau_i}\right)\left(\sum_{i=1}^{m} C^{\tau_i}\right)\right]\right\} \cdot \left(\sum_{i=1}^{m} \tau_i C^{\tau_i}\right)$$
$$+ \left\{\left[-m \cdot \left(\sum_{i=1}^{m} Z_i C^{\tau_i}\right) + \left(\sum_{i=1}^{m} C^{\tau_i}\right)\left(\sum_{i=1}^{m} Z_i\right)\right] \div \left[m \cdot \left(\sum_{i=1}^{m} C^{2\tau_i}\right) - \left(\sum_{j=1}^{m} C^{\tau_i}\right)\left(\sum_{i=1}^{m} C^{\tau_i}\right)\right]\right\} \times \left(\sum_{i=1}^{m} \tau_i C^{2\tau_i}\right) = 0 \quad (19)$$

Equation 19 can be solved for c by using a trial-and-error method or a simple numerical analysis method. Once c is determined, a and b can be computed from Equations 17 and 18, respectively, and their corresponding values can be used to estimate α (and thus P_f), ρ, and β from Equations 10c, 10d, and 10e.

In the case of the method discussed in this paper, the observed data P_i and W_i are transformed into z_i and τ_i, respectively. It is noted that both parameters a and b are linear functions of the transformed data for a fixed choice of c. Therefore, a valid strategy is to consider a collection of c values and for each one conduct a regression analysis, while monitoring the acceptability of the c value in terms of Equation 16 or its equivalent Equation 19. The error of the estimation procedure is summarized by the degree at which Equation 19 is held as an equality. Of course, for each choice of c the variance of the estimates of the parameters a and b can be measured by using well-known regression analysis results. These results are not given in this paper but are available in any textbook on elementary statistics that includes a discussion of the variance of the estimates in regression analysis.

PREDICTION OF PAVEMENT DISTRESS

Pavement distress is best represented in two separate components: density and severity. Density may be expressed as either the percentage of the total pavement surface area that is covered by the distress, or total crack length per unit area or crack spacing or similar measures. Severity may be expressed as either an objective or subjective measure. Examples of objective measures are crack width, crack depth, and relative displacement at a joint. Subjective measures may be assessed reliably by comparing the observed distress with photographs of different levels of severity. The severity may be described as none, slight, moderate, or severe and may be given numerical ratings such as 0, 1, 2, and 3, respectively, or be assigned numbers that are proportional to these in a range between 0 and 1. The change of either area or severity of distress can be evaluated by using the previously discussed equations.

To study the behavior of the area covered by a given type of distress and the corresponding level of severity, two indices are introduced: (a) the distress area index, and (b) the distress severity index. Each of these indices represents a number between 1 and 0 that decreases as the level of traffic is increased. Note that the present serviceability index has a similar behavior, with the exception that it decreases from P_o to P_f.

Specifically, the distress area index decreases from a value A_o ($A_o \leq 1$) to a value A_f ($0 \leq A_f \leq A_o$) as the traffic increases; similarly, the distress severity index decreases from a value of S_o ($S_o \leq 1$) to a value S_f ($0 \leq S_f \leq S_o$) as the traffic level increases. Note that both the area and severity indices are reduced as traffic increases; that is, a recently rehabilitated pavement will have indices close to one, as opposed to pavements in need of rehabilitation, which will have indices close to zero.

The distress area index (A) is expressed by a relationship similar to that of Equation 3, namely,

$$A = A_o - (A_o - A_f)\exp[-(\rho/W)^{\beta}] \quad (20)$$

Similarly, the distress severity index (S) is expressed as

$$S = S_o - (S_o - S_f)\exp[-(\rho/W)^{\beta}] \quad (21)$$

Using the A, S, and W data from the Texas Transportation Institute data base it is possible to estimate A_f, S_f, ρ, and β following the procedure described by Equations 5-19 for each of the following types of distress: rutting, raveling, flushing, alligator cracking, longitudinal cracking, transverse cracking, and patching.

APPLICATION, SUMMARY, AND CONCLUSIONS

The S-shaped performance curve is found to adequately describe the performance of flexible pavements in Texas as a result of increased traffic levels. This behavior has been analyzed primarily in terms of the decrease in the present serviceability index (PSI) as a function of the number of 18-kip equivalent axle loads. The proposed performance curve was developed on the basis of observed data for pavements in each of the following categories: black base, hot mix, and overlays. A more detailed description of the curve fit parameters, along with the original data, can be found in the report by Garcia-Diaz et al. (1). The data in Table 1 give the number of test sections in each category along with mean value and minimum and maximum observed values of the design parameters. The mean values of the curve fit parameters were obtained from the statistical procedure described earlier in this paper. Figure 3 shows the average performance curves obtained by using these design parameters (see Table 1) for each of the three pavement types.

The analysis of the data revealed four possible cases for the curve fit. Typical test sections for

TABLE 1 Serviceability Performance Curve Parameters by Pavement Type

Pavement Type	Black Base	Hot Mix Asphalt Concrete	Overlays
Number of Test Sections	51	36	77
ρ (mean)	2.321	1.960	1.974
ρ (min)	0.005	0.100	0.013
ρ (max)	17.239	11.098	9.188
β (mean)	1.337	1.952	1.196
β (min)	0.300	0.095	0.095
β (max)	6.277	7.259	2.893
P_o (mean)	4.15	3.87	3.92
P_o (min)	2.79	2.86	2.07
P_o (max)	4.77	4.78	4.88
P_f (mean)	1.962	1.661	2.121
P_f (min)	0.000	0.000	0.004
P_f (max)	4.295	4.305	4.391

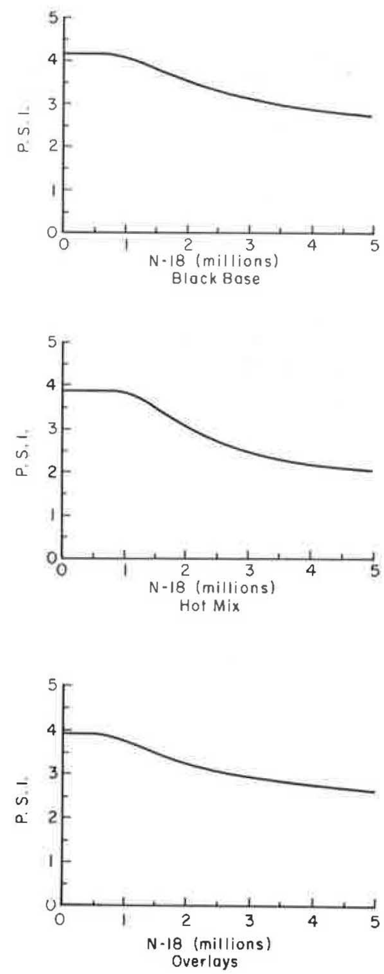

FIGURE 3 Performance curves from the mean design parameters.

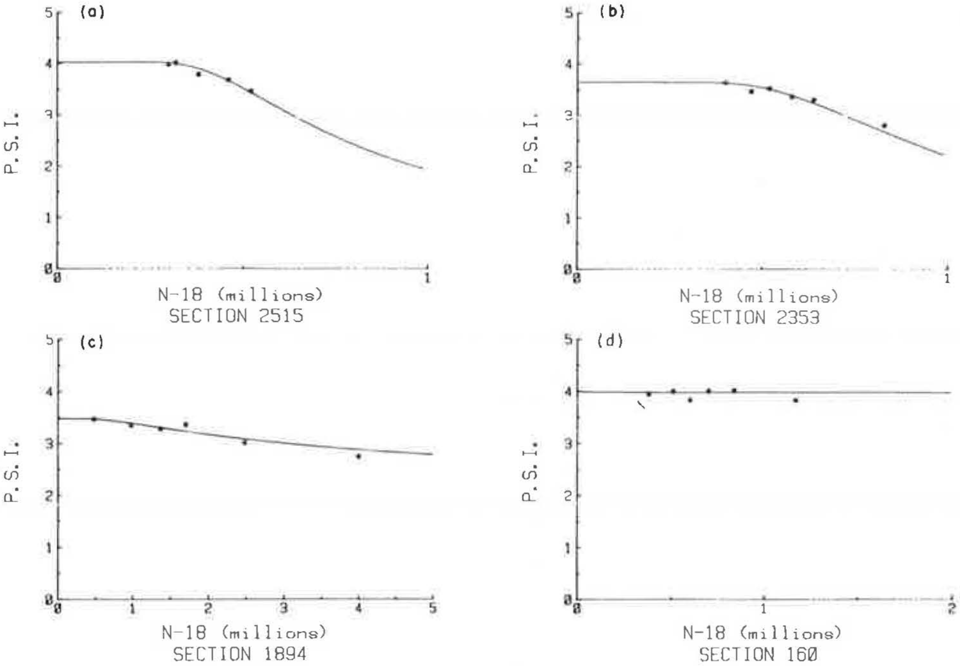

FIGURE 4 Typical sections for the four cases of serviceability performance.

each case are shown in Figure 4. A description for each case follows.

1. Case 1 (Figure 4a): $\rho > 1$, $\beta > 1$, and $P_o > P_f$. Note that the complete S-shaped pattern can be distinguished. The percentage of pavements of this type = 26.9. The example shown in for a black base (test section 2515).
2. Case 2 (Figure 4b): $\rho > 1$, $\beta > 1$, and $P_o > P_f$. Note that the upper half of the S-shaped curve is observed. The percentage of pavements of this type = 28.6. The example shown is for an overlay (test section 2353).
3. Case 3 (Figure 4c): $\rho > 0$, $\beta < 1$, and $P_o > P_f$. Note that the lower half of the S-shaped curve is observed. The percentage of pavements of this type = 21.3. The example shown is for an overlay (test section 1894).
4. Case 4 (Figure 4d): $\rho > 0$, $\beta = 0$, and $P_o = P_f$. Note that no noticeable curve is observed. The percentage of pavements of this type = 21.3. The example shown is for an overlay (test section 160).

TABLE 2 Primary Distress Type and Curve Fit Parameters by Pavement Type

Pavement Type	Black Base	Hot Mix Asphalt Concrete	Overlays
Type of Distress	Alligator Cracking Severity	Alligator Cracking Area	Transverse Cracking Severity[a]
ρ(mean)	1.19	0.93	85.57
ρ(min)	0.14	0.07	24.13
ρ(max)	3.01	3.63	194.83
β(mean)	2.54	3.43	1.47
β(min)	0.89	0.50	0.50
β(max)	8.78	18.21	5.52

[a]The ρ and β terms for this case are determined in terms of the number of months the pavement has been in service.

The S-shaped performance curve is also applicable in the analysis of distress data. For this case the assumptions that $A_o = S_o = 1$ and $A_f = S_f = 0$ will simplify the analysis because only the parameters ρ and β remain to be estimated. These values can then be used to develop a performance curve for each of the two distress indices (area and severity). The development and application of distress models for rutting, alligator cracking, longitudinal cracking, and transverse cracking are summarized elsewhere (1). Sample results of this analysis for distress types found to be critical [using the method of discriminant analysis as discussed by Allison et al. (3)] are given in Table 2.

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Development and Testing of a Portable Microprocessor-Based Capacitive Weigh-in-Motion System

DAVID R. SALTER and PETER DAVIES

ABSTRACT

In this paper current techniques for the measurement of dynamic axle load are reviewed and the development and testing of a prototype portable weigh-in-motion (WIM) system at the University of Nottingham, England, are described. Current techniques for dynamic axle load detection tend to be characterized by a need for permanent sensors within the road pavement that can be both costly to install and maintain. A portable WIM system that uses a temporary capacitive weighmat has been developed at the National Institute for Transport and Road Research of South Africa, and extensive tests have revealed that the system can provide accurate results for accumulated axle loadings over large samples of vehicles, but that results for individual axle loads are subject to larger errors. In an attempt to improve the reliability of weight data from the capacitive sensor, a new microprocessor-based detector unit was developed at the University of Nottingham. Some current WIM systems developed in the United States and in the United Kingdom are evaluated, and the theory of operation of the capacitive sensor is described. Laboratory tests undertaken to determine the response of the sensor to controlled loading are discussed in detail, and the prototype detector unit is described along with the results of field trials conducted on a main U.K. highway. The results of these trials indicated that improvements over previous capacitive WIM systems had been achieved and that the combination of a commercially available capacitive sensor and a new microprocessor-based detector unit provided axle load data that was within ±15 percent of static weights. A commercially available capacitive WIM system, developed by the Golden River Corporation from this prototype system, is described along with a second-generation automatic weighing and classification system that was under development at the time of writing.

The geometric design of a highway is based on estimates of the expected total flow of traffic and the expected changes in the traffic flow over the design life of the road. Pavement design depends on other factors, and structural damage is caused almost exclusively by commercial vehicles (1). The normal practice in Europe and North America is to express the expected number of commercial vehicles as the cumulative total of equivalent standard axles. In the United Kingdom this is calculated by multiplying the expected number of commercial vehicles by two design factors, one characterizing the average number of axles per commercial vehicle and the other the number of standard axles per commercial axle.

The accuracy of the standard axle estimates is dependent on the reliability of data on past and present trends in axle loadings, which are in turn related to the size of the vehicle samples monitored. When manual classification and recording of axle loads are necessary, sample periods may be too short to allow accurate annual estimates of traffic loading to be deduced. In addition, axle load statistics tend to lag behind trends toward larger and heavier trucks.

The use of automatic vehicle classification systems (AVCSs) will enable data on axle numbers and vehicle types to be collected over extended periods. This could allow better estimates of heavy vehicle flows and of average numbers of axles per truck. Weigh-in-motion (WIM) facilities in conjunction with classification equipment could go further by improving the estimates of numbers of standard axles for particular vehicle classes or by direct measurements of standard axle loadings.

Investigations into the incorporation of WIM with automatic vehicle classification have been undertaken by the Transport and Road Research Laboratory (TRRL). Results presented by TRRL (2) have indicated that current axle weighing systems can provide the required data; however, these systems are unlikely to gain widespread acceptance because of their high cost. Only a minority of automatic traffic monitoring installations are likely to incorporate WIM using current, high-cost technology.

The unsuitability of current systems and the increasing demand for axle load data led to a research program at the University of Nottingham on dynamic axle load measurement, complementing ongoing work on automatic vehicle classification. Initial studies indicated that several possibilities existed for low-cost WIM sensors, and two of these were selected for further development. Vibracoax piezoelectric cable is being studied at Nottingham under contract to TRRL. Capacitive weighmats were the second technique to be studied, and these form the subject of this paper.

The capacitive weighmat concept was first developed and patented at TRRL. Basson (3) reviewed further development of the approach carried out at the National Institute for Transport and Road Research (NITRR) in South Africa. Field tests with this system, reported by Basson and Paterson (4), indicated that cumulative axle load measurements were accurate for large samples of vehicles, but that individual axle results were poor. The aims of the work at Nottingham were to note if accuracies could be improved by using digital processing techniques, and to develop a capacitive system that would be compatible with automatic vehicle classification equipment.

PREVIOUS SYSTEMS

Axle weighing systems fall into two main categories. The first can be used for static weighing or, in some cases, for weighing axles moving at low speeds (less than 8 km/h). The second is designed to record the axle loads of vehicles as they travel at normal speeds within the traffic flow.

Systems that are only able to measure axle loads of vehicles that are stationary or moving very slowly have little application for the automatic recording of axle weights because vehicles have to be separated from the normal traffic stream and directed over the weighing equipment. The cost of manual weighing is high, both for the public agency responsible and in terms of truckers' delays. Many systems fall into these static and slow-speed categories.

The most common static weighing devices use electrical resistance load cell units to support a steel deck onto which vehicles are driven. Alternatively, the plate can be fitted with strain gauges to measure its bending under load. The load cells and deck form a platform that may be sunk into the pavement or it may be surface mounted. Using this technique, wheel loads can be determined to a high degree of accuracy, especially if precautions are taken to eliminate the effects of vehicle tilt. Some of these systems are portable, but most are permanently fixed into pits in the pavement.

A few systems have been designed to measure the axle loads of vehicles moving at slow speeds in order to increase the efficiency of the weighing operation. They are generally large pieces of equipment consisting of a weighing platform wide enough to accommodate an entire axle or tandem. These slow-speed devices are similar to static weigh scales and accept a small loss of accuracy in exchange for an increased throughput of vehicles.

The first in-motion weighing systems were developed in the United States (5). Later work by Lee (6) led to the development of the Radian portable weigh scale, which consisted of steel platforms supported on eight load cells and mounted in a shallow pit. Wright (7) reported that the accuracy of the system is acceptable for most design purposes. However, running costs are high because constant manning is required during operation.

Several European systems use different techniques. The TRRL dynamic weighbridge consists of three or four units mounted side-by-side across one wheel track. Each unit contains an arrangement of load cells and springs, and each is mounted in a steel frame set in reinforced concrete. Bundesanstalt fur Strassenwesen (BAST) in West Germany developed its bending plate system for in-motion weighing (8) with three wider steel plates supported by a light steel frame in a shallow pit. Finally, the French Laboratoire Central des Ponts et Chausees (LCPC) developed a dynamic balance by using piezoelectric transducers located within alloy units sunk into the pavement (9).

Bergan and Dyck (10) describe the development of a dynamic weighing platform at the University of Sascatchewan in Canada. The unit consists of two rectangular plates resting on a common foundation. One platform is located in each wheel track. Loads on the platforms produce a vertical movement in a centrally located oil-filled piston, which acts as a load cell. Additional vehicle parameters are measured by using inductive loops.

These dynamic axle-weighing systems are all characterized by a need to excavate the road pavement for installation. Installation costs are generally high, and the work disrupts traffic flow for substantial periods of time. The objective of the work described in this paper was to investigate fully portable systems that do not require any permanent fixtures in the highway for their operation.

CAPACITIVE SYSTEMS

A design for a flexible weighpad consisting of two or more parallel sheets acting as the plates of a capacitor was patented in 1968 by J.J. Trott and J.W. Grainger (Improvements in Capacitors, U.K. Patent No. 1234083, filed April 2, 1968). This device consisted of three perforated plates separated by and enclosed in layers of natural rubber. Subsequent inventions by R.P. Miller (Electrical Weighing Apparatus Using a Capacitive Flexible Mat, U.S. Patent No. 3565195, filed April 16, 1969) and by S.H. Kuhn, C.R. Freeme, R. Beulink, and J.E.B. Basson (Measuring Transient Loads, U.S. Patent No. 3782486, filed May 12, 1971) were devices of slightly different construction, but which operated on the same principle.

The theoretical performance of the capacitive weighpad is described by Basson (3) by considering the deflection of a mechanical model of the weighpad sensor under load. Empirical data on the deflection of the actual sensor and on the relationships between tire contact area and wheel load were applied to this model to produce a theoretical relationship between axle load and percentage change in capacitance. An outline of this evaluation is presented here.

Consider the mechanical model shown in Figure 1. A wheel load (P) acts over a tire contact area (Aa) on a senor of total area (A). The sensor has an unloaded capacitance of Cu and a loaded capacitance of Cw. The sensor comprises three plates, each separated by a material of thickness X. The deflection of the sensor under load is dX.

The sensor capacitance in the unloaded state is given by

$$Cu = K \cdot 2 \cdot A/X \tag{1}$$

where K is a constant, depending on the dielectric properties of the material between the capacitor plates.

The sensor capacitance in the loaded mode is given by

$$Cw = [K \cdot 2 \cdot (A - Aa)/X] + \{K \cdot 2 \cdot Aa/[1 - (dX/2)]\} \tag{2}$$

Therefore the change in capacitance is given by

$$\begin{aligned} dC &= Cu - Cw \\ &= (K \cdot 2 \cdot A/X) - \{K \cdot 2\ [(A - Aa)/X]\} - \{K \cdot 2 \cdot Aa/[1 - (dX/2)]\} \\ &= K\left((2 \cdot Aa/X) - \{2 \cdot Aa/[X-(dX/2)]\}\right) \end{aligned} \tag{3}$$

Hence the change in capacitance relative to the initial sensor capacitance is given by

$$dC/Cw = \left((2 \cdot Aa/X) - \{2 \cdot Aa/[X-(dX/2)]\}\right)/(2 \cdot A/X) \tag{4}$$

or

$$dC/Cw = (Aa/A)\ \{1 - X[X-(dX/2)]\} \tag{5}$$

In order to evaluate the theoretical performance of the sensor, the relationships between wheel load, inflation pressure, and tire contact area were established by empirical tests on a variety of tire types. Similarly, the deflection of a weighmat sensor for varying loads and tire contact pressures was also measured in a laboratory testing rig. For a given wheel load and inflation pressure, it was therefore possible to calculate the tire contact area, the average contact pressure, and hence the sensor deflection. These could then be substituted into Equation 5 to give the theoretical change in capacitance for a particular wheel load and inflation pressure.

The results of the exercise indicated that the relationship between wheel load and percentage change in sensor capacitance was linear and passed through the origin. There was also no apparent difference in the relationship for the range of wheel loads and inflation pressures found on single or dual-wheeled axles.

Construction of the three-plate capacitive sensor developed at NITRR is described by Basson (3). Initial designs comprised steel mesh conductors separated by a polyurethane dielectric. Problems with the mechanical strength of the mesh and dielectric and the sensitivity of the polyurethane to changing loads led to a final design comprising steel plates separated by natural rubber. The plates are encased in a tough synthetic rubber compound. The sensor unit, which is 1.8 m x 0.4 m x 7 mm in thickness, is secured to the road in one wheel track by means of perforated plates pop-riveted to its sides. The perforated plates are fixed to the road with strips of bituminous tape and road nails, as shown in Figure 2.

The equipment has had extensive testing in South Africa and has also been appraised in other countries. Results from South Africa, reported by Basson and Paterson (4), indicate that with correct calibration the system gave accurate results for accumulated axle loadings over large samples of vehicles. However, individual vehicle results were subject to large errors, as indicated for a 500-vehicle sample given in Table 1. Graphical results from a 13-site evaluation program in South Africa are also shown in Figure 3, where dynamic load from the Axle Weight Analyser is plotted against actual static axle load.

The apparent conclusion that might be drawn from the NITRR weighmat studies is that practical capacitive WIM systems have not entirely lived up to their theoretical performance. On the other hand, the systems demonstrate considerable promise as relatively low-cost, portable approaches that might be refined to produce results of higher accuracy. With these tentative conclusions in mind, a program of laboratory and development studies was commenced at the University of Nottingham during 1982.

LABORATORY TESTS

At the start of the laboratory tests an existing capacitive weighmat system was tested under laboratory conditions to determine its response characteristics to loading. This was followed by the development of a microprocessor-based monitoring

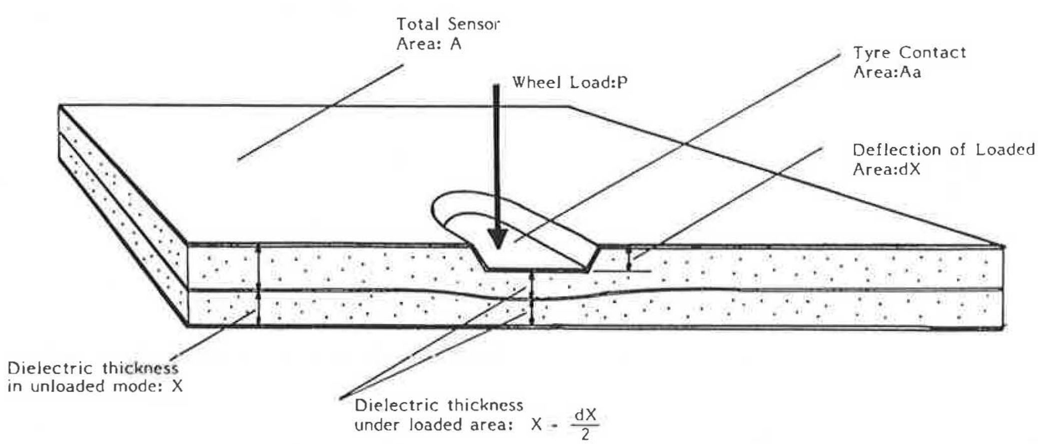

FIGURE 1 Model of the capacitive sensor under load.

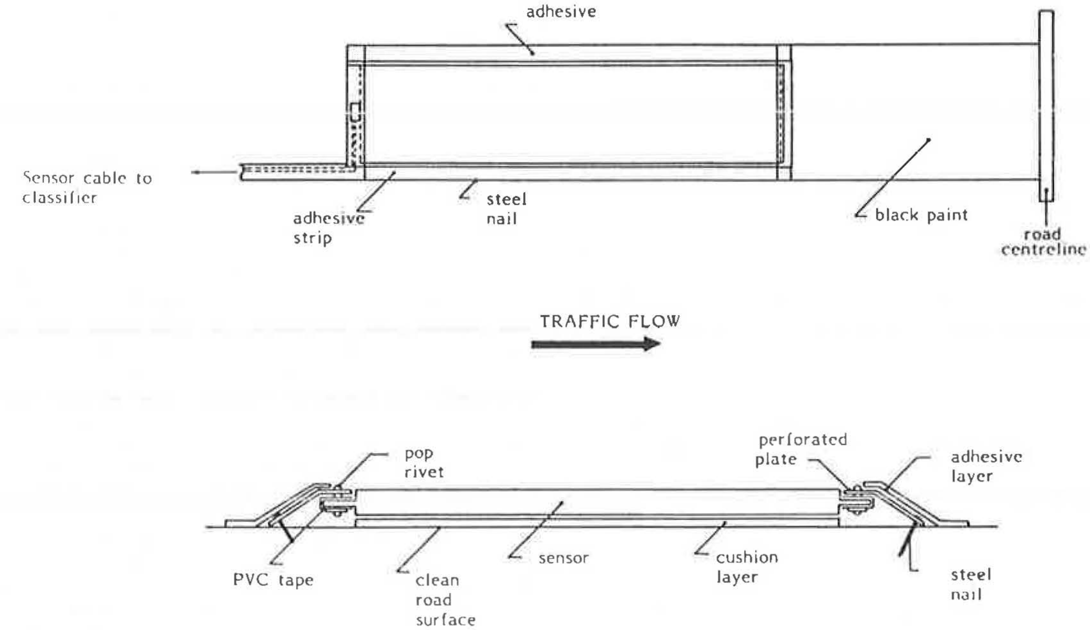

FIGURE 2 Sensor layout for the NITRR dynamic weighbridge.

TABLE 1 Accuracy of Static and Dynamic Load Comparisons with NITRR Equipment

Confidence Level (%)	Percentage Errors			
	Total Mass		Single-Axle Mass	
	Lower	Upper	Lower	Upper
80	-4	+11	-42	+67
90	-6	+14	-58	+83
95	-9	+16	-71	+96
99	-14	+21	-97	+122

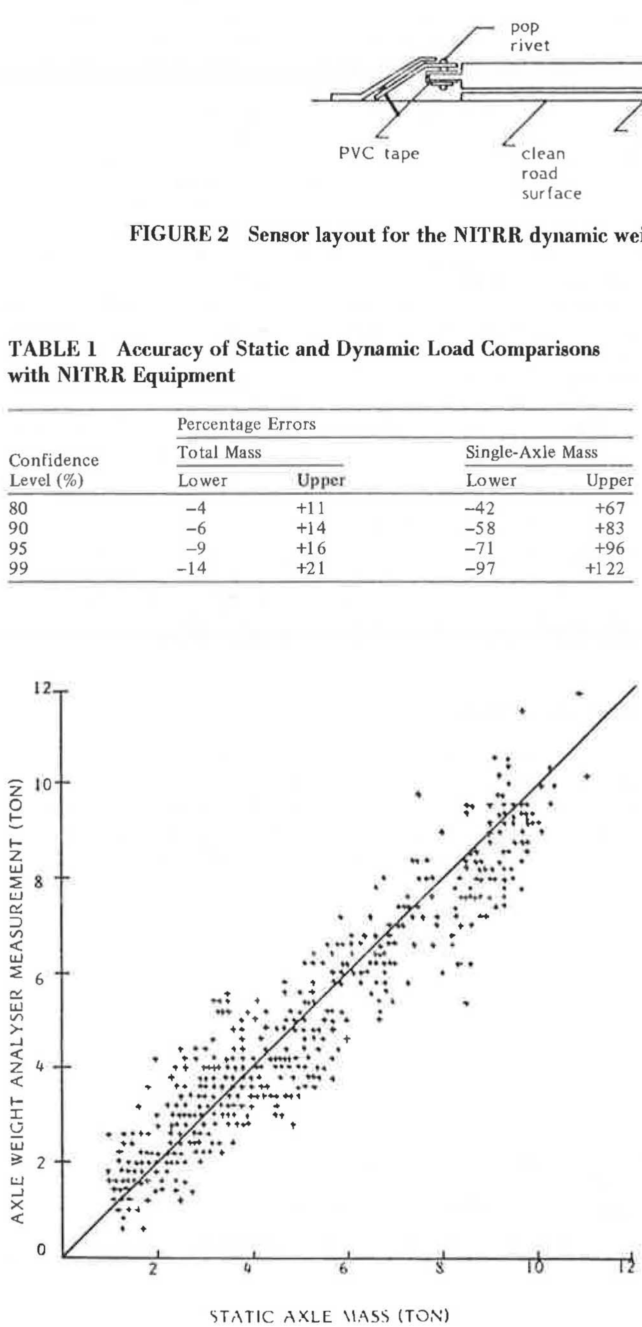

FIGURE 3 Comparison between static and dynamic axle loads with the NITRR weight analyzer.

system that was capable of automatically recording the weight of axles passing over the weighmat. The new system was then tested under field conditions to assess its accuracy and reliability.

The existing system tested at the start of the program consisted of a capacitive weighmat sensor and a dedicated electronic detector unit. The sensor was constructed from perforated steel sheets encased in rubber, which formed the parallel plates of a capacitor. The dedicated electronics comprised a frequency-to-voltage convertor that translated changes in sensor capacitance to a proportional analogue voltage output. For this purpose, the sensor formed part of a tuned circuit driven by a sinusoidal oscillator. Increases in sensor capacitance produced a reduction in the frequency of oscillation that was converted to a change in the voltage output. The analogue voltage was then used as the input to an electro-mechanical counter unit.

The laboratory tests carried out on the system involved the application of static and dynamic loads applied in an electronic servo-controlled hydraulic load testing machine, similar to those developed at the University of Nottingham for the testing of subgrades, pavements, and piezoelectric axle load sensors. A fairly large capacity machine was required to simulate actual tire contact pressures over the loading plates, which were approximately the same size in contact area as a vehicle tire. The loading rig was capable of producing at least 10 tonnes at a frequency of 10 Hz. For load applications, the mat was divided into 150-mm sections numbered 1 to 12, and loading tests were conducted at each position.

The weighmat was subjected to four basic loading tests to determine its response to static and dynamic loading. Initially, the capacitance change of the weighmat under various static loads was measured directly with a Wayne Kerr capacitance bridge unit. Figure 4 shows the capacitance change of the weighmat over the load range of 0 to 10 tonnes. The relationship is not linear, but capacitance increases throughout with load.

To verify that the signal-conditioning elec-

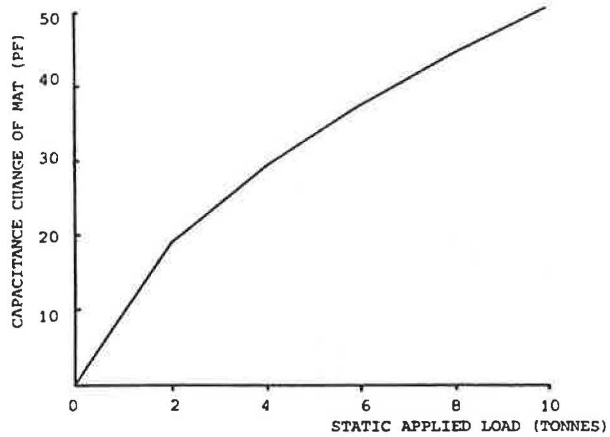

FIGURE 4 Change in weighmat capacitance with applied load.

tronics of the weighmat system were not distorting this relationship, the same loadings were applied at the same position on the mat and the corresponding output from the detector unit was recorded. The two sets of results were then constrained to coincide for loads of 0 and 10 tonnes on a composite graphical plot. Figure 5 shows that for observations at a constant temperature, the voltage output from the detector unit follows close to the capacitance change of the mat.

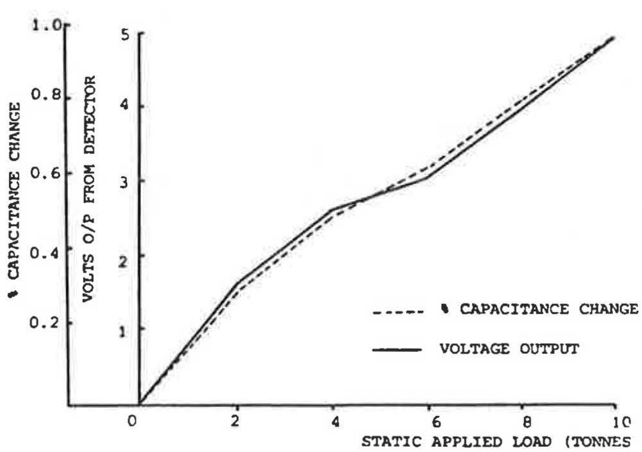

FIGURE 5 Voltage output from detector compared with mat capacitance change under varying load.

The weighmat was then loaded through four different footprint areas to determine the effect of changing tire size and configurations. The results, shown in Figure 6, indicate that the output from the weighmat is not wholly independent of tire contact area as would be suggested by the theory of operation.

Using the dynamic capability of the loading rig, a series of tests was undertaken to determine the sensitivity of the weighmat to variations in the frequency of the applied load. Loads of 0 to 10 tonnes were applied at a variety of positions along the mat and at a range of frequencies. Figure 7 shows the peak output from the detector for various dynamic loads applied at frequencies of 2, 5, and 10 Hz. These results are typical of the plots obtained

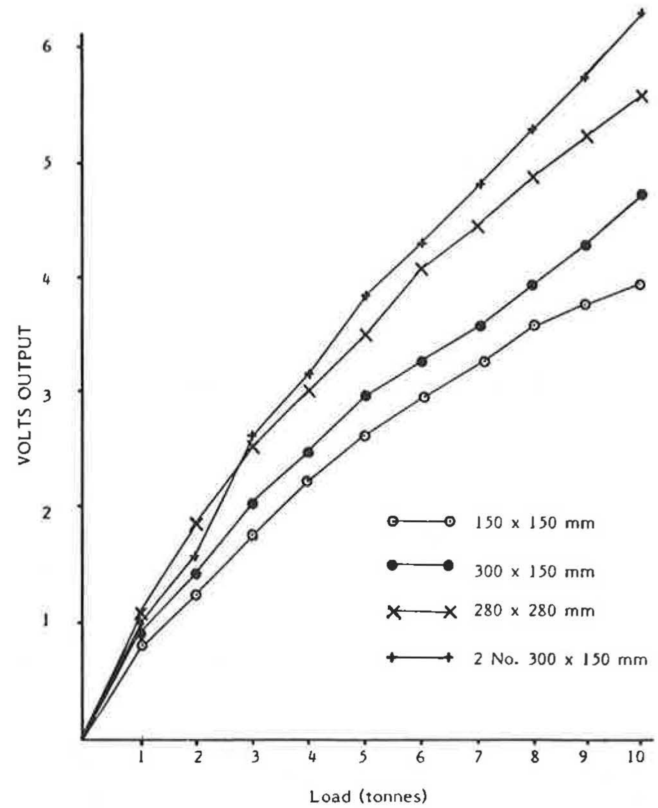

FIGURE 6 Weighmat output with varying load and tire footprint area.

at other positions on the mat and indicate that the detector output is independent of the frequency of the applied load.

Figure 8 shows the variation in positional sensitivity of the weighmat for a range of dynamic loads, all applied at a frequency of 5 Hz. The results indicate that the sensitivity of the weighmat is not constant over its length. Variations of approximately ±10 percent are evident over the width of the sensor.

PROTOTYPE HARDWARE DEVELOPMENT

Following the tests on this first design of the weighmat, a second weighmat was tested to the current NITRR specification. The two main differences from the earlier weighmat design were (a) the parallel plates of the capacitor were fabricated from three solid-steel sheets rather than from perforated plates, and (b) the sine-wave oscillator that drives the tuned circuit was incorporated in the cable connector fixed to the weighmat on the road. This latter development has the effect of eliminating any effects of capacitance changes in the connecting cable between the mat and the detector unit.

A prototype microprocessor detector unit was developed to work with the modified design of the weighmat, based on digital loop detector technology. Digital loop detectors use the loop as the inductive element in a tuned circuit, where changes in loop inductance produce a change in the resonant frequency of that circuit. By using two crystal oscillators, the number of oscillations of the tuned circuit in a given time period can be counted. This count is used by the unit as a digital measure of loop inductance.

The digital measurement of capacitance was ap-

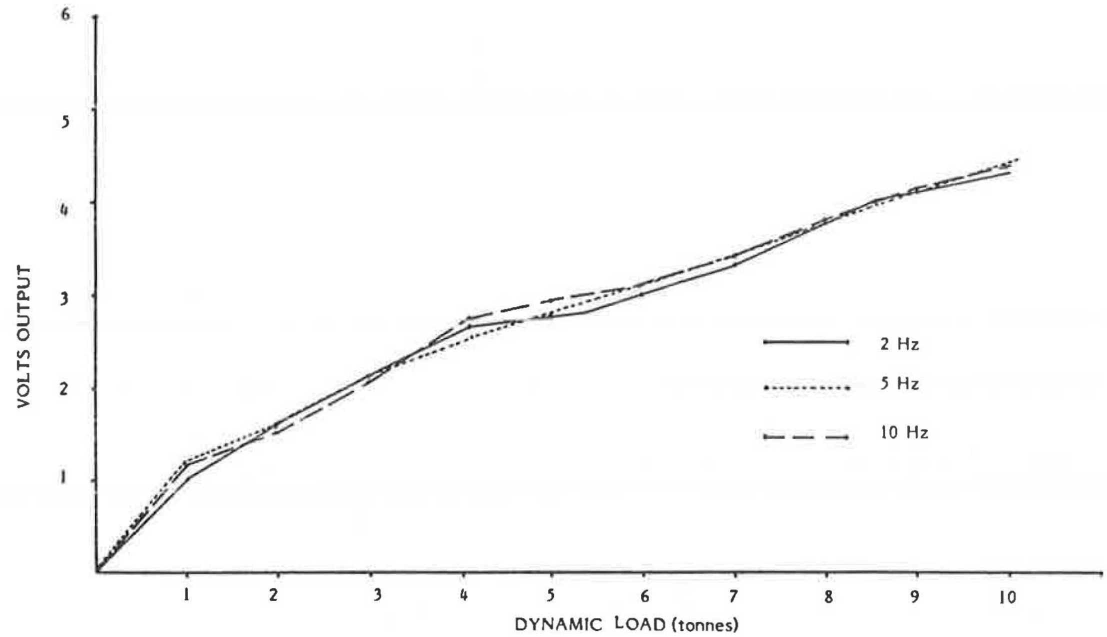

FIGURE 7 Weighmat output for three different loading frequencies.

proached in the same way. The capacitive weighmat replaced the inductive loop as the variable element in a current digital loop detector unit. This modified loop board was mounted in a Golden River portable roadside Environmental Computer at the University of Nottingham. Machine code routines were written for the manipulation of this digital weigh board output, initially for its display on a video display unit (VDU). A further series of laboratory tests was undertaken by using this new detector unit to establish its compatibility with the capacitive sensor for axle load detection.

Initially, further laboratory tests were undertaken to determine the response of the new sensor and detector units to controlled loading. The weighmat was loaded in the hydraulic testing rig at a variety of positions by using three standard static loads. Figure 9 shows the digital output from the detector for loads of 2, 4, and 6 tonnes. The results indicated that the combination of the capacitive sensor and new detection hardware produced linear outputs with load, and that variations in sensitivity along the length of the mat were on the order of ±10 percent.

To examine the sensitivity of the mat to changing environmental conditions, a series of temperature tests was undertaken. The sensor unit was heated with infrared lamps, and the temperature of the top and bottom surfaces of the mat, in the vicinity of the load position, was recorded with

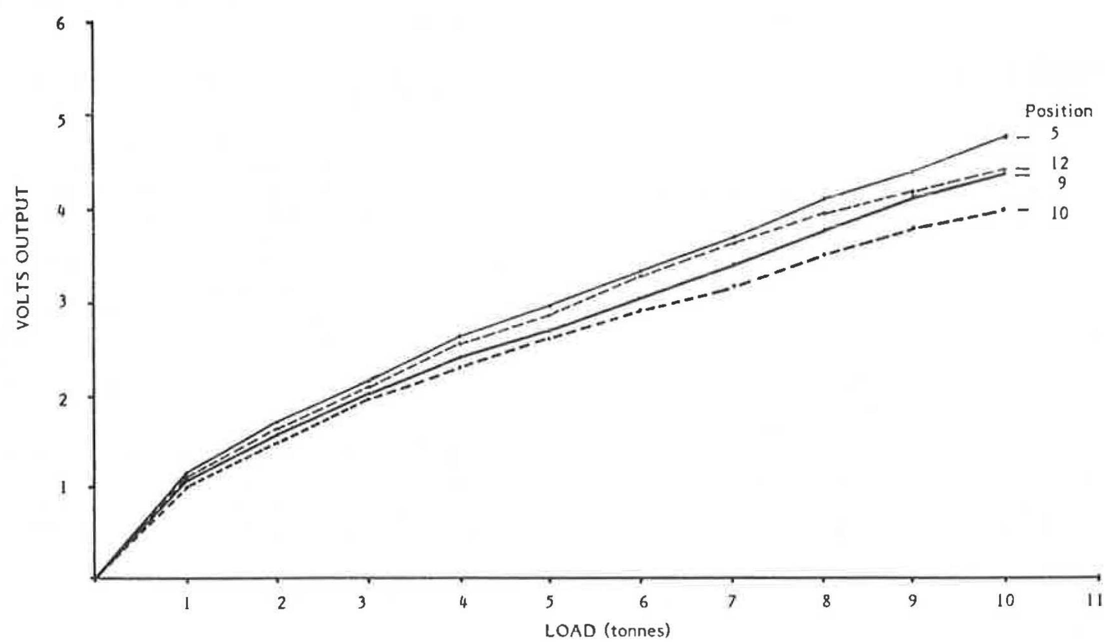

FIGURE 8 Positional sensitivity of the weighmat with dynamic loading.

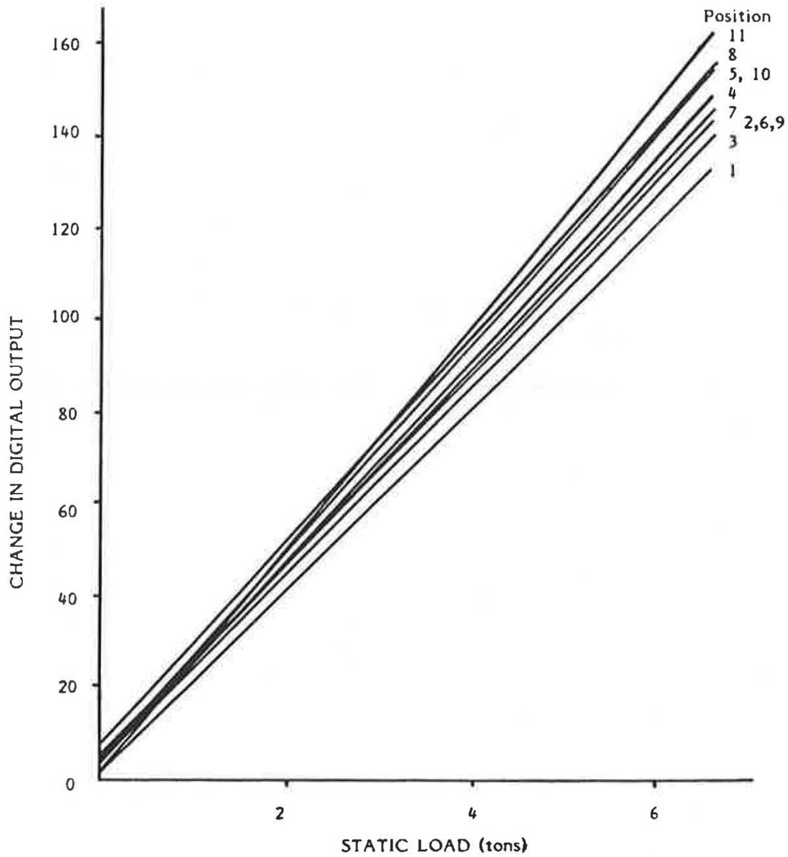

FIGURE 9 Weighmat sensitivity with varying load and lateral position.

thermocouples. When stable temperatures were reached, static loads were applied and the detector output noted. Figure 10 shows the variations in the output for four mat temperatures.

Both the absolute output from the mat and the changes in output with load vary with temperature. The absolute output variations could be caused by oscillator characteristics as well as by expansion of the rubber dielectric of the capacitor because of heating. The changing sensitivity with load was attributed to variations in the elastic properties of the rubber dielectric between the capacitor plates with temperature. However, the effect is only marked at relatively high temperatures and wheel loads.

For the laboratory tests, the software used for

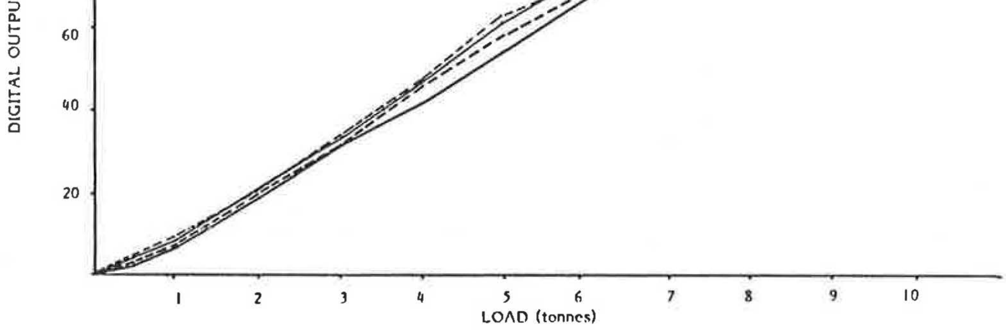

FIGURE 10 Variations in weighmat output with temperature.

the digital readout from the new detector system had been limited to simply reading and displaying the raw digital value from the modified loop board. Before the system could be used for continuous monitoring of vehicle loads, software had to be developed to determine when an axle was on the mat, to smooth the signal, and to read and store the maximum digital value produced by that axle.

SOFTWARE DEVELOPMENT

An assembly language program was written at the University of Nottingham to perform the tasks of signal processing and data storage for axle load measurement. The Golden River Environmental Computer's Forth Assembler was used to produce a machine code routine capable of scanning the weighmat at high speed in real time.

The software for the detection, manipulation, and storage of digital signals from the weighmat had to be capable of performing six principal functions:

1. To determine, by smoothing and processing signals from the mat, when an axle is present on the sensor;
2. To read the digital output from the modified loop board at a fast enough scanning rate to ensure that the peak of the signal is detected;
3. To determine, in the period between signal scans, the amplitude of the signal and assess whether it is the peak value;
4. To determine when the axle has left the mat and store the difference between the peak and the base value;
5. To track any gradual changes in mat capacitance resulting from environmental drift; and
6. To allow user control over the running of the program and the reading of the stored data.

A typical vehicle wheel traveling at 40 mph will pass over the mat in approximately 30 to 40 milliseconds. The duration of the peak value was not known at first, but it was thought likely to be only a few milliseconds. To achieve the fast scanning rate required to detect this peak, functions 1-6 were performed in the machine code portion of the routine. The user control program was written in Forth, which is a higher-level language fast enough for between-vehicle processing.

The main steps of the code routine are as follows:

1. The routine tells the weighmat processor board that a digital value of mat capacitance is required. A special function of the capacitance value is taken to speed processing at a later stage.
2. The value read on this scan of the mat is smoothed exponentially with a previously weighted value to reduce the effect of noise.
3. The smoothed value is then compared with a threshold. If the current value exceeds this threshold, a wheel is assumed to be on the mat.
4. The maximum value of the signal from the weighmat is accumulated as the wheel passes over the sensor.
5. If the mat has been active for a relatively long period, it is assumed to be locked on. This might be as a result of sudden heavy rainfall, or could be caused by a vehicle parking on the sensor. In this event the routine is automatically reset.
6. If the mat has just changed from being active to inactive, the maximum value it has accumulated is stored in memory.
7. Automatic tracking and compensation take place when the sensor is inactive to allow for factors such as temperature changes.
8. The user keyboard, located on the front panel of the computer, is checked to determine whether the user wishes to suspend program operation.

The Forth program that permits user control allows the user to operate the computer in one of four modes: (a) run the assembler program, (b) successively display the last N peak values stored in memory on the eight-digit LCD display on the computer's front panel, (c) display the number of memory bytes already used for data storage, and (d) display capacitance scans (signatures) of vehicles. The Environmental Computer operates off its own internal battery supply and therefore, by using this software, it was possible to test the weighmat in the field and to assess its performance for dynamic axle load detection.

FIELD TRIALS

To assess the performance of the system for automatically recording axle loads, a series of correlation exercises was undertaken. The sensor unit was fixed to a highway that had a good ride quality. Vehicles that had previously been weighed at nearby static scales were recorded as they passed over the weighmat. Weights were subsequently compared by matching license plate numbers, which is easy in Europe because of the large size of the plates.

In addition to the recording of these random vehicles, two test vehicles were driven over the sensor at a range of speeds. Single-axle loads of up to 10 tonnes were recorded. Tandem axles were weighed together on static scales but were recorded separately on the weighmat. These dynamic recordings were combined to allow them to be plotted on the calibration graph given in Figure 11. Recordings were taken on three separate days, and the weighmat was removed at the end of each of the recording periods.

The results of this correlation exercise indicated that the capacitive sensor, coupled with a prototype microprocessor-based detector unit, was capable of recording dynamic axle loads in the range 0 to 10 tonnes to within approximately ±15 percent of their static value at a confidence level of 95 percent. For the heaviest loads, where the individual axles of tandems were recorded separately and combined to correlate with weighbridge data, the accuracy of the comparisons improved to approximately ±10 percent for 95 percent confidence. These figures correspond to statistical accuracies (standard errors) of about 7.5 and 5 percent, respectively.

PRODUCTION SYSTEMS

Production capacitive WIM systems have been developed by Golden River Corporation from the research prototype development system previously described. These systems have involved complete hardware and software redesigns to take account of a number of development program findings and to ensure compatibility with other Golden River Marksman and Retriever equipment.

The Marksman Axle Weight Classifier uses a capacitive weighmat sensor with a portable roadside Marksman microprocessor traffic counter and classifier. Axle counts in 12 user-defined weight bins are stored in solid-state memory at preset intervals of between 1 min and 24 hr on the internal clock and calendar. Individual axle weights can also be displayed in any appropriate unit for checking of calibration.

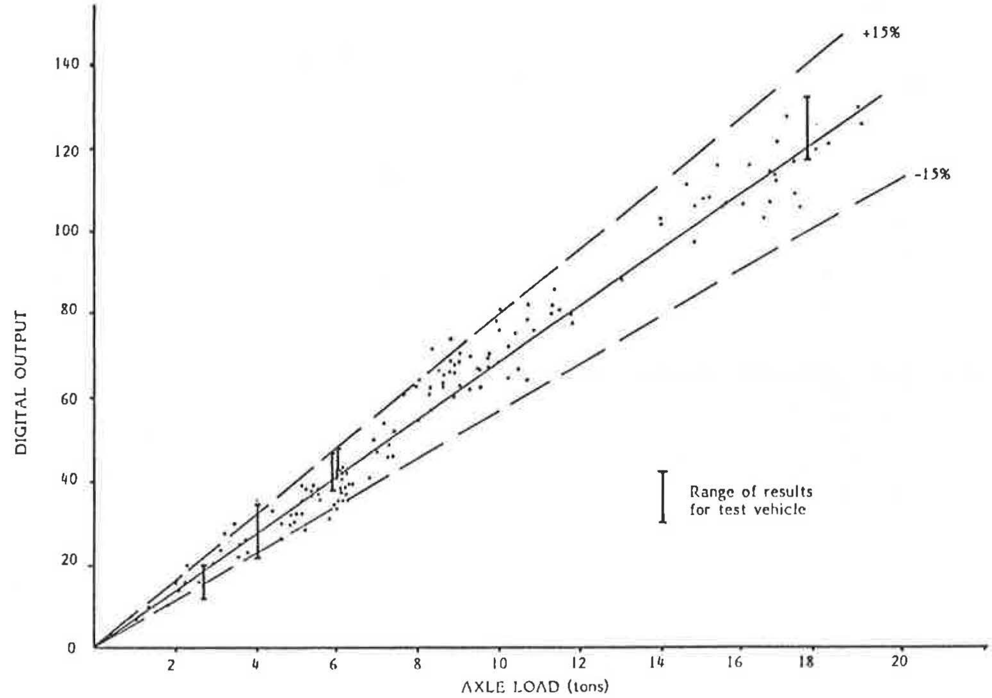

FIGURE 11 Results from the weighmat calibration exercise.

Data retrieval is by a separate microprocessor-based Retriever, which is compatible with other Marksman counters and classifiers, or by direct telephone modem. Internal rechargeable batteries will support the Retriever for several days and the Marksman for about 5 weeks. The Marksman weight classification equipment has been tested in the United Kingdom and in the United States in Arizona.

The second production capacitive weighing system is the Golden River Advanced Vehicle Classification and Weighing System (AVCWS), which will be available shortly. This equipment will count, weigh, and classify vehicles by type, as required by the user. Operation is fully automatic and takes place in real time. Vehicles are classified into 1 of 14 categories recommended by FHWA; for each class, leading axles, other axles, tandems, and gross vehicle weights are recorded in 12 user-defined weight bins, thereby giving a large number of possible categories. Alternatively, data can be grouped into fewer weight or type categories, or individual vehicle dimensions, weights, speeds, and class can be stored in memory for subsequent analysis.

Recording intervals can again be preset by the user, although necessarily with short intervals and using maximum numbers of categories, because memory storage capacity limits the interval between data retrieval. Compliance with the Bridge Formula is also tested for each vehicle in real time, and the proportion of vehicles violating the formula is stored in each time period. Fully automatic adjustment of axle weights is included in the microprocessor algorithms according to vehicle speeds and ambient temperature.

Road sensors for classification and weighing consist of two inductive loops and one capacitive weighmat per lane. If classification is required without weighing, the weighmat can be replaced by a pneumatic tube. Fully portable operation can be achieved by the use of bituthene or similar temporary loops, or permanent speed measuring loops can be used instead. The equipment is housed in a standard Marksman box with internal rechargeable battery power, and is again serviced by a Retriever for initialization and data retrieval. System accuracies have yet to be established, but are likely to be similar to the prototype for individual axle weights, rather better for gross vehicle weight, and around 90 to 95 percent accurate for 14 vehicle type classifications.

SUMMARY

Capacitive weighmat systems have been tested under laboratory conditions to determine their response characteristics to load. Investigations revealed that the sensor unit had a reasonably linear response to load, but that the output was also related to the tire contact area and to the position at which the sensor was loaded.

A new microprocessor-based detector unit has been developed for the capacitive weighpad, and initial field trials have indicated that the statistical accuracy of dynamic axle load determination using this portable system is in the region of ±7.5 percent. On the basis of this work a Marksman Axle Weight Classifier has been developed and further tests on the system are currently in progress. The combination of axle weighing and automatic classification facilities is now a practical proposition, and the first AVCWSs are expected to be available shortly.

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Development of a Data Base for Nondestructive Deflection Testing of Pavements

M. Y. SHAHIN, D. D. DAVIS, and S. D. KOHN

ABSTRACT

Currently the U.S. Army is using a pavement management system called PAVER that was developed by the U.S. Army Construction Engineering Research Laboratory (CERL). Along with the Army, the American Public Works Association, the U.S. Navy, and the U.S. Air Force have implemented the management system at several sites. The present system has been developed over several years. The system is centered around a hierarchical data base used to store pertinent information [System 2000 is the data base management system (note that System 2000 is a registered trademark of Intel Corporation)]. Using the data base and interface analysis programs, the user is provided with rapid report generation and analysis of critical information, which allows objective input to the decision-making process. A recent addition to the data base structure is the ability to store nondestructive deflection testing data. The development of the data structure used to store this information and its planned use are described.

Nondestructive testing (NDT) deflection data are an important addition to the PAVER pavement management system for the purpose of pavement design and evaluation and condition prediction. The PAVER system is designed to be a comprehensive management tool (1,2). Therefore, it is imperative that all relevant pavement information for management at both the project and network levels be included. The concept of storing all data in a comprehensive data base structure, where it can be manipulated and processed, is also appealing to the user from an organizational viewpoint.

At the project level NDT data are used for the purposes of pavement evaluation and subsequent restoration, rehabilitation, and resurfacing design. There are several deflection-approach pavement design schemes that require deflection information as input. NDT can also be used to determine in situ material properties of individual layers such as modulus of elasticity (E). This is usually done based on deflection values, layer thicknesses, and using analysis techniques such as elastic layer or finite-element methods. The in situ material properties are used for computing stresses and strains for the selected design vehicle(s), which are in turn used to compute remaining pavement structural life based on past and future traffic.

At the network level NDT data can be used for planning and forecasting. The deflection values, normalized for temperature and time of year, can be assumed to be constant until very near failure. Thus a pavement's deflection, or a derivative function of deflection, can be used as an indicator of pavement strength. This indicator then becomes an independent

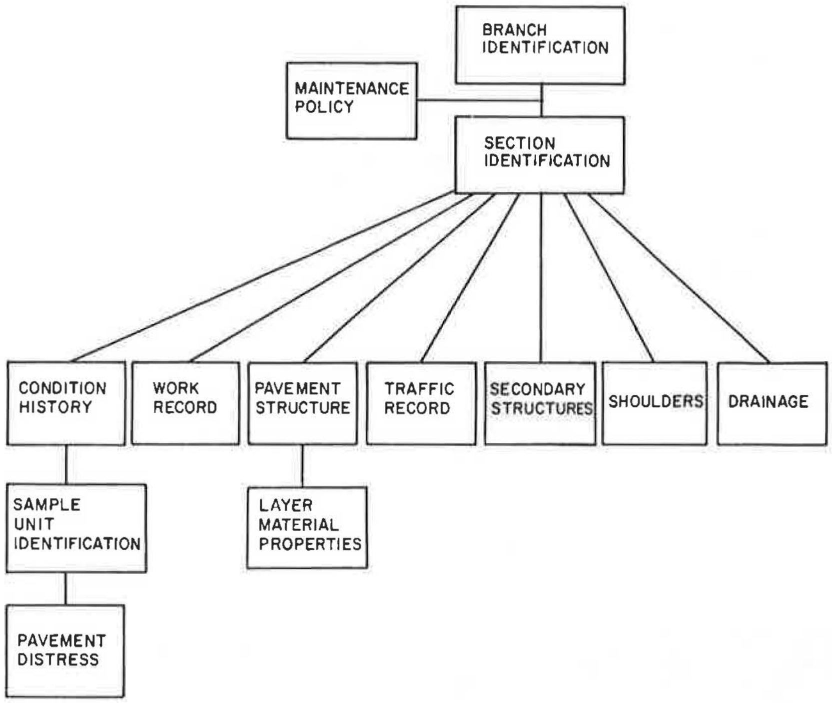

FIGURE 1 Existing PAVER data base logic structure.

variable for predicting pavement performance (3). Also, a sudden increase in deflection values would indicate imminent failure.

In the following sections of this paper the data base structure for the NDT data, its relation to the other data in the PAVER system, and planned use of the NDT data in conjunction with the existing and newly developed pavement analysis programs are described.

NDT DATA BASE STRUCTURE

In order to describe the NDT data structure, it is necessary to describe the overall PAVER data structure. Figure 1 is a conceptual diagram of the existing PAVER data base. Each box shown in the figure is composed of a group of data elements; these groups can be repeated as necessary to store the information on a pavement network. Thus each box is called a repeating group. Within each repeating group, one or more elements called critical elements are defined. The critical elements serve as the unique address identifiers and allow the user to repeat the data group when the critical elements are changed.

The NDT data structure is designed to fit into the existing data base and provide the user with a great deal of flexibility. The new structure consists of the repeating groups shown in Figure 2. (Note that the data groups delineated by dotted lines are from the original PAVER data base.) As an example of the data contained in a repeating group, the elements of the Device ID group are shown in Figure 3. The element marked with the exclamation point (!) is the critical element of this group. Thus each time the Device ID description (Desc) is changed, a new data set can be entered.

The NDT data groups are structured so that they can accommodate analysts who wish to record specific test location data or those who wish to record only summary data. Any level of detail can be stored, depending on the requirements of the engineer. Following is a description of each of the NDT repeating groups.

Device ID

The data elements in the Device ID group will accommodate pertinent information on any NDT testing device in use today and others under development by the U.S. Army Corps of Engineers. Devices in use

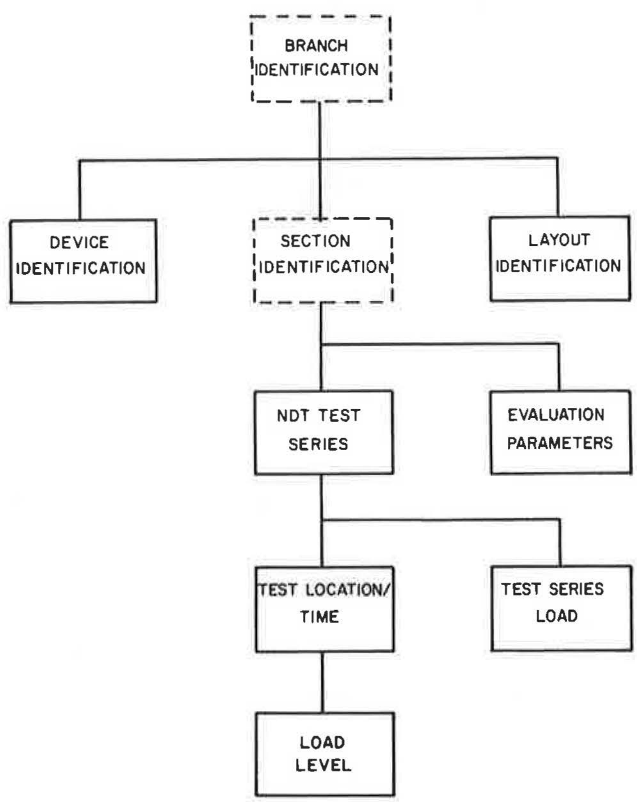

FIGURE 2 NDT data base structure.

```
1200*   DEVICE ID POLICY (RG);
1201*   DEVICE ID DESC (NAME X(5) IN 1200);
1202*   PLATE DIAMETER (DECIMAL NUMBER 9(3).99 IN 1200);
1203*   DIAMETER UNITS (NON-KEY NAME XXX IN 1200);
1204*   NUMBER OF SENSORS (NON-KEY INTEGER NUMBER 99 IN 1200);
1205*   FREQUENCY (DECIMAL NUMBER 9(3).9 IN 1200);
1206*   FREQUENCY UNITS (NON-KEY NAME XXX IN 1200);
1207*   MASS (INTEGER NUMBER 9(6) IN 1200);
1208*   MASS UNITS (NON-KEY NAME XXX IN 1200);
1209*   LOADED AREA (DECIMAL NUMBER 9(6).9 IN 1200);
1210*   AREA UNITS (NON-KEY NAME XXX IN 1200);
1211*   VEHICLE SPEED (INTEGER NUMBER 999 IN 1200);
1212*   SPEED UNITS (NON-KEY NAME X(3) IN 1200);
```

FIGURE 3 Device ID data group.

today can be grouped into three types based on the type of load applied: static, vibratory, and impulse.

The Benkelman beam (4) is an example of a device used for measuring deflection under static loading. The road rater (4) is an example of the vibratory loading type where a sinusoidal force is applied to the pavement through a loading plate. This is usually achieved by applying a dynamic force on a static mass existing on the plate (Figure 4). Therefore, data elements have been defined for storage of plate diameter, mass, and frequency. The falling weight deflectometer (FWD) is an example of the impulse loading type, where a weight is dropped onto a loading plate (Figure 5). The U.S. Army Corps of Engineers and Purdue University have been working on the development of a noncontact NDT deflection and profile measuring system that uses lasers (5) mounted on the side of a load vehicle. For this device, two data elements have been defined for vehicle speed and loaded area.

FIGURE 4 Road rater vibratory load NDT device.

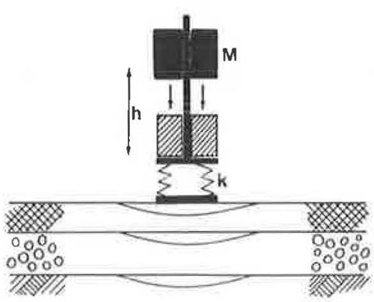

FIGURE 5 Schematic diagram of the FWD.

Sensor Layout ID

The Layout ID group provides detailed information about deflection measuring sensors. Because a particular device may have more than one sensor layout or configuration, a separate repeating group is provided. The Layout ID group has data elements (Sensor Distance and Sensor Offset) to describe the location of up to seven sensors in two dimensions. (Seven is the largest number of sensors available on commercial NDT devices.) This is also a practical limit for characterizing layer material properties based on a deflection basin profile. The Layout ID group also has a data element called Loaded/Unloaded for each sensor. This element is of great significance when testing jointed concrete pavements. It is used to indicate whether a sensor is on the same pavement slab as the load plate. Such information is essential for testing load transfer across transverse and longitudinal joints in concrete pavements.

Several sensor layout patterns may be defined and stored. For example, a typical sensor layout for testing asphalt concrete pavements is a linear arrangement with sensors set at 12-in. intervals, whereas the sensor layout for testing load transfer at a concrete pavement joint may be just two sensors (one each 6 in. from the joint on adjacent slabs).

The position of the Device ID and sensor Layout ID repeating groups in the data base structure allows them to be stored in the most efficient manner. Information on the various devices and sensor layouts used to test all of the pavements in a network needs only be stored once.

NDT Test Series

The NDT Test Series repeating group is used to store summary information for a particular test series. A series is defined as a group of tests on various locations in the pavement section that are considered to be of the same population. This group has average or representative values for a given pavement section.

The data elements in the NDT Test Series group (Figure 6) can be divided into three general subgroups. The first subgroup consists of the basic test series information. The pavement section as well as the device and sensor layout are specified. The NDT Test Series is also identified and described.

Figure 7 is an example of two test series for an asphalt highway pavement section (plan). Figure 8 is an example of four test series for a jointed concrete runway pavement section (plan). The total number of test locations for each series as well as the test interval (average distance between test locations) may also be recorded. Such information provides an indication of the adequacy of coverage of the tests as well as the reliability of the representative values for the test series.

The second subgroup of data elements in the NDT Test Series group consists of representative or mean weather conditions found during testing. The air and pavement temperatures are important for normalizing test deflections recorded in all types of weather

```
3200*   NDT TEST SERIES (RG IN 1000);
3201*   !DEVICE ID (NAME X(5) IN 3200);
3202*   !LAYOUT ID (NAME X(5) IN 3200);
  3203*  ! NDT TEST DATE (DATE IN 3200);
  3204*  ! TEST SERIES (NAME X(10) IN 3200);
  3205*    TEST SERIES DESCRIPTION (NON-KEY NAME X(15) IN 3200);
  3206*    NUMBER OF TESTS (INTEGER NUMBER 99 IN 3200);
  3207*    TEST INTERVAL (DECIMAL NUMBER 9(4).9 IN 3200);
  3208*    INTERVAL UNITS (NON-KEY NAME XX IN 3200);
  3209*    REP AIR TEMP (DECIMAL NUMBER 9(3).9 IN 3200);
  3210*    TEMP UNITS (NON-KEY NAME X IN 3200);
  3211*    REP PAVEMENT TEMP (DECIMAL NUMBER 9(3).9 IN 3200);
  3212*    TEST COMMENTS (NAME X(44) IN 3200);
  3213*    REP SERIES LOAD (INTEGER NUMBER 9(5) IN 3200);
  3214*    REP SERIES LOAD UNITS (NON-KEY NAME XX IN 3200);
  3215*    REP CORRECTED DSM (INTEGER NUMBER 9(5) IN 3200);
  3216*    DSM UNITS (NON-KEY NAME X(3) IN 3200);
  3217*    REP CORRECTED DSM STD DEV (DECIMAL NUMBER 9999.9 IN 3200);
  3218*    REP CORRECTED LONG LOAD TRANSFER (INTEGER NUMBER 9(3) IN 3200);
  3219*    REP CORRECTED LONG LD TRANS STD DEV
           (NON-KEY DECIMAL NUMBER 9(2).9 IN 3200);
  3220*    REP CORRECTED TRANS LOAD TRANSFER (INTEGER NUMBER 9(3) IN 3200);
  3221*    REP CORRECTED TRANS LD TRANS STD DEV
           (NON-KEY DECIMAL NUMBER 9(2).9 IN 3200);
  3223*    REP BASIN CHARACTERISTIC 1 (DECIMAL NUMBER 9(3).9 IN 3200);
  3224*    REP BASIN CHARACTERISTIC 2 (DECIMAL NUMBER 9(3).9 IN 3200);
  3225*    REP BASIN CHARACTERISTIC 3 (NON-KEY DECIMAL NUMBER 9(3).9 IN 3200);
  3226*    REP BASIN CHARACTERISTIC 4 (NON-KEY DECIMAL NUMBER 9(3).9 IN 3200);
  3227*    REP SERIES DEFL UNITS (NON-KEY NAME XX IN 3200);
  3228*    REP SERIES DEFL 1 (DECIMAL NUMBER 9(3).9 IN 3200);
  3229*    REP SERIES DEFL 2 (DECIMAL NUMBER 9(3).9 IN 3200);
  3230*    REP SERIES DEFL 3 (DECIMAL NUMBER 9(3).9 IN 3200);
  3231*    REP SERIES DEFL 4 (DECIMAL NUMBER 9(3).9 IN 3200);
  3232*    REP SERIES DEFL 5 (DECIMAL NUMBER 9(3).9 IN 3200);
  3233*    REP SERIES DEFL 6 (DECIMAL NUMBER 9(3).9 IN 3200);
  3234*    REP SERIES DEFL 7 (DECIMAL NUMBER 9(3).9 IN 3200);
  3235*    REP SERIES DEFL 1 STD DEV (DECIMAL NUMBER 99.9 IN 3200);
  3250*    TEST-CONCAT (NAME X(38) IN 3200);
```

FIGURE 6 NDT Test Series data group.

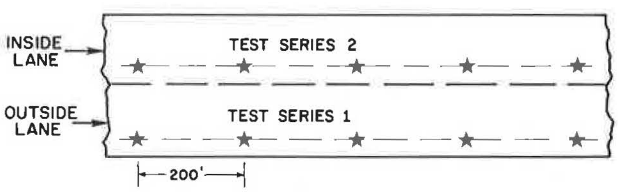

FIGURE 7 Example of test series for a two-lane highway asphalt concrete section.

conditions. Also, the Test Comments data element can be used to record other relevant information.

The third and largest subgroup of data are representative information of the test series in terms of load level, corresponding deflections, and other computed parameters such as the dynamic stiffness modulus (DSM) (6). DSM (Figure 9) is the slope of the straight line portion of load versus deflection and is a required input to the Corps of Engineers airfield and highway pavement evaluation procedures. Load transfer across transverse and longitudinal joints can also be stored. Such data are especially important for design and evaluation of concrete pavements. Data elements are also provided for recording the standard deviation of key section characteristics such as DSM, load transfer, and Sensor 1 deflection. The standard deviations provide valuable information on section variability.

The flexibility of the data base is exemplified further by the four Basin Characteristic elements. These are not specified explicitly, but can be used for such items as deflection basin areas or slopes. Because there are no universally accepted deflection basin parameters, the user is free to choose the ones that prove to be the most useful for his evaluation procedure. Similarly, data elements are provided to store units of measure.

All data elements in the NDT Test Series group can be repeated for each different combination of NDT Test Data and Test Series data elements.

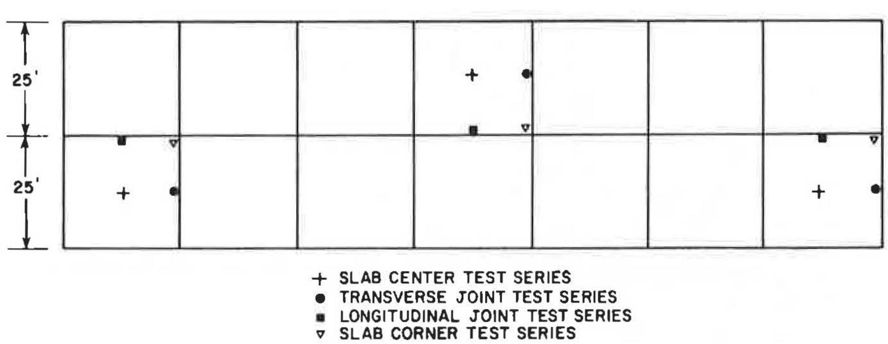

FIGURE 8 Example of test series for jointed portland cement concrete runway pavement section.

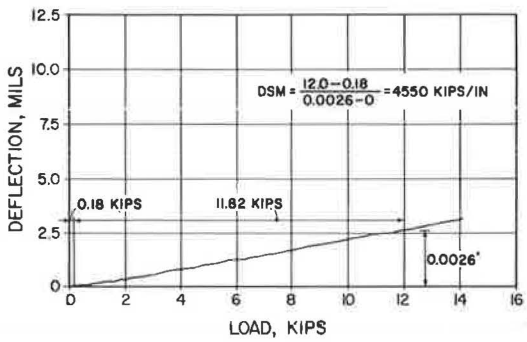

FIGURE 9 Example DSM calculation.

Test Series Load

The Test Series Load group is a subset of the NDT Test Series group. It allows for storage of deflection data for various load levels. Thus an unlimited number of representative series loads and deflections can be stored for future analysis.

Test Location/Time

The Test Location/Time repeating group is also a subset of the NDT Test Series group. It consists of data elements that describe test results for a particular test location and time of testing. The location information consists of the test location station and offset. The Time of Test is a critical data element in this data group. It is used to distinguish between test results of the same location or station but tested at different times. This is important when using a specific test location as a reference point to establish a temperature correction relationship for a given test series. Because the pavement temperature varies over the course of a day, the NDT deflections are also likely to vary, as shown in Figure 10. Thus a temperature-deflection relationship should be established.

The data elements for the Test Location/Time group are shown in Figure 11. Both the air temperature and the pavement surface temperature may be recorded. Parameters computed from deflection data such as DSM, Load Transfer, and Basin Characteristic 1 through 4 may also be stored at this level.

All data elements in the Test Location/Time group can be repeated for each different Time of Test data element.

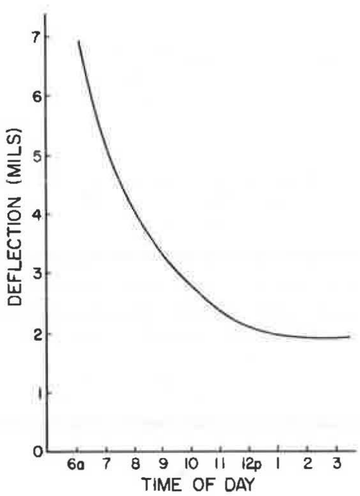

FIGURE 10 Example deflection change with time of day (or temperature) for a concrete slab corner.

Load Level

The Load Level repeating group is a subset of the Test Location/Time group and represents the last level in the data base structure. This group consists of the actual test values for a particular load and its resulting deflections. The data elements are shown in Figure 12. Once again, there are seven data elements, labeled Defl 1 through Defl 7, available to record deflection values. Also, at the end of this group there is space for comments.

EVALUATION PARAMETERS DATA GROUP

The Evaluation Parameters data group is a direct subset of Section Identification (i.e., information in this group can be stored regardless of whether NDT data are stored or not). The data elements of this group are shown in Figure 13. The information in this group can be repeated for any different combination of the following data elements: Evaluation Date, Design Vehicle, Design Load, and Design Passes. The group is designed, as for the rest of the PAVER system, for use for both highway and airfield pavements. The following example from an evaluation of a U.S. Army airfield is provided for illustration purposes. For a given pavement section with a design aircraft C141, a design load of 323 kips, and design passes of 20,000, the evaluation parameters given in Table 1 were computed (7). As can be seen in Figure 13, a data element has been defined for the storage of each of the determined evaluation parameters.

```
3300*   TEST LOCATION/TIME (RG IN 3200);
  3301* ! TIME OF TEST (INTEGER NUMBER 9(4) IN 3300);
  3302*   LOCATION-STATION (NAME X(7) IN 3300);
  3303*   LOCATION-OFFSET (NAME X(10) IN 3300);
  3304*   SURFACE TEMPERATURE (NON-KEY DECIMAL NUMBER 9(3).9 IN 3300);
  3305*   AIR TEMPERATURE (NON-KEY DECIMAL NUMBER 9(3).9 IN 3300);
  3306*   DSM (INTEGER NUMBER 9(5) IN 3300);
  3307*   CORRECTED DSM (INTEGER NUMBER 9(5) IN 3300);
  3308*   LONG LOAD TRANSFER (INTEGER NUMBER 9(3) IN 3300);
  3309*   CORRECTED LONG LOAD TRANSFER (INTEGER NUMBER 9(3) IN 3300);
  3310*   TRANS LOAD TRANSFER (INTEGER NUMBER 9(3) IN 3300);
  3311*   CORRECTED TRANS LOAD TRANSFER (INTEGER NUMBER 9(3) IN 3300);
  3312*   BASIN CHARACTERISTIC 1 (DECIMAL NUMBER 9(3).9 IN 3300);
  3313*   BASIN CHARACTERISTIC 2 (DECIMAL NUMBER 9(3).9 IN 3300);
  3314*   BASIN CHARACTERISTIC 3 (NON-KEY DECIMAL NUMBER 9(3).9 IN 3300);
  3315*   BASIN CHARACTERISTIC 4 (NON-KEY DECIMAL NUMBER 9(3).9 IN 3300);
  3316*   LOCATION COMMENTS  (NON-KEY NAME X(40) IN 3300);
  3350*   TESTLOC-CONCAT (NAME X(43) IN 3300);
```

FIGURE 11 Test Location/Time data group.

```
3700*   LOAD LEVEL (RG IN 3300);
  3701* ! LOAD (INTEGER NUMBER 9(5) IN 3700);
  3704*   DEFL 1 (DECIMAL NUMBER 9(4).9 IN 3700);
  3705*   DEFL 2 (DECIMAL NUMBER 9(4).9 IN 3700);
  3706*   DEFL 3 (DECIMAL NUMBER 9(4).9 IN 3700);
  3707*   DEFL 4 (DECIMAL NUMBER 9(4).9 IN 3700);
  3708*   DEFL 5 (DECIMAL NUMBER 9(4).9 IN 3700);
  3709*   DEFL 6 (DECIMAL NUMBER 9(4).9 IN 3700);
  3710*   DEFL 7 (DECIMAL NUMBER 9(4).9 IN 3700);
  3712*   LOAD COMMENTS (NAME X(40) IN 3700);
```

FIGURE 12 Load Level data group.

The remaining two data elements in this group are only applicable to airfield pavements. They are provided for the storage of the internationally required evaluation parameters known as the Aircraft Classification Number (ACN) and the Pavement Classification Number (PCN). An FAA circular is being printed that describes the determination of these parameters based on allowable aircraft load (8).

OVERALL PAVER DATA BASE STRUCTURE

The PAVER data base structure, including NDT, is shown in Figure 14. It is shown in an inverted tree structure, with data groups located at levels 0 through 4. The amount of detail increases with the increase in level number. For example, at level 0 an entire street is defined, at level 1 each uniform section of the street is defined, and so forth. This is true as long as the groups are linked. Thus for Drainage, no more details can be stored beyond level 2, whereas for Pavement Structure the Layer Material Properties group can be repeated as desired.

There are three groups that are not associated with the Section Identification group: Device ID, Layout ID, and Maintenance Policy. The information in these groups need not be changed among pavement

TABLE 1 Evaluation Parameters

Parameter	Value
Allowable load for 20,000 passes (kips)	229
Allowable passes for 323 kips	30
Asphalt concrete overlay required for 20,000 passes of 323 kips (in.)	5
Portland cement concrete (in.)	
Fully bonded overlay required	–[a]
Partially bonded overlay required	7
Unbonded overlay required	8

[a]Not evaluated.

```
2800*   EVALUATION PARAMETERS (RG IN 1000);
  2801* ! EVALUATION DATE (DATE IN 2800);
  2802* ! DESIGN VEHICLE (NAME X(8) IN 2800);
  2803* ! DESIGN LOAD (NAME X(6) IN 2800);
  2804* ! DESIGN PASSES (INTEGER NUMBER 9(8) IN 2800);
  2805*   ALLOWABLE LOAD ( INTEGER NUMBER 9(6) IN 2800);
  2806*   ALLOWABLE PASSES ( INTEGER NUMBER 9(8) IN 2800);
  2807*   REMAINING LIFE ( INTEGER NUMBER 99 IN 2800);
  2808*   AC OVERLAY REQUIRED ( DECIMAL NUMBER 99.9 IN 2800);
  2809*   PCC-F OVERLAY REQUIRED ( DECIMAL NUMBER 99.9 IN 2800);
  2810*   PCC-P OVERLAY REQUIRED ( DECIMAL NUMBER 99.9 IN 2800);
  2811*   PCC-U OVERLAY REQUIRED ( DECIMAL NUMBER 99.9 IN 2800);
  2812*  ACN (INTEGER NUMBER 999 IN 2800);
  2813*   PCN (INTEGER NUMBER 999 IN 2800);
  2814*   PCN CODE (NAME X(4) IN 2800);
2825*  EVAL-CONCAT (NAME X(40) IN 2800);
```

FIGURE 13 Evaluation Parameters data group.

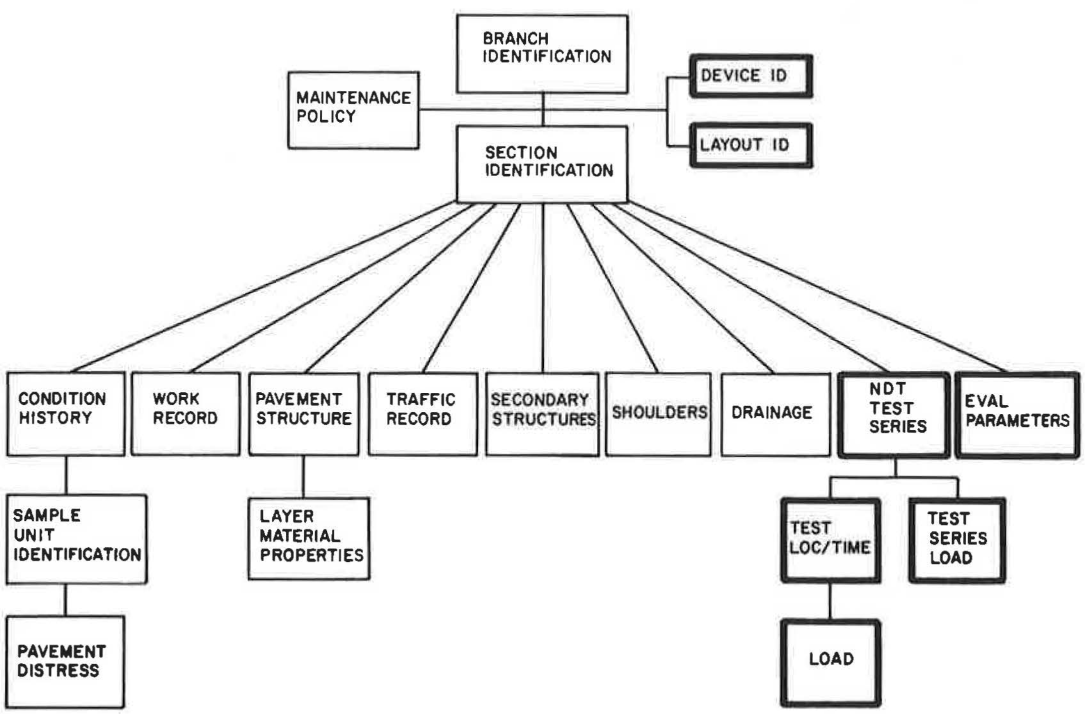

FIGURE 14 New PAVER data base logic structure.

sections; rather it can be used as a reference. Thus the groups are stored under a dummy Branch ID. The groups are also designed so that any number of Device ID or Layout ID groups could be defined.

PLANNED USE OF NDT DATA BASE

The PAVER system is a dynamic system. New developments and improvements are regularly added. Those developments planned that will make use of NDT data include pavement condition forecasting models and pavement structural evaluation routines.

The proposed pavement condition prediction models will be based on the pavement condition index (PCI). PCI is a repeatable index that is a key to the PAVER system. PCI is highly correlated to the level of maintenance required. Thus PCI is an excellent indicator of the amount of money required to maintain a pavement network.

PCI prediction models are based on available relevant variables such as NDT data. These models provide valuable input to both network- and project-level management. At the network level they are used for condition forecasting and budget planning. At the project level they are used to determine the consequence of changes in traffic or the impact of a given maintenance strategy. Structural evaluation and overlay design models that use NDT data are also currently being interfaced with PAVER. Such programs would greatly expedite the calculations required to evaluate every pavement section for several design vehicles. The possible number of design vehicles could be extremely large for airfield pavements.

The planned use of NDT data is not intended to replace the need for qualified pavement engineering expertise, but it will reduce the amount of labor and tedious work involved.

SUMMARY

NDT data groups have been added to the PAVER pavement management system data base. The data were arranged in several repeating groups on four hierarchical levels. The NDT data elements are flexible enough to handle various types of testing devices and patterns. Many levels of data can be stored, from summary network-level information to specific local test information. These data are currently being interfaced with analysis programs for improvement in the evaluation, design, and condition prediction capabilities of the PAVER system.

Although these elements have been designed for the System 2000 data base management system, it is believed that, to have full use of NDT data, a similar set of data elements are needed in any system. (Note that System 2000 is a registered trademark of Intel Corporation.)

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Serviceability Evaluation of a Complete Interstate Highway Network

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ABSTRACT

Pavement management systems (PMSs) are becoming more widely used for the efficient management of highway networks at all levels of government. PMS strategies usually provide management with two levels of information: network level and project level. Network-level information provides broad-based data about the entire system, whereas project-level data are specific about construction information. The present serviceability index (PSI), or roughness of the highway, is becoming a standard tool for network evaluation. In this paper the results of an analysis of the roughness of the Interstate highway network in Pennsylvania are presented. PSI measurements were made on the network for three years (1976, 1978, 1981) and are summarized for the system as a whole, by districts, and by routes. This framework provides a picture of the change in average conditions over time and the variation in average condition from district to district and route to route. A regression analysis was performed on the data to predict conditions on the Interstates during the succeeding 5 years. An analysis was also made of the impact of betterment projects on the serviceability of the system. Results of the study indicate that the weighted mean PSI of the system has declined from 3.74 in 1976 to 3.51 in 1981. The average rate of decline annually was 0.04 between 1976 and 1978, and 0.05 between 1978 and 1981. An increase in the standard deviation suggests that the variability in conditions is increasing across the state.

Pavement management systems (PMSs) are becoming more widely used for efficient management of highway networks at all levels of government. Regardless of the size of the highway network or the sophistication of the PMS procedure, most PMS strategies can offer assistance at two levels: the network level and the project level (1,2). Network-level strategies are usually designed to provide management with broad views of the highway system as a whole. Information for planning purposes and fiscal analysis is often provided by the network analysis. On the other hand, project-level PMS information can include specific details about engineering design, construction management, and cost accounting. Obviously, the data required for each level differ considerably.

Data for a project-level PMS are unique and specific for each individual highway section. Material properties, layer thicknesses, deflection measurements, and condition surveys are some of the data collected at this level. This type of information usually requires considerable time to assemble. Conversely, at the network level information must be gathered quickly to provide a "snapshot" view of the entire system within a reasonable amount of time. Several PMS concepts use pavement serviceability (roughness) as the primary tool for network analysis (3,4). Pavement roughness is a variable that can be measured quite rapidly with consistent results. This is especially important for agencies that have large highway networks.

SCOPE OF STUDY

The study described in this paper was conducted for the Pennsylvania Department of Transportation. The objective of the research was to demonstrate the feasibility of computer-aided graphical mapping of the present serviceability index (PSI) for the entire Interstate highway network in Pennsylvania (5). Before this study, PSI values were hand-plotted on the Interstate maps by department personnel. This proved to be a time-consuming job, and thus it was desirable to computerize the process.

To plot each segment of Interstate PSI and represent it with a color code on the map, the Interstate system had to be digitized with a PSI value assigned to each highway segment. This type of data provided an ideal opportunity to summarize the PSI values for the entire Pennsylvania Interstate system. The PSI statistical information for the entire network is summarized herein.

The PSI data for three years (1976, 1978, and 1981) are summarized for the system as a whole, by district, and by route. Such a framework provides a picture of the change in average conditions over time and of the variation in average conditions from district to district and from route to route. A regression analysis was performed to predict conditions on the Interstates during the next 5 years, and an analysis was made of the impact of betterment projects on average conditions.

MEASURING PSI

Roughness measurements have been made by the Pennsylvania Department of Transportation since the mid-1960s. Since that time the department has used several devices for the measurement of roughness. The first device used was the Bureau of Public Roads (BPR) road roughness indicator. It was used by the department from 1965 to 1967. In 1967 the department evaluated a Portland Cement Association (PCA) road meter, which was referred to as an Autoflect. The Autoflect was used for roughness measurements until 1972. In 1972 the department evaluated and subsequently adopted the Mays ride meter as the device to measure pavement roughness in Pennsylvania. It should also be noted that the department purchased a GMR Surface Dynamics profilometer and quarter-car simulator in 1968.

In the mid-1970s the department undertook an extensive evaluation of its pavement roughness measurement program (6). At that time the entire highway system was classified according to maintenance functional classes. Also, a study was conducted to provide a relationship between pavement roughness as measured with the department's Mays meters and the original AASHO Road Test concept of PSI determined by a rating panel. The study resulted in the development of the following mathematical relationships. For a rigid pavement,

$$PSI = (11.10) - 3.67(\log RF) - 0.09\sqrt{C + P} \quad (1)$$

and for a flexible pavement,

$$PSI = (11.33) - 4.06(\log RF) - 0.01\sqrt{C + P} - 1.34\,\overline{RD}^2 \quad (2)$$

where

PSI = present serviceability index (0 to 5),
RF = Mays meter roughness factor (in./mile),
C = cracks (ft/1,000 ft^2),
P = patches (ft^2/1,000 ft^2), and
RD = average rut depth (in.).

Because the department considered the measurement of rutting and cracking and patching to be too time-consuming in terms of the size of the system (45,000 miles), an evaluation was made to determine the effect of these parameters on the PSI value. It was determined that, over the range of roughness measured, the reduction in PSI caused by rutting and cracking averaged 0.37 for rigid pavements and 0.17 for flexible pavements. Consequently, the relationships for PSI as measured by the Mays meter in Pennsylvania are, for rigid pavements,

$$PSI = (10.73) - 3.67 (\log RF) \quad (3)$$

and for flexible pavements,

$$PSI = (11.16) - 4.06 (\log RF) \quad (4)$$

These equations have been used by the department since 1975 to convert the Mays meter roughness value (in./mile) to a PSI value. All roughness measurements discussed in this paper were measured with the department's Mays meter and converted to PSI.

An important question to consider when measuring roughness is the calibration of the measuring device. Changes in vehicle characteristics (shocks, springs, and so forth) will change the measured response rather than indicate an actual change in pavement roughness. To prevent this, the department calibrates its Mays meters by driving them over test sections of pavement and comparing the output with the GMR profilometer response. This ensures that a consistently calibrated Mays meter was used for testing purposes over the years of the study described in this paper.

STATEWIDE PRESENT SERVICEABILITY CONDITIONS

The average condition of the Interstate highway system on a statewide basis is summarized in Table 1. The weighted mean PSI value of the system declined from 3.74 in 1976 to 3.51 in 1981. A weighted mean was necessary to account for differing lengths of PSI measurements. The average rate of decline annually was 0.04 between 1976 and 1978, and 0.05 between 1978 and 1981. The increase in standard deviation from 1976 to 1981 suggests that the variability in condition across the state is increasing. It should be pointed out that the entire Interstate system was measured. This provides information about the complete population of the data set. Consequently, it is not necessary to make statistical conclusions as if only a sample of data was available. The values given in Table 1 indicate that the system is in fact becoming more rough with time, which will directly result in a systemwide decrease in the PSI value.

The deterioration in condition and the increase in variability of PSI values are shown in Figure 1. The distribution of miles of Interstate highway by PSI value resembles a bell-shaped normal distribution. (Tests for normality are described in the following paragraphs.) A shift to the left in the curves from 1976 to 1981 demonstrates the decline in mean PSI values; the increasing breadth and the decreasing height of the curves demonstrate the increase in variability.

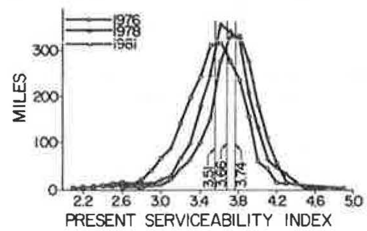

FIGURE 1 Distribution of PSI values for all Interstate highways in Pennsylvania in 1976, 1978, and 1981.

The data in Table 2 give another measure of the deterioration in the condition of the Interstates: the miles of Interstate highway below Pennsylvania's minimum acceptable PSI value of 3.3. The number of miles of Interstate highway below a PSI value of 3.3 has more than quadrupled from 1976 to 1981, when more than 18 percent of the system was below the minimum acceptable value.

TABLE 2 Miles of Pennsylvania Interstate with PSI Values Less than 3.3, Statewide

Year	Miles Less than 3.3	Percentage Less than 3.3
1976	82.0	4.2
1978	155.1	7.5
1981	395.1	18.5

TABLE 1 Mean PSI Values on Pennsylvania Interstate System Network

Year	Miles Measured[a]	Weighted Mean PSI	Standard Deviation
1976	1,968.7	3.74	0.271
1978	2,074.8	3.66	0.332
1981	2,136.0	3.51	0.337

[a]Route miles in both directions.

Two tests for normality were performed on the PSI data because normality was an inherent assumption of the regression analysis (described later) and because statistical descriptions would be simplified in the event the data were normally distributed. The primary test was a Minitab (7) procedure that involved three steps: (a) calculating normal scores for the data, (b) producing a normal probability plot, and (c) measuring the straightness of the plot

TABLE 3 PSI Values by District, Interstate System

	1976				1978				1981			
District	Mean PSI	Miles Measured	Miles Less than 3.3	% Less than 3.3	Mean PSI	Miles Measured	Miles Less than 3.3	% Less than 3.3	Mean PSI	Miles Measured	Miles Less than 3.3	% Less than 3.3
1	3.81	276.0	4.0	1	3.78	332.4	11.0	3	3.62	332.4	36.3	10
2	3.75	192.0	0.0	0	3.67	196.0	4.0	2	3.45	188.0	33.0	17
3	3.65	106.0	5.3	4	3.62	106.0	5.0	4	3.49	106.0	8.0	7
4	3.69	349.7	10.0	2	3.62	368.4	12.7	3	3.47	371.1	63.8	17
5	3.62	254.6	22.8	8	3.60	289.6	34.1	11	3.46	296.6	66.3	22
6	3.64	82.0	9.2	11	3.50	129.2	18.1	14	3.35	141.2	46.4	32
8	3.86	310.8	5.0	1	3.78	315.6	2.4	0	3.67	318.6	13.5	4
9	3.58	46.7	7.1	15	3.55	48.1	6.9	14	3.48	48.1	7.5	15
10	3.77	160.0	8.0	4	3.65	162.0	13.6	8	3.42	160.0	53.0	33
11	3.71	52.0	1.0	1	3.76	55.0	0.0	0	3.58	69.0	6.0	8
12	3.76	138.9	9.6	6	3.49	176.4	47.3	26	3.49	209.4	61.3	29

by using a correlation coefficient. (A straight-line plot with a high correlation coefficient is consistent with normality.) The normal probability plot indicated some curvature, although the correlation coefficients for each year were fairly high (0.992, 0.974, and 0.986). These results suggested that although there was a slight deviation from normality, it could be assumed that there is a normal distribution of the data. A second test, the SAS Univariate procedure (8), was used to measure two other aspects of the distribution. The results revealed that there was some skew in the data, indicated by the larger tail to the left of the mean, which accounted for some of the deviation from normality. A second measurement was taken for kurtosis, which measures the flatness or steepness of a curve. This indicated that the distribution was slightly flatter than a perfectly normal distribution.

These results suggest that, by making a normal approximation for the distribution of PSI values, the proportion of mileage less than the mean or less than the minimum acceptable PSI value would be a conservative estimate.

CONDITIONS BY ENGINEERING DISTRICT

The average condition of the Interstates by district was considered to determine whether variations existed from district to district. The data in Table 3 summarize the PSI data for each engineering district. Considerable variations can be observed. Three measures are suggested to evaluate the difference in conditions between districts.

1. The first measure is the mean PSI value, which ranged in 1981 from a low of 3.35 in District 6 to a high of 3.67 in District 8.
2. The second measure is the number of miles and the percentage of Interstate highways with PSI values less than 3.3. Districts 4, 5, and 12 each had more than 60 miles with PSI values less than 3.3, representing 17, 22, and 29 percent, respectively, of their total Interstate mileage. In both Districts 6 and 10 almost one-third of the Interstates had PSI values less than 3.3. In contrast, in Districts 3, 8, and 11 less than 10 percent of the Interstates were deficient.
3. A final measure of the differences by district is the rate of decline in mean PSI values from 1976 to 1981. The greatest decline occurred in District 10, where the mean PSI value dropped by 0.35, compared with a decline of 0.23 in the statewide mean PSI value (Table 1). Deterioration was also greater than the statewide average in Districts 2, 6, and 12. In the other districts the rate of decline was less than the statewide average, with the smallest decline (0.10) occurring in District 9.

Several factors that might explain the differences among districts are suggested here, although a thorough investigation is beyond the scope of this paper. One factor is the age of the Interstates in each district. The average year of completion of the Interstates for each district, compiled from the Pennsylvania Department of Transportation's Road Log, is given in Table 4. (In respect to pavement life, age is more accurately measured by the amount of heavy truck traffic carried than by the number of years since the completion of the highway.) The quality of initial construction may also be a variable. Other factors would include not only the level of funding for Interstate maintenance and resurfac-

TABLE 4 Average Year of Interstate Completion by District

District	Avg Year of Completion
1	1964
2	1968
3	1965
4	1965
5	1962
6	1963
8	1962
9	1961
10	1965
11	1959
12	1964

TABLE 5 PSI Values by Route

	1976				1978				1981			
Route	Mean PSI	Miles Measured	Miles Less than 3.3	% Less than 3.3	Mean PSI	Miles Measured	Miles Less than 3.3	% Less than 3.3	Mean PSI	Miles Measured	Miles Less than 3.3	% Less than 3.3
70	3.55	90.6	15.7	17	3.27	130.5	54.2	42	3.38	163.5	65.6	40
76	0.00	0.0	0.0	0	3.43	42.0	9.0	21	3.16	46.0	27.1	59
78	3.60	65.9	7.0	11	3.71	95.9	8.0	8	3.50	95.9	22.1	23
79	3.80	355.0	2.0	1	3.76	357.5	0.0	0	3.63	358.0	12.3	3
80	3.66	606.8	30.7	5	3.58	622.8	58.5	9	3.42	612.8	171.3	28
81	3.79	464.0	8.4	2	3.71	464.9	2.7	1	3.56	466.0	40.3	9
83	3.87	102.0	3.0	3	3.70	100.0	2.4	2	3.61	102.0	7.5	7
84	3.75	90.4	3.0	3	3.72	104.2	3.0	3	3.62	104.8	3.3	3
90	4.26	38.0	0.0	0	4.04	92.4	4.0	4	3.67	92.4	19.7	21
95	3.64	82.0	9.2	11	3.54	85.2	8.1	10	3.46	93.2	17.3	19
176	3.87	22.0	0.0	0	3.79	22.0	1.2	5	3.65	22.6	0.6	3
279	0.00	0.0	0.0	0	0.00	0.0	0.0	0	3.47	13.0	3.0	23
283	0.00	0.0	0.0	0	3.86	5.8	0.0	0	3.71	5.8	0.0	0
378	0.00	0.0	0.0	0	0.00	0.0	0.0	0	3.57	6.4	0.4	6
380	3.65	52.0	3.0	6	3.62	54.0	3.0	6	3.58	56.0	2.6	5
676	0.00	0.0	0.0	0	3.30	2.0	1.0	50	2.90	2.0	2.0	100

TABLE 6 Average Year of Interstate Completion by Route

Route	Avg Year of Completion
70	1961
76	1951
78	1958
79	1966
80	1966
81	1965
83	1958
84	1973
90	1959
95	1968
176	1958
283	1964
378	1961
380	1966
676	1959

ing, rehabilitation, and reconstruction (3R) work, but also the degree of effectiveness in the use of those funds. Finally, environmental conditions are likely to vary from one district to another.

CONDITIONS BY INTERSTATE ROUTE

Statistics describing average conditions by Interstate route were developed to determine the extent to which conditions varied from route to route. These statistics are summarized in Table 5. For reference, the average year of completion of the Interstates is given in Table 6.

Excluding spur routes, mean PSI values in 1981 ranged from a low of 3.16 for I-76 to a high of 3.67 for I-90. Several Interstates had considerable mileage with PSI values less than 3.3, whereas others had very little. For example, 171.3 miles (28 percent) of I-80 were deficient in 1981, as were 59 percent of I-76 and 40 percent of I-70. On the other hand, less than 10 percent of I-79, I-81, I-83, and I-84 were deficient. Such statistics can help determine which Interstates are in the poorest condition.

CONDITIONS BY INTERSTATE ROUTE AND DISTRICT

Another level at which the variability of the conditions of the Interstates can be evaluated is the individual routes by district, because certain routes (I-70, I-78, I-79, I-80, I-81, and I-380) run through several districts. For example, I-80 runs through six engineering districts, and there might be interest in whether the condition of I-80 in District 1 is better or worse than in District 5. Variability at this level was evaluated by summarizing,

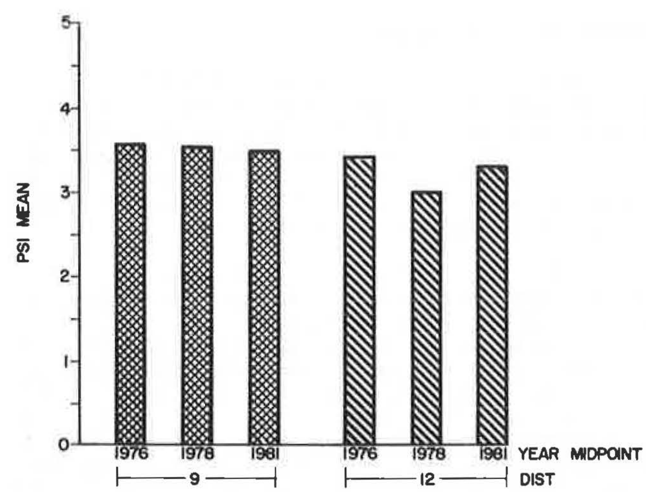

FIGURE 2 Weighted mean PSI values, I-70.

in a histogram, the mean PSI value of the route in each district through which it runs. These histograms are shown in Figures 2-7. Comparisons among districts can be made in terms of both the relative mean PSI values in a given year and the relative rate of change in conditions between 1976 and 1981.

Inspection of the histograms suggests that, for an individual route, conditions do not vary a great deal from one district to another. Some variability is observed, however, in the relative rate of change in conditions. For example, on I-80 the mean PSI value declined at a much faster rate in District 10 than it did in District 1. Comparisons among routes and districts can provide valuable information to decision makers in identifying problem areas and in determining how to allocate funds.

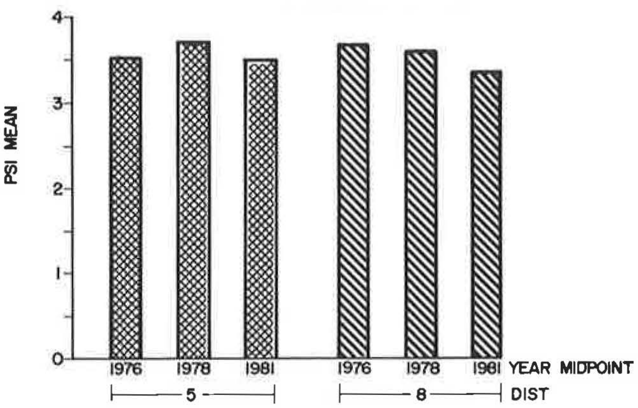

FIGURE 3 Weighted mean PSI values, I-78.

PREDICTION OF FUTURE CONDITIONS

A regression analysis of the Interstate PSI data was performed as a means of predicting future PSI values. The analysis used the statewide weighted mean PSI values for 1976, 1978, and 1981 (presented earlier), rather than the values for each Interstate section. The objective was to estimate average conditions for the succeeding 5 years, based on the assumption that the historical trends of statewide mean PSI values would continue.

Because only three data points were available and additional points were desirable, a fourth point was estimated that represented the PSI value of the Interstates when they were new. The average year of completion of all Interstates in Pennsylvania, based on data compiled from the department's Road Log, was 1964. Therefore, the fourth point used in the regression analysis was a mean PSI value for new pavements in 1964. The average PSI value for new highways in Pennsylvania is approximately 4.2.

Two regression models were tested: a linear model with the mean PSI value as a function of the number of years after construction (i.e., AGE [MEAN PSI = a + b (AGE)]) and a power function [MEAN PSI = a(AGE)^{b}]. A power function was tested because AASHO Road Test data suggested that such a function best described the relationship between PSI and accumu-

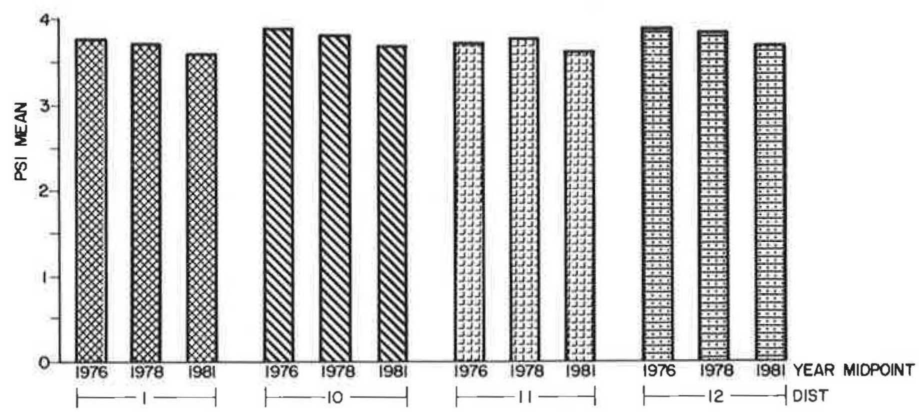

FIGURE 4 Weighted mean PSI values, I-79.

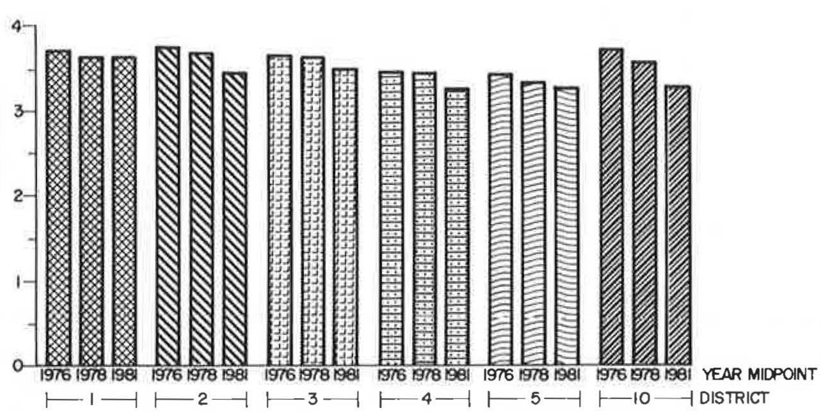

FIGURE 5 Weighted mean PSI values, I-80.

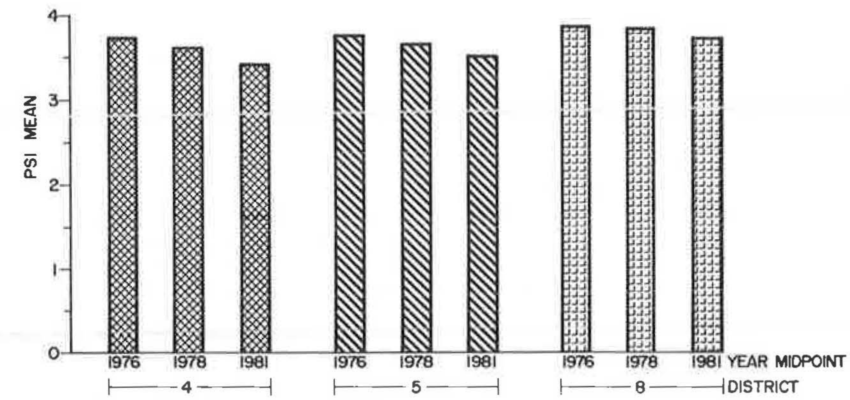

FIGURE 6 Weighted mean PSI values, I-81.

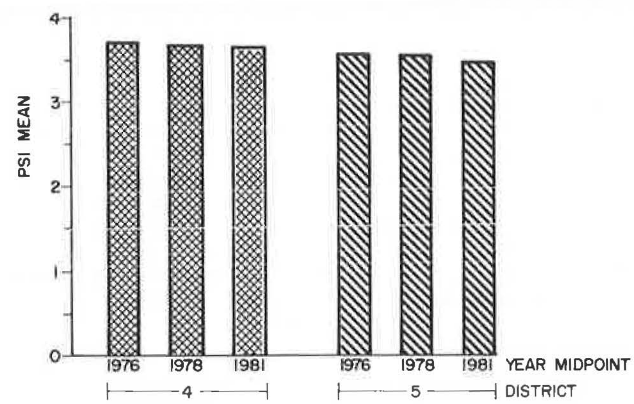

FIGURE 7 Weighted mean PSI values, I-380.

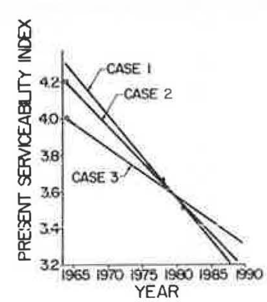

FIGURE 8 Linear-regression models for mean PSI as a function of year.

lated axle loads. The power function suggested by AASHO is

$$C_o - P = K_w{}^B \tag{5}$$

where

C_o = initial PSI,
P = PSI at time τ,
K = constant,
W = equivalent axle load (EAL) application with time, and
B = positive power.

This equation describes the relationship between the change in PSI and accumulated axle loads. The regression performed with Interstate data provides for the empirical relationship between PSI and years after initial construction. These two models were tested for each of three cases. Case 1 involved the use of the weighted mean values for PSI measurements in 1976, 1978, and 1981. Case 2 included a fourth point--a mean PSI value of 4.2 for new pavements in 1964. Case 3 used a mean PSI value of 4.0 for new pavements in 1964.

In each case the linear model fitted the data better than the power function did. A plot of the data points (Figure 8) shows some curvature, but the results suggest that the curvature is not significant enough to be modeled better by a power function.

The results from the regression analysis are given in Table 7. These results suggest that if the Interstate system in Pennsylvania continues to decline in the future at the same rate it has in the past, the statewide mean PSI value will drop to between 3.24 and 3.39 by 1987. This prediction means that by 1987 almost one-half of the Interstate system will be below the acceptable PSI level of 3.3. This would represent a 32 percent increase in efficient highways. It should be noted that these mean PSI values are for all sections of Interstates in Pennsylvania and include sections that have not been rehabilitated as well as sections that have. This point raises a question that will be addressed in the next section, What has been the impact of betterment projects on the decline in statewide mean PSI values?

ANALYSIS OF BETTERMENT PROJECTS ON THE INTERSTATE SYSTEM

Considerable effort and significant amounts of money have been expended on betterment projects on the Interstate system in Pennsylvania. It would be expected that these betterments have had a positive impact on the average condition of the Interstates, either reversing the decline in mean PSI value or reducing it. Statewide mean PSI values presented earlier, however, indicate that the mean PSI value declined by 0.04 per year between 1976 and 1978 and by 0.05 per year between 1978 and 1981. This raises the question whether the PSI measurements and mean PSI values are accurate, and, if so, why the betterments have not had their intended effect.

A partial answer to these questions can be found in Table 8, which indicates that a significant number of miles of betterment projects did not exist until 1980. (Data were not available on the sections

TABLE 7 Summary of Linear-Regression Models for Estimating Weighted Mean PSI Values

Case	PSI Value in 1964	Regression Equation[a]	R^2	Estimated PSI Value in 1987
Power Function				
1	--	Log (MEAN PSI) = 0.772 - 0.184 LOG(AGE)	97.3	3.32
2	4.2	Log (MEAN PSI) = 0.625 - 0.0559 LOG (AGE)	91.2	3.54
3	4.0	Log (MEAN PSI) = 0.604 - 0.0377 LOG (AGE)	72.5	3.57
Linear Function				
1	--	Mean PSI = 4.30 - 0.0463 [AGE]	99.3	3.24
2	4.2	Mean PSI = 4.25 - 0.0426 [AGE]	99.8	3.27
3	4.0	Mean PSI = 4.05 - 0.0289 [AGE]	94.9	3.39

[a]The coefficient for the [AGE] term can be interpreted as the rate of decline in statewide mean PSI value per year.

TABLE 8 Miles of Betterments on Interstate System in Pennsylvania per Year

Year	Miles of Betterments
1978	11.7
1979	11.0
1980	99.9
1981	176.4
1982	201.8

on which betterments were performed before 1978.) Further, an analysis of the sections on which betterments occurred indicates that the betterments in 1978 and 1981 were performed after PSI measurements had been taken. Therefore, only the mean PSI value for 1981 would reflect the impact of betterments, and then only the betterments performed in 1978, 1979, and 1980. This suggests that the PSI measurements and mean values are reasonably accurate.

The finding that the mean PSI value declined at a faster rate between 1978 and 1981 (during which betterments occurred) than between 1976 and 1978 (during which no betterments occurred) appears incongruous. A more valid comparison can be made between the observed rate of decline from 1978 to 1981 with betterments versus what the rate would have been without those betterments. To make this comparison, those sections on which betterments occurred before 1981 were sorted out, and a mean PSI value was calculated for only those sections on which no betterment occurred. The mean PSI value for sections without betterments was lower than the mean PSI value for all sections by less than 0.01. This suggests that there were so few miles of betterments between 1978 and 1981, compared with the total mileage of the Interstate system, that their impact, while discernable, was virtually negligible. A much larger number of miles were improved in 1981 and 1982, and it would be useful to identify their impact on the statewide mean of the next set of PSI measurements, scheduled to be taken in 1983, and on the predicted statewide mean PSI value for 1987.

Another analysis was designed to evaluate whether the sections on which betterments occurred were logical selections from the standpoint of their PSI values. The total mileage of betterment projects was obtained from department records. Figures 9 and 10 are examples of the analysis performed on sections with betterments in 1981 and 1982, respectively. The distributions of PSI values in 1981 indicate that a significant number of betterment projects (211.8 out of 378.2 miles) were performed on sections of Interstates with PSI values greater than 3.3. A comparison of this information with the number of miles of Interstates with PSI values less than 3.3 in 1981 (Table 2) suggests that betterments were not necessarily performed on the sections with the lowest PSI values. Although 395 miles (Table 2) of Interstates had PSI values less than 3.3 in 1981, only 109 out

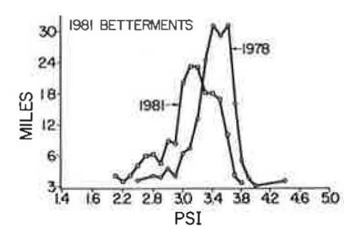

FIGURE 9 Distribution of PSI values in 1978 and 1981 for sections with betterments in 1981.

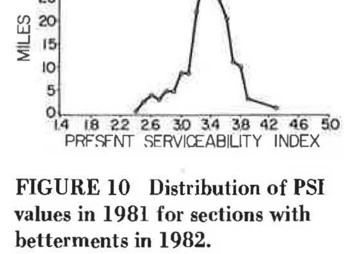

FIGURE 10 Distribution of PSI values in 1981 for sections with betterments in 1982.

of 176 miles (Table 8) of betterments in 1981 and 58 out of 202 miles (Table 8) of betterments in 1982 had PSI values less than 3.3.

Obviously, other factors come into play when determining how to allocate funds for betterments on the Interstates; for example, a significant amount of the work involves the repair of joints, and in many cases this does not improve ride quality.

VALIDATION OF PAVEMENT DESIGN PROCEDURES

The validation of the Pennsylvania pavement design procedure is beyond the scope of this paper; however, network analysis can provide information on this topic. Most of the Interstates were designed in accordance with AASHTO procedures. The majority of Pennsylvania's Interstate pavements are 10-in.-thick portland cement concrete with joints spaced every 61.5 ft. A dowel basket with 1.25-in. dowels provides the load transfer at the joint. After reviewing the overall network condition of Pennsylvania's Interstates, it can be concluded that the pavements certainly are deteriorating; however, based on their predicted design life, the pavements should be in much worse condition. Because the system is, on average, older than its 20-year design life, the mean PSI value would be expected to be near 2.5 (terminal PSI for AASHTO design). By simplifying the power function of PSI loss to a linear relationship, as described earlier, it was demonstrated that the annual loss of PSI over 20 years should be about 0.085 units (assuming that initial PSI = 4.2). The average rate of PSI loss in Pennsylvania is about 0.04 to 0.05 units.

It is speculative to predict what the average PSI of the Interstate system will be in the next 5 or 10 years. Figure 11 shows three possibilities: (a) an extrapolation of the data based on the linear equation developed earlier; (b) an extrapolation of data assuming that a PSI of 2.5 is reached 20 years after construction; and (c) an extrapolation assuming that a PSI of 2.5 is reached 25 years after construction.

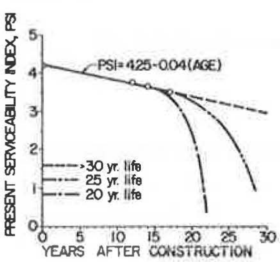

FIGURE 11 Prediction of average Interstate PSI.

It is doubtful that the Interstate system will deteriorate at a rate that will result in either case b or case c. Therefore, it can be said the the AASHTO design procedure, as used in Pennsylvania, is conservative. The mean PSI of 3.51 in 1981 is a good indication that the overall system is performing well, considering the amount of heavy vehicle traffic carried and the age of the pavements.

SUMMARY AND CONCLUSIONS

In this paper the serviceability evaluation of the complete Interstate highway system in Pennsylvania has been described. It has been demonstrated that PSI can be used at the network level to provide management with an overall picture of the system. With such information it is possible to determine which routes are deteriorating most rapidly. It is also possible to compare deterioration by district. The historical trends of PSI data can also be used to predict future network conditions on the Interstate system. Specific conclusions drawn from the study are as follows:

1. On a network level, the PSI of the Interstate system steadily declined since 1976 to a mean value of 3.51 in 1981;
2. Approximately 18 percent of the Interstate system in 1981 was less than the acceptable level of 3.3;
3. PSI values can be used on a network level to rank districts and individual routes according to need for funding; and
4. Betterment projects carried out between 1978 and 1980 had only minimal effects on the rate of deterioration of the Interstate system.

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Status of Highway Condition Scoring in New York State

DAVID T. HARTGEN

ABSTRACT

In this paper highway condition rating methods and results in New York State, as of the summer of 1983, are summarized. The focus of the paper is on procedures that the New York State Department of Transportation uses to assess the condition of highways in the state, and to provide that information in an accurate, rapid, and consistent fashion. The history of highway condition assessment in New York is briefly reviewed, and activities in scoring are described. Improvements in training procedures, tests of field consistency, improvements in data processing, and similar activities that are being undertaken in other agencies are reviewed. Results indicate that the overall condition of New York's 15,750-mile Touring Route System is generally good, but that 13 percent of road surfaces and 17 percent of road bases are rated in poor condition. It is concluded that the highway condition rating procedures used in New York are currently moving into a "shake-down" phase, where required improvements are less from year to year and results are generally satisfactory.

Pavement management is a broad strategy to protect the capital investment in the highway system and to ensure maximum serviceability of the highway system to the motoring public at a reasonable cost. It encompasses all aspects of highway planning, design, construction, and maintenance of pavement systems. Pavement management involves comparing investment alternatives for individual projects as well as network strategies, coordinating the various activities of the highway agency in maintaining and improving the highway system, and using information to make decisions in the best long-term interests of the motoring public.

The New York State Department of Transportation (NYSDOT) has established a Pavement Management Task Force to review its procedures and practices in pavement evaluation and to recommend methods for improving these procedures. The department has also developed a model to predict long-term network condition and funding requirements for alternative rehabilitation strategies, as well as to identify sections likely to need repair in future years.

An additional major effort described in this paper is the department's highway condition rating procedures. The most recent progress on this particular subject, which builds on earlier papers presented to the Transportation Research Board (1,2), is reviewed.

REVIEW OF NYSDOT ROAD RATING PROCEDURES

As Figure 1 shows, the department has conducted several types of surveys of highway condition. The condition survey consisted of road ratings on a verbal scale of 1 to 10. It is similar to the pavement serviceability rating (PSR). Highway condition assessment was undertaken periodically by regional teams in the department's 11 regional offices. Over time individual regional teams began to drift apart in consistency of using these verbal scales, and difficulties in processing the data and making it available in a useful form considerably reduced its value. As a result, the highway condition rating process, termed the sufficiency process, fell into disfavor in the 1970s and was conducted only periodically. In the meantime, the department used a road roughness measure called the present rideability index (PRI). But changes in vehicles and calibration problems associated with the system, as well as concerns about the process by which technical data on road roughness were correlated with perceptual data on rideability, led to the decline in the use of the system. PRI data were last collected in 1981. The department has currently (spring 1984) reassessed its system-level condition data needs.

The department also uses various sample-based road rating systems: (a) the federally mandated Highway Pavement Management System (HPMS) sample of 2,800 sections, which uses the familiar PSR rating scale; (b) the department's own system of continuous counters at which roads are rated by using the highway condition assessment; and (c) the Albany County highway deterioration study, which tracks the deterioration of 121 test and control sections using several condition measures.

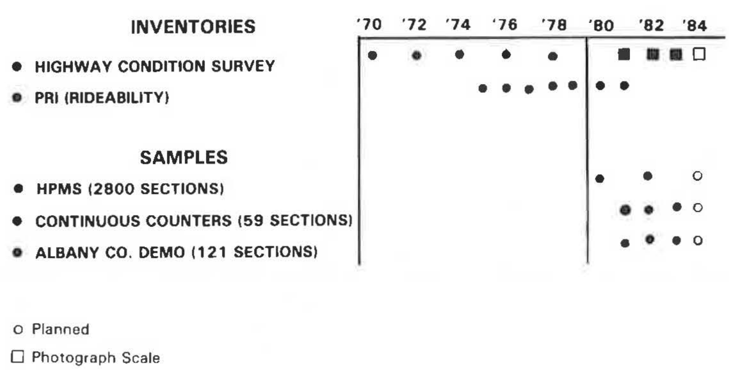

FIGURE 1 NYSDOT highway condition data.

Improvements in condition rating procedures were initiated by NYSDOT in 1981. To remove some of the problems with the highway condition survey, the department undertook the following major improvements.

1. Photograph scale: To improve the consistency and accuracy of data collected in the field, the department constructed a scale of photographs for use in the field. Each photograph was selected by standard methods in psychology and arranged in such a way that they formed a 10-point interval scale. The photographs were then reproduced and made available to regional rating teams along with revised verbal and technical description material on particular distress signals. Examples of this scale are given elsewhere (1,2).
2. Training: Detailed training was held in Albany for regional rating teams on the use of the photographs and the revised verbal scales.
3. Consolidation: The number of sections and the information collected were reviewed and consolidations of information were made wherever possible to streamline the rating effort.
4. Computer processing: Computer processing was steamlined and automated so that data would be made available to regional offices within 2 to 3 weeks of receipt. This allows data collected in one year to be used directly in program development for the next year.
5. Summaries: New summary capabilities were added to the department's processing system, so that relevant and useful summaries of the data could be provided to the department's program managers. Summaries of the condition of the entire system were prepared and distributed.
6. Forecasts of condition: The department developed a condition forecasting method, called the highway condition projection model (3), which uses these data to project highway condition in the future and estimate the cost of highway repair strategies.

These activities have greatly improved the repeatability, consistency, accuracy, and relevance of highway condition data. The highway condition information now being collected is finding its way into many aspects of the department's project development process, and it is being used as the primary method of highway condition assessment for the state highway system.

RECENT IMPROVEMENTS

The 1981 and 1982 experience indicated that the visual rating procedure was extremely easy to use in the field and it was satisfactory in accuracy, consistency, and reliability of the data provided. Therefore, additional improvements since 1983 have been minor.

Tie-Breaker Photographs

The department's 10-point scale is generally suitable, but great accuracy is needed in the 5- to 8-point range of this scale, where most decisions concerning pavement rehabilitation are made. Accordingly, the department selected tie-breaker photographs for insertion between points 5 and 6, 6 and 7, and 7 and 8. These photographs allow better judgments of which condition level a particular pavement section falls into. The department considered the development of half-point positions in the middle of the scale, but found the effort unnecessary. A typical condition scale is shown in Figure 2.

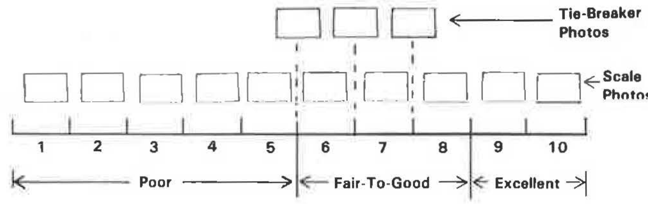

FIGURE 2 Schematic of photographic condition scale.

Special Codes

Although the two major highway condition scales (surface or base) allow for the classification of pavement condition, certain other rating needs arise. An excellent example would be the recent extensive joint-failure problems associated with I-84 in southern New York. These problems are associated with block faulting as a result of failure of load-transfer devices between the concrete sections, primarily in the driving lane. Such problems are associated with underlying distress and are recognized as such by regional raters, but management believed that it was important to specifically identify certain distress symptoms. Accordingly, a third code was added, called the special code, to provide other information believed necessary to have a thorough picture of the road condition. The special codes used for the 1983 surveying effort are given in Table 1. As may be noted, the codes focused primarily on faulting problems and on particular distress signals associated with overlays, such as edge faulting.

TABLE 1 Special Codes, 1983

Code	Description	Pavement Type
1	Faulting, low to medium severity (<0.25 in.)	PCC/overlay
2	Faulting, high severity (>0.25 in.)	PCC/overlay
3	Shoulder wash-out	All
4	Widening dropoff	Overlay
5	Distortion	Flexible
6	Localized, severe distress	All
9	Other (write in margin of score sheet)	All

Note: PCC = portland cement concrete.

Special codes may be changed each year as new issues develop, thus increasing substantially the flexibility of the rating process. In effect, the special codes operate like a marginal note, allowing the regional scorer to tell the main office about particular problems that need attention, but they do not necessarily fall under the more systemitized rating procedures developed thus far.

Maintenance Index

The department periodically asks its resident engineers (those individuals responsible for highway maintenance in each of the state's 62 counties) to prepare an assessment of the difficulty of maintaining each section of highway in its present condition. Discussions with the department's maintenance people and with the resident engineers led to improvements in this particular index for the 1983 rating effort. The maintenance index is given in Table 2. Note that the rating is not quantified in visual terms; it is essentially a verbal scoring system.

TABLE 2 Maintenance Index

Code	Description
9-10	Facility in excellent condition; requires little or no nonroutine maintenance
8	Facility shows some minor distress; requires some attention with nonroutine maintenance such as pothole repair
5-7	Facility shows some significant distress; requires considerable attention by maintenance operations
3-4	Facility requires an inordinate amount of maintenance resources; needs attention by construction contract
1-2	Facility is beyond the capability of maintenance forces to maintain; requires immediate attention by construction contract

Flip-Book Format

The use of visual scales is greatly assisted in the field if color photographs are arranged in flip-book format. The book may then be held up against a windshield and the photographs compared with the driver's eye view of the road. The highway condition scoring manuals were reformatted into flip-book formatting for the 1983 effort.

These improvements are generally minor compared with major improvements in the development of the scale, and the preparation of consistent codes, which was undertaken in 1981 and 1982. The effort for 1983 largely refined the existing scale and made minor improvements in scoring procedures.

TRAINING

The major improvements of the 1983 rating effort focused on training. Each of the department's 11 regions has a team responsible for rating the state highway system within its region. For the 1983 effort, training consisted of field tests and a training film.

Field Tests

Previous training processes were highly successful in introducing consistency and accuracy among the 11 regional scoring teams. The resulting consistency in estimates of condition by 11 regional teams was described in previous papers (1,2), but the estimates were based on films of roads, not actual field tests. Therefore, training did not ensure that field scores would be consistent. To ensure this, the department instituted a field test for the 1983 survey. The 11 regional teams were asked to drive over a route in the Albany area consisting of 10 test sections. These sections had been filmed for training purposes. The route (Figure 3) contains pavements of different types (rigid, flexible, and overlay), varying condition levels, and varying urban and rural settings. Regional scoring teams drove over the route shown in the figure, and recorded their estimates of surface and base ratings as well as the special codes previously described. The data were then discussed thoroughly the following day.

Training Film

The department developed materials for a training film on highway rating. The film shows the field test described in the previous section, and the results of the field tests and the visual field rating principles are discussed.

The videotape of the field test sections was

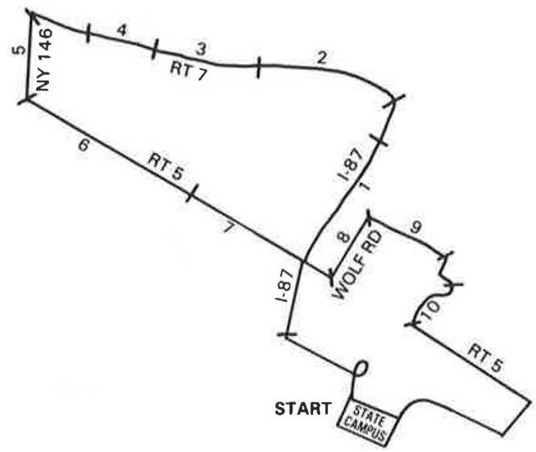

FIGURE 3 Test routes for training session.

shown to regional teams after they had completed their training. After each test section was traversed, the regional rating teams were invited to discuss their scores against the discussion on the videotape. The training film will also be used to train other governments in condition rating.

OVERALL PAVEMENT CONDITION, 1983

The overall pavement condition of the State Touring Route system remains quite good. Road surfaces are in slightly better condition than road bases. As of 1983, approximately 87 percent of road surfaces were in fair to excellent condition (level 6 or higher), compared with about 83 percent of road bases. Approximately 13 percent of road surfaces and 17 percent of road bases were in poor condition (level 5 or lower). These percentages are slightly different from those obtained in 1982, but the differences are smaller than probable scoring errors. About 1,909 sections out of 18,331 (10.4 percent) have special problems, primarily faulting and localized distress.

The average surface condition of the New York State Touring Route is about 6.93 (Table 3). Overall, the average condition has remained quite stable during the past 3 years. However, the department's continuing emphasis on resurfacing and rehabilitation has added mileage to the excellent (score 9 to 10) category, which now totals 12.3 percent, an encouraging sign. Approximately three-quarters of the mileage in the system is in the fair-to-good range (score 6 to 8), and not much progress has been made in decreasing this total. This means that the potential remains for major deterioration of the system (from the 7 to 6 level to the 5 level) if repair work is not increased. In particular, the level 6 condition (just above the poor level) continues to represent more than one-quarter of all mileage. These are sections that can often be rehabilitated with medium-to-heavy overlays (2.5 to 4 in.); but at an average cost of $300,000 per 2-lane mile, the backlog of work at this level alone is $1.58 billion. Unfortunately, if the department is unable to increase its attention to roads in this category, they will deteriorate into the poor category, where work is considerably more expensive. Thus, although the overall system is in good shape, many sections of pavement currently need attention.

The data in Table 4 indicate that the 1983 average condition of road bases is about 6.79 and has remained quite stable. These 1982-1983 differences are not large. More than 70 percent of the mileage

TABLE 3 Surface Condition, New York State Touring Route System

Condition Level		1981 Lane Miles	1981 Percent		1982 Lane Miles	1982 Percent		1983 Lane Miles	1983 Percent	
Excellent	10	1,188	3.0	6.6	1,021	2.6	9.9	1,487	3.7	12.3
	9	1,439	3.6		2,904	7.3		3,435	8.6	
Good to fair	8	8,381	21.1		7,656	19.3		7,761	19.4	
	7	13,487	34.0	80.4	11,858	29.8	76.1	11,638	29.1	75.0
	6	10,012	25.3		10,745	27.0		10,593	26.5	
Poor	5	3,828	9.7		4,249	10.7		4,214	10.5	
	4	1,152	2.9		1,041	2.7		699	1.8	
	3	154	0.4	13.0	234	0.6	14.0	168	0.4	12.7
	2	17	0.05		19	0.05		10	0.02	
	1	2	0.005		2	0.005		–	–	
Total		39,661			39,729			40,005		
Avg			6.82			6.82			6.93	

TABLE 4 Base Condition, New York State Touring Route System

Condition Level		1981 Lane Miles	1981 Percent		1982 Lane Miles	1982 Percent		1983 Lane Miles	1983 Percent	
Excellent	10	1,115	2.8	6.4	1,044	2.6	9.5	1,275	3.2	12.8
	9	1,442	3.6		2,747	6.9		3,859	9.6	
Good to fair	8	6,473	16.3		6,461	16.3		6,298	15.7	
	7	10,610	26.8	72.6	11,039	27.8	70.4	10,795	27.1	70.1
	6	11,712	29.5		10,439	26.3		10,982	27.4	
Poor	5	5,641	14.3		5,763	14.5		5,130	12.8	
	4	2,217	5.6		1,753	4.4		1,352	3.4	
	3	409	1.0	21.0	403	1.0	20.1	257	0.6	17.0
	2	39	0.1		77	0.2		57	0.2	
	1	3	0.008		3	0.008		–	–	
Total		39,661			39,729			40,005		
Avg			6.53			6.64			6.79	

remains in the good-to-fair range, with an especially large proportion (55 percent) in categories 7 and 6 (just above the poor range). No progress has been made in decreasing this proportion, and without attention, this mileage will slip into the lower range, thus necessitating more extensive work in the future.

FIELD CONSISTENCY IN RATING

The field training goes a long way to ensure that when highway sections are rated in the field, they will be rated consistently by different teams. But unless sections are checked in the field, there is no guarantee that the rating was accurate. To ensure this, NYSDOT undertook two measures.

Double-Scoring

In 1982 and 1983 the department field-checked a representative sample of highway sections. These sections, totaling approximately 10 percent of the state highway system, are shown in Figures 4 and 5. For half of these highway sections, a field team from the Albany main office estimated the condition of the section (Figure 4). For the other half, regional teams double-scored sections in adjacent regions (Figure 5). The data in Table 5 indicate that in a high proportion of cases, the difference between the two scores was one point or less. These results substantially improved the department's confidence in the overall rating process as it occurs in the field.

Consistency with Field Distress

To ensure that the department's photograph scale system is consistent with more detailed field dis-

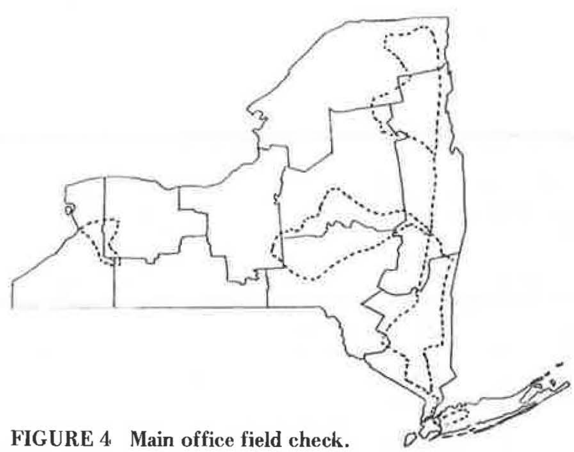

FIGURE 4 Main office field check.

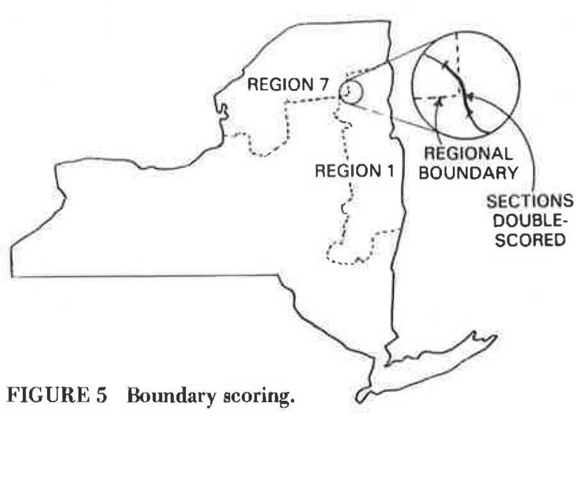

FIGURE 5 Boundary scoring.

TABLE 5 Field Consistency in Rating

	1982	1983
Total sections checked	1,130	1,173
Surface		
Average difference (points)	−0.11	+0.05
Probable range (points)	0.52	0.53
Percentage within ± 1 point	96.6	95.5
Base		
Average difference (points)	−0.39	+0.05
Probable range (points)	0.55	0.57
Percentage within ± 1 point	91.2	92.0

Note: Data are based on 1,173 double-scored sections, 674 by the main office and 499 by other regions.

tress, detailed field scoring is being undertaken of the sites in the Albany County demonstration project. This is a study that contains 121 test and control sections in Albany County. These sections have been treated with various kinds of improvements and then allowed to deteriorate. Detailed distress information on cracking, rutting, and patching are collected annually for each test and control section on the same highway. To develop the regression relationships between these detailed parameters and the overall scores, the analysis team has visually scored the same 121 sections annually.

PROCESSING

The following improvements have been made to the department's procedures for processing the highway condition data.

Summaries of Deteriorated Sections

The department has streamlined the procedure for developing its "red flag" list (i.e., highway sections that are in deteriorated shape). An "English names" version of highway sections is being added to the file so that the data may be listed in verbal rather than coded form. This facilitates the process of looking up deteriorated sections and identifying their locations. In addition, coordinates are being added to the highway sufficiency file so that the locations of sections may be plotted. This plotting capability will allow visual map-type summaries of data to be provided directly to the regional offices.

Processing and Modeling

The department's highway condition projection model (HCPM) (3) is a recently developed tool that allows the analysis of alternative rehabilitation strategies. A projection is made of a condition of a system in the future by using deterioration rates developed from history. Rehabilitation strategies are applied and the repairs are simulated. HCPM keeps track of estimated costs and conditions, and it provides this information at various levels of geographic detail, as well as an overall summary and a 5-year construction program.

Tools such as this are considerably more valuable if they can be made available to the individuals responsible for developing such strategies. At NYSDOT this is done primarily by the regional offices. Therefore, considerable effort has been undertaken to regionalize HCPM (i.e., make it available to regional offices through existing computer equipment and terminals available in each of the department's regional offices).

Paneling of Data

To ensure the accuracy of deterioration rates used in a variety of studies, the department has tightened its highway section measurement procedures. The result is that condition tracking of sections over time is now possible. Deterioration rates for highway sections over time from 1981, 1982, and 1983 highway condition surveys are given in Table 6.

TABLE 6 Deterioration of Sections, 1982 to 1983

	Centerline Miles			Avg
	Surface	Base	Combined	Change
Condition improved (2+ point increase)	894	1,082	988	+3.14
Unchanged or rescored (0, +1)	9,537	9,464	9,500	−0.18
Deteriorated				
−1	3,581	3,410	3,496	
−2	273	329	301	
−3	22	23	22	
−4, −5	2	2	2	
Total	14,309	14,310	14,309	

Of the 15,750 centerline miles inventoried in 1983, the department was able to match and compare the 1982 condition on 14,309 miles (90.6 percent). (Unmatched sections generally result from added or realigned mileage and are generally in good-to-excellent shape.) The data in Table 6 indicate that, of these, about 988 miles were improved an average of 3.14 points, whereas about 9,500 miles remained unchanged (0 or +1 in score), 3,496 miles declined 1 point, and 325 miles deteriorated 2 or more points. The particular causes of this rapid deterioration are unknown, and the department is attempting to isolate the factors. Therefore, although the overall data suggest a stable mean condition, the underlying reality is inexorable but slow deterioration that will eventually result in even greater future costs.

APPLICATIONS BY OTHER GROUPS

The highway condition ratings procedures described here have received considerable attention and interest from other groups. Current efforts to use the highway condition assessment process in other places are as follows.

1. New York State Thruway: The New York State Thruway Authority, which is responsible for the 587-mile-long Thruway, used this scoring process. Thruway personnel attended the training sessions and conducted a rating assessment of the Thruway in 1983.
2. Albany area: The metropolitan planning organization responsible for planning in the Albany metropolitan area used the rating process to evaluate the condition of Federal-Aid highways in the Albany area. They are also rating a sample of local roads, so that overall condition of different systems in different jurisdictional and functional classes can be compared on a consistent basis.
3. Niagara County: In the Buffalo metropolitan area a complete rating of county roads was undertaken in 1982 using the department's highway condition rating procedures. The data were processed by the Niagara County planning and engineering staff. Erie County (Buffalo City) is currently studying the use of the method for its needs.
4. New York City: New York City is planning to rate highway sections under its jurisdiction, and it is currently undergoing training by NYSDOT personnel in the use and application of the rating methods.

CONCLUSION AND DISCUSSION

The status of the department's highway condition rating efforts is reviewed. Great progress has been made in the past 3 years in improving the consistency, accuracy, and quality of the highway condition data collected by New York and in providing it to a variety of clients in rapid and relevant fashion. Virtually all aspects of the highway condition assessment and data processing effort have been reviewed and streamlined. The big effort, in terms of methodology development, is over, and the procedure is now moving into an implementation and "shake-out" phase in which refinements to the methodology are becoming more detailed and fewer changes are occurring from year to year. Overall, the department is pleased with the methodology, and is placing greater reliance on the results of the survey and on the analyses that are conducted from it.

No highway condition assessment procedure should be static. Issues, highway conditions, and concerns change. The procedure being developed by the department is flexible and is capable of undergoing change to meet evolving needs, while at the same time retaining consistency in data so that trends may be computed. A fully integrated and static data base is probably beyond the need of the department, but it can be reasonably well approximated by the application of consistent measurement principles and a tightened rating and data provision process. This is the goal that the department is working toward, and it is the goal to which the department believes it has made considerable progress.

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Use of Pocket Computers for Rehabilitation of Rural Roads in Dominica

LOUIS BERGER and JACOB GREENSTEIN

ABSTRACT

A 50-km rural road that connects Roseau-Pont Casse and Hatton Garden in Dominica was evaluated by means of the Benkelman beam in February 1983. The rebound deflection basin obtained under a dual-wheel axle load was interpreted by means of a pocket computer with 8-K RAM. The subgrade modulus, subgrade California bearing ratio, base modulus, asphalt modulus, and the required asphalt concrete overlay were calculated for each point while performing the nondestructive testing (NDT) survey. Although measurement of deflection basins with the Benkelman beam is not common practice, satisfactory results were obtained. A team composed of the truck driver and his assistant, an experienced engineer and his assistant, and two traffic control men was able to measure 80 to 100 deflection basins, or about 10 km of road, in a typical working day. By using the pocket computer, all calculations, including the overlay thickness of each tested point, can be completed in about 1 min. Therefore, the road rehabilitation design can be completed while conducting the NDT. In Dominica both the NDT and the strengthening design of the 50-km road were done simultaneously and completed in 1 week. The detailed methodology and computer programs are presented in this paper. The program is based on the theory of linear elastic systems and written in BASIC language. It can be easily adjusted and implemented with other nondestructive pavement evaluation devices such as the road rater or the falling weight deflectometer.

In the evaluation process of pavement systems by means of nondestructive testing (NDT), the response of the pavement is observed and material properties can be back-calculated. Among the different responses of the pavement to load, the only practical measurements are elastic deflections. Two methods for determining the elastic deflections are generally used. According to the first method (1-3), in each location only the center or the standard maximum deflection is determined. The magnitude of this deflection is interpreted to predict pavement performance.

In the second procedure, which is a rational one, the deflection basin (i.e., the center deflection) and at least one offset deflection are determined (4-7). The deflection basin is used to back-calculate the elastic modulus of the subgrade and pavement system. These strength parameters and the projected traffic loading are used to design pavement strengthening.

This rational procedure was implemented in Dominica by means of a pocket computer (Sharp PC-1500) to upgrade 50 km of low-volume road between Roseau-Pont Casse and Hatton Garden (see Figure 1). The computer program (Figure 2) is written in the BASIC language and can be implemented on any personal or pocket computer that has 8-K RAM. Because the program is based on the theory of the linear elastic system, it can be easily adjusted and used with other nondestructive pavement evaluation devices such as the road rater (pavement profiler) or the falling weight deflectometer.

BENKELMAN BEAM DEFLECTION PROCEDURE

The Benkelman beam is a widely used device to measure surface deflections in all types of pavement structures. The beam operates on the lever principle, as shown schematically in Figure 3. Every vertical movement of the tip of the beam generates a rotation of the beam through the pivot. A proportion of the tip movement is read with the dial gauge installed at the far end of the beam. The ratio of the rotating lengths of the beam is generally 1:4 (including the beam used in Dominica); thus the dial gauge (Figure 1d) at the end of the beam moves one-fourth of the vertical movement at the tip of the beam. Often the dial gauge is already calibrated to read the full tip movement (i.e., no multiplication by four is required).

The truck used in Dominica had a single dual-wheel rear axle weighing 7174 kg and a tire pressure of 4.9 kg/cm^2. This load was chosen instead of the commonly used 8200-kg axle load because Dominica truck loads seldom reach the 8200-kg level. The dual-wheel load (PP) (in kg) is specified in line 5050 in the program, and any value of PP can be used.

The so-called "rebound method" was used in the Benkelman beam measurements. The truck moved away from the testing point at creep speed, and the rebound deflections were measured. This method was used to measure not only the maximum deflection under the rear axle (DØ), but also to measure two additional deflections--D4Ø and D8Ø--at 40 and 80 cm away from the maximum, respectively. This nonroutine Benkelman beam deflection procedure was used to characterize the whole deflection basin that is needed in the structural evaluation methodology explained in the following sections.

The choice of 40 and 80 cm was not arbitrary. The goal was to choose one distance where the deflection would be about 50 percent of the maximum deflection. In Dominica this distance was between 30 and 40 cm,

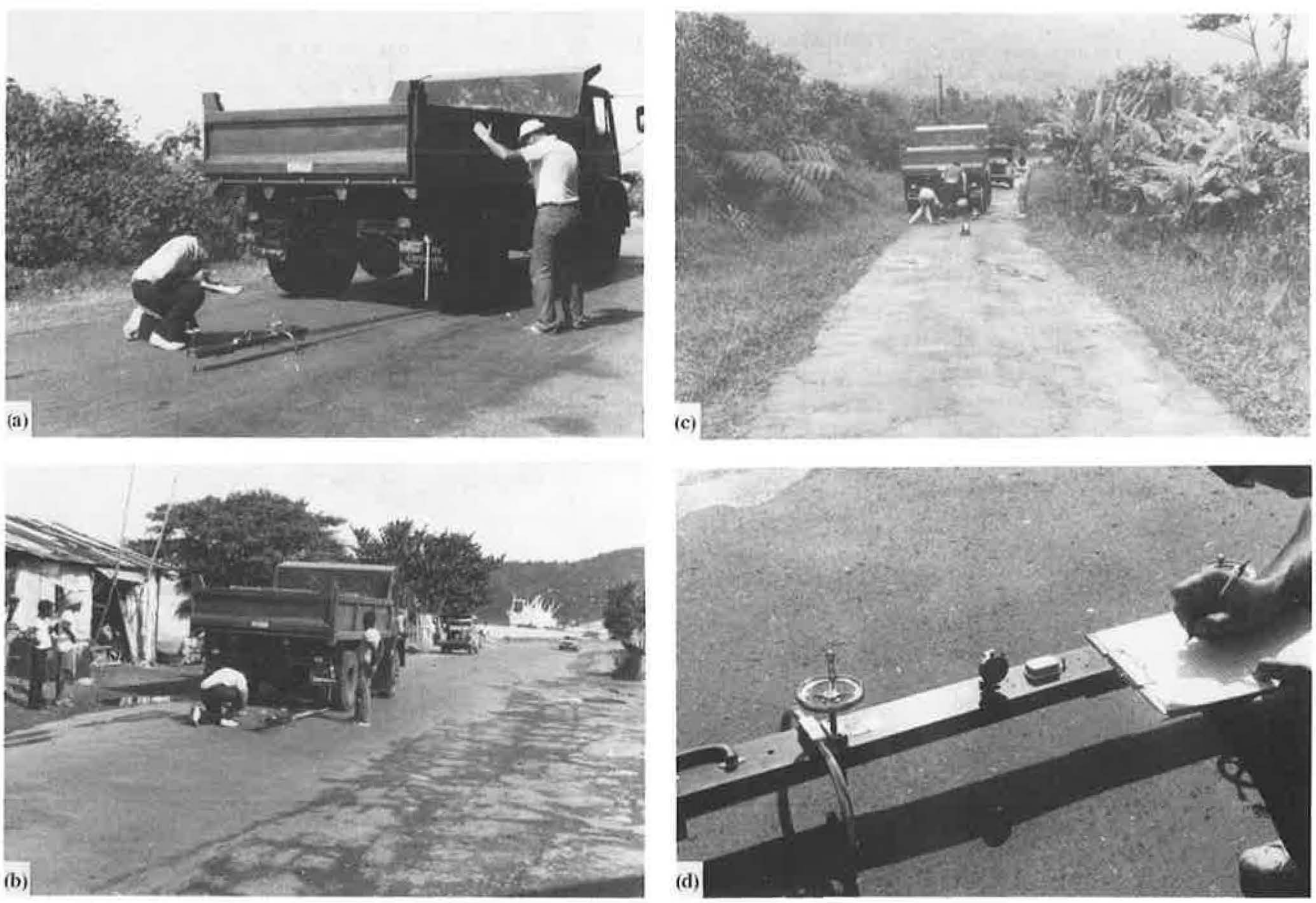


FIGURE 1 Benkelman beam operations in Dominica (Roseau-Hatton Garden Road).

```
100:REM NAME BENKE
    LMAN BEAM;DOMI
    NICA FEB./1983
105:REM  NAME"BBDO
    M/2"
110:REM  ASSUME WH
    EEL DIST.=3A
120:REM FEB./1983
200:GOSUB 5000:REM
    DATA
210:AC=A
220:INPUT "H/L=(10
    ,INFINIT)";M$
230:CODE=3
240:IF M$="10"AND
    V3=.5THEN LET
    CODE=1
250:IF M$="10"AND
    V3=.4THEN LET
    CODE=2
255:REM CODE=1(H/L
    =10,V=.5)
260:IF CODE>1THEN
    GOTO 320
265:Y8=.620:M8=.18
    3:M9=.520:I9=.
    1614:ZZ=.1925/
    .1614
270:B=0:BB=1.6890:
    AA=.4065
280:IF DR/D0>.7
    THEN GOTO 900
290:B=2:BB=4.5663:
    AA=2.6947E-3
305:GOTO 900
310:REM CODE 2(H/L
    =10,V=.4)
320:IF CODE>2THEN
    GOTO 380
325:Y8=.602:M8=.19
    2:M9=.480:I9=.
    1689:ZZ=.1925/
    .1689
330:B=0:BB=1.8246:
    AA=.3804
340:IF DR/D0>.426
    THEN GOTO 900
350:B=3:BB=4.9903:
    AA=4.3795E-4
365:GOTO 900
370:REM CODE 3(H/L
    =INFINIT)
380:B=0:BB=1.7117:
    AA=.3210
390:Y8=.525:M8=.18
    0:M9=.440:I9=
    1925:ZZ=1.0
900:IF E3<>0THEN
    GOTO 1200
1000:REM  COMPUTE
     E3 HOGG
1040:R5=R*(1/AA^(
    1/BB)-B)/((1
    /AA*(D0/DR-1
    ))^(1/BB)-B)
1050:LPRINT "R50
    CM=";INT (R5
    *10)/10
1060:AC=A
1070:L0=(Y8*R5+
    SQR ((Y8*R5)
    ^2-4*M8*AC*R
    5))/2
1080:IF AC/L0<.1
    THEN LET L0=
    (Y8-.1*M8)*R
    5
1085:LPRINT "L0
    CM=";INT (L0
    *10)/10
1087:WAIT
1090:AC=A
1100:SR=(1-M9*(AC
    /L0-.1))^-1:
    REM SR=S/S0
1110:IF SR<1THEN
    LET SR=1
1120:E3=(1+V3)*(3
    -4*V3)/2/(1-
    V3)*I9*PP/D0
    /SR/L0
1200:LPRINT "E0 K
    G/SQ.CM=";
    INT (E3)
1201:WAIT
1202:CBR=E3/CE
1203:LPRINT "CBR
    % =";INT (CB
    R*10)/10
1205:DS=E3*L0^3*2
    *(1-V3)/(1+V
    3)/(3-4*V3)
1208:WAIT
1210:REM  CENTER
    DEF BETWEEN
    WHEELS ;ULIT
    Z OFFSET DEF
     USED TO CAL
    CULATE D0
1220:AA=2*(1-V3)
1223:NN=100
1225:IF CODE=1
    THEN LET NN=
    10
1227:IF CODE=2
    THEN LET NN=
    10
1230:RR=1.5*A
1240:EC=2*E3
1250:DC=AA/EC/RR
1260:ZC=HC+.6*A*A
    /HC
1270:RC=SQR (ZC*Z
    C+RR*RR)
1280:DC=DC-(AA+(Z
    C/RC)^2)/RC/
    EC
1290:HE2=.9*HC*((
    EC/E3)^(1/3)
    )
1300:ZC2=HE2+.6*A
    *A/HE2
1310:RC2=SQR (ZC2
    *ZC2+RR*RR)
1320:DC=DC+(AA+(Z
    C2/RC2)^2)/R
    C2/E3
1330:ZC3=HE2+NN*L
    0+.6*A*A/(HE
    2+NN*L0)
1340:RC3=SQR (ZC3
    *ZC3+RR*RR)
1350:DC=DC-(AA+(Z
    C3/RC3)^2)/R
    C3/E3
1355:DC=DC*(1+V3)
    *PP/π/2
1360:IF ABS (DC-D
    0)/D0<.01
    THEN GOTO 16
    10
1370:EC=EC*DC/D0
1380:GOTO 1250
1610:LPRINT "E* K
    G/SQ.CM=";
    INT (EC)
1620:WAIT
1890:REM  COMPUTE
     E1
1900:IF E1<>0THEN
    GOTO 2110
2000:REM  COMPUTE
     E1
2010:IF TRB=0THEN
    LET TRB=99.1
    3-26.35*.434
    3*LN (PEN)
2020:P1=20*TRB+50
    0*.4343*LN (
    PEN)-1951.55
2030:P1=P1/(TRB-5
    0*.4343*LN (
    PEN)+120.15)
2040:SB=1.157E-6*
    TS^-.368*2.7
    183^-P1*(TRB
    -TC)^5
2050:VG=(1-MB)/GS
    *(1-VA)
2060:VG=VG/(MB/GB
    +(1-MB)/GS)
2070:CV=VG/(1-VA)
2080:IF VA>.03
    THEN LET CV=
    CV/(.97+VA)
2090:N9=.83*.4343
    *LN (4E5/SB)
2100:E1=ATM*SB*(1
    +2.5/N9*CV/(
    1-CV))^N9
2110:LPRINT "E1 K
    G/SQ.CM=";
    INT (E1)
2120:WAIT
2390:REM  COMPUTE
     E2
2400:IF E2<>0THEN
    GOTO 2610
2500:REM  COMPUTE
     E2
2510:A8=H2/H1
2520:N8=1.0:REM N
    8=E1/E2
2530:REM E6=(E*/E
    1)TRIAL
2540:E6=A8^4+4*A8
    ^3*N8+6*A8^2
    *N8+4*A8*N8+
    N8^2
2550:E6=E6/N8/(A8
    +N8)/(A8+1)^
    3
2560:E7=EC/E1
2570:IF ABS (E6/E
    7-1)<.0001
    THEN GOTO 26
    00
2580:N8=N8*E6/E7
2590:GOTO 2540
2600:E2=E1/N8
2610:LPRINT "E2 K
    G/SQ.CM=";
    INT (E2)
2620:WAIT
2900:REM COMPUTE
    DH
3000:HH=(EC*(HC^3
    )/3.0/E3)^(1
    /3)
3010:LPRINT "HEQ
    CM=";INT (HH
    )
3100:WAIT
3200:DH=(MJ*(CBR^
    -.53)-HH)/AE
3300:LPRINT "DH
    CM=";INT (DH
    *10)/10
4000:END
5000:REM DATA
5010:REM UNITS:CM
    .,KG.
5020:READ ATM
5030:DATA 1.0
5040:READ PP,A,TS
    ,TC
5050:DATA 3587,10
    .5,.010,27
5080:READ PEN,TRB
5090:DATA 50,55.0
5100:READ VA,GS
5110:DATA .035,2.
    65
5120:READ GB,MB
5130:DATA 1.04,.0
    6
5140:READ H1,H2,H
    C
5150:DATA 1.0,14.
    0,15.0
5160:READ V3,AE
5170:DATA .4,3.5
5180:READ E1,E2
5190:DATA 0000.,0
5200:READ E3,EC,C
    E:REM CE=E/C
    BR
5210:DATA 0,0,130
5220:READ D0,R,DR
    ,MJ
5230:DATA 0.85E-1
    ,40,0.44E-1,
    76.20
5240:RETURN

     RUN    h/ℓ = 10

R50 CM= 41.5
L0  CM= 21
E0 KG/SQ.CM= 447
CBR % = 3.4
E* KG/SQ.CM= 4596
E1 KG/SQ.CM= 22289
E2 KG/SQ.CM= 2367
HEQ CM= 22
DH  CM= 4

     RUN   h/ℓ = ∞

R50 CM= 41.6
L0  CM= 17.3
E0 KG/SQ.CM= 594
CBR % = 4.5
E* KG/SQ.CM= 2773
E1 KG/SQ.CM= 22289
E2 KG/SQ.CM= 1128
HEQ CM= 17
DH  CM= 3.9
```

FIGURE 2 Computer program printouts.

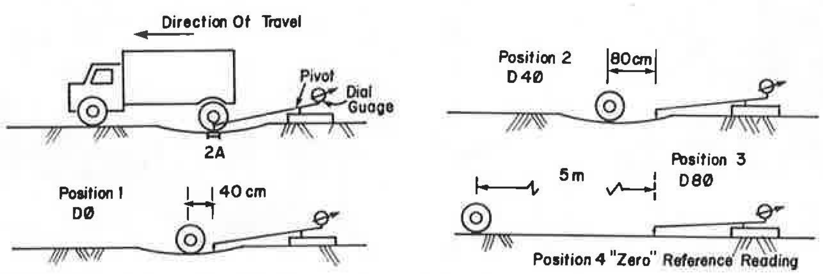


FIGURE 3 Schematic of Benkelman beam deflection procedure.

and 40 was chosen for routine measurements. With little practice it is possible to measure the offset deflections without having to stop the moving truck. A team composed of the truck driver and his assistant, an experienced engineer and his assistant, and two traffic control men was able to measure 80 to 100 deflection basins, or about 10 km of road, in a typical working day. It is desirable that a small pickup truck is used to carry the men, the beam, and miscellaneous equipment.

DETERMINATION OF THE SUBGRADE MODULUS

In this section the subgrade modulus (EØ) and the California bearing ratio (CBR) are determined [see lines 1000 to 1023 of the program (Figure 2)]. The modulus of the subgrade EØ is determined by using the Hogg model (4-6) with a finite subgrade at a depth of H = 10 x LØ (see Figure 4) or at a depth of infinity,

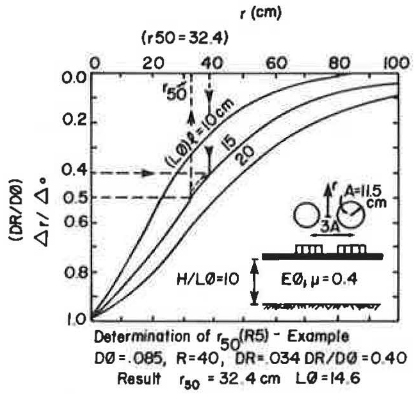


FIGURE 4 Deflection basins for Hogg model and Benkelman beam loading.

h = ∞. Figure 5 shows the computerized procedure that determines EØ from the interpretation of the deflection basin. This procedure is based on the following steps:

Step 1: Determination of r50 = R50;
Step 2: Determination of ℓ(LØ); ℓ(LØ) is the characteristic length (4-6);
Step 3: Determination of the ratio S_o (point load stiffness) to S (area load stiffness); and
Step 4: Calculation of EØ (E3).

Step 1: Determination of r50 (R5 or R50)

For purposes of this paper, r50 = R5, which is the offset distance R at which $\Delta r/\Delta 0$ = DR/DØ = 0.5. DØ and DR are the center and offset deflection, respectively.

The shape of the deflection bowl for point loading is described by the following equations:

$$(D\emptyset/DR) - 1 = A[(R/\ell) + B]^C \quad (1)$$

or

$$R = \ell\left\{(1/A)[(D\emptyset/DR) - 1]\right\}^{1/C} - B \quad (2)$$

where A, B, and C are curve-fitting coefficients (see Table 1) and R is the distance offset of DR. For DR/DØ = 0.5,

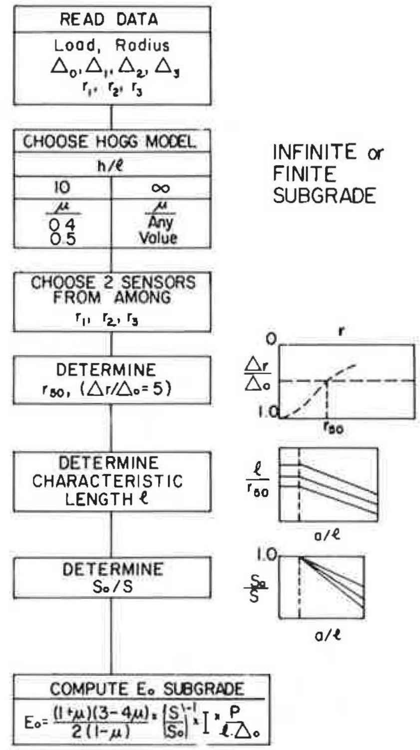


FIGURE 5 Determination of E subgrade from deflection measurements using Hogg model.

$$r50 = R5 = \ell[(1/A)^{1/C} - B] \quad (3)$$

Thus Equations 2 and 3 give

$$R5 = R\left([(1/A)^{1/C} - B] / \left\{(1/A)[(D\emptyset/DR) - 1]^{1/C} - B\right\}\right) \quad (4)$$

The values for A, B, and C, as obtained for the Hogg model, are given in Table 1. For example, for μ = 0.4, h/LØ = 10, DØ = 0.085 cm, DR = 0.034 cm, R = 40 cm, and DR/DØ = 0.034/0.085 = 0.40, use Equation 4 to find R5 = 32.4 cm.

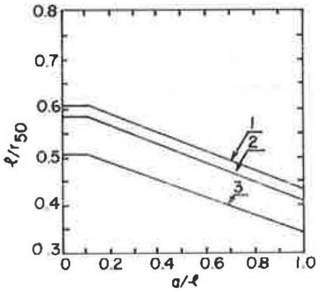
TABLE 1 Curve-Fitting Coefficients

H/LØ	DR/DØ	μ	A	B	C
H/LØ = ∞	Any value	Any value	1.3210	0	1.7117
10	>0.7	0.5	0.4065	0	1.689
10	>0.7	0.5	2.6947×10^{-3}	2	4.5663
10	>0.426	0.4	0.3804	0	1.8246
10	<0.426	0.4	4.3795×10^{-4}	3	4.9903

Figure 4 shows a graphical verification of the computerized solution. Enter the figure with DR/DØ = 0.40 and r = R = 40 cm. Draw a line parallel to the LØ lines until meeting the DR/DØ = 0.5 horizontal line. Read r50 on the horizontal axis (R5 = r50 = 32.4 m). The methodology for determining r50 is described in lines 1040 and 1050 of the computer program (see Figure 2).

Step 2: Determination of Characteristic length ℓ(LØ)

Figure 6 shows the theoretical relationships between ℓ/r50 and a/ℓ for different values of H/ℓ and μ [a(A) is the radius of the contact area between the tire and the surface]. The equation shown in Figure 6 gives the same relationships in analytical form. Each different line in Figure 6 (1, 2, or 3) is described in the equation by different values of the parameters Y_0 and m for different values of μ and subgrade depth H/LØ = 10 or ∞.



$$\ell = \frac{y_o r_{50} + \sqrt{(y_o r_{50})^2 - 4 m a r_{50}}}{2}$$

CODE	1	2	3
h/ℓ	10	10	∞
μ	0.5	0.4	all values
y_o	0.620	0.602	0.525
m	0.183	0.192	0.180

FIGURE 6 Benkelman beam loading [$\ell = f(r_{50}, a/\ell)$].

The equation is used in the personal computer program to determine ℓ(LØ). As an example, use the data that were used to determine r50: H/LØ = 10, μ = 0.4, a(A) = 11.5 cm, DØ = 0.085 cm, r50 = 32.4 cm, and PP = 3587 kg. Thus by using Code 2 in Figure 6, ℓ(LØ) is

$$\ell = \{0.602 \times 32.4 + [(0.602 \times 32.4)^2 - 4 \times 0.192 \times 11.5 \times 32.4]^{1/2}\}/2.$$

$$\ell = 14.6 \text{ cm}.$$

The methodology for determining ℓ(LØ) is described in lines 1070 and 1080 of the computer program.

Step 3: Determination of Ratio S_o/S

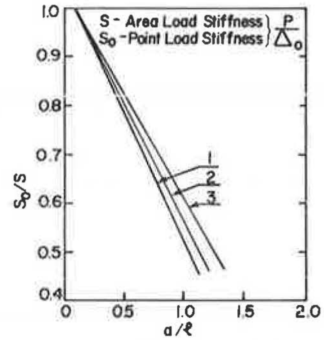
To develop numerical solutions of the subgrade modulus that are programmable in pocket computers, it is necessary to use this intermediate step. This step finds the theoretical relationship between point load (S_o) and area load (S) stiffnesses for a given ratio a/ℓ. This relationship is shown in Figure 7. Stiffness is defined as the ratio of the load to the deflection. The different lines (1, 2, and 3) in Figure 6 have an analytical expression that is also shown in Figure 7. A different value of the parameter $\bar{m}$ is used for different values of H/LØ and μ. In the numerical example,

$$A/L = a/\ell = 11.5/14.6 = 0.79;$$

thus

$$S_o/S = 1.0 - 0.48\ (0.79 - 1.0) = 0.67.$$

The ratio S_o/S is determined in the computer program between lines 1100 and 1110.



CODE	1	2	3
h/ℓ	10	10	∞
μ	0.5	0.4	all values
$\bar{m}$	0.520	0.480	0.440
I	0.1614	0.1689	0.1925

$$\frac{S_o}{S} = 1.0 - \bar{m}\left(\frac{a}{\ell} - 0.10\right)$$

$$\Delta = I \times \frac{P}{\ell} R \left(\frac{S}{S_o}\right)^{-1}$$

$$R = E_o \frac{2(1-\mu)}{(1+\mu)(3-4\mu)}$$

FIGURE 7 Benkelman beam loading [$S_o/S = f(a/\ell)$].

Step 4: Calculation of EØ

EØ is finally found by using the following equation (see lines 1120 to 1200 of the program):

$$E\emptyset = \{[(1+\mu)(3-4\mu)]/2(1-\mu)\} \times (I{*}PP/L\emptyset{*}D\emptyset) \times (S_o/S) \qquad (5)$$

where I is a fitting parameter that depends on H/LØ and μ (see table in Figure 7). For the data of the example,

$$E\emptyset = \{[(1 + 0.4)\ (3 - 4 \times 0.4)]/2(1 - 0.4)\} \times [(0.1689 \times 3587)/(14.6 \times 0.085)] \times 0.67 = 534 \text{ kg/cm}^2.$$

Determination of Subgrade CBR

The subgrade modulus can be used to calculate the CBR (4-6,8,9):

$$\text{CBR} = E\emptyset\ (\text{in kg/cm}^2)/\text{CE} \qquad (6)$$

(see lines 1202 and 1203), where CE is an empirical factor that varies between 100 and 160 for in situ CBR between 2 and 30 (4). CE = 130 (see data lines 5200 to 5210) was used and found to be appropriate for the subgrade of the Roseau to Hatton Garden road in Dominica.

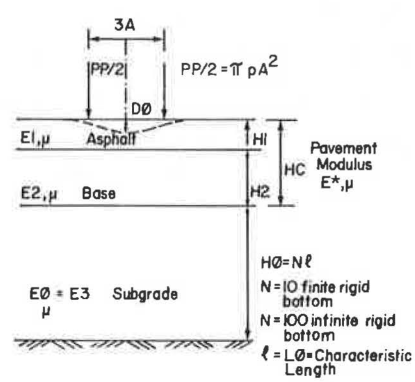


FIGURE 8 Determination of pavement modulus (E*).

DETERMINATION OF PAVEMENT MODULUS (E*)

The combined modulus E* of the asphalt concrete and the base layers with a combined thickness of HC = H1 + H2 (see Figure 8) is determined by using the Odemark-Ullidtz equations (10) for equivalent thickness (11). The equivalent thickness is determined according to the following equation:

$$HE = 0.9\ HC\ (E^*/E\emptyset)^{1/3} \tag{7}$$

The relationship between the center deflection DØ (between the dual wheels), the elastic modulus of the subgrade (EØ), and the pavement (E*) is given in the following equation:

$$\begin{aligned} D\emptyset = {} & [(1+\mu)(PP)/2\pi] \Big[(1/E^*)\Big([2(1-\mu/r] - [1/R(1)] \big\{2(1-\mu) \\ & + [Z(1)/R(1)]^2 \big\} \Big) + (1/E\emptyset)\Big([1/R(2)] \big\{ 2(1-\mu) + [Z(2)/R(2)]^2 \big\} \\ & - [1/R(3)] \big\{2(1-\mu) + [Z(3)/R(3)]^2 \big\} \Big)\Big] \end{aligned} \tag{8}$$

where

$$r = 1.5A = 1.5a \tag{9}$$

$$Z(1) = HC + 0.6\ (A^2)/HC \tag{9a}$$

$$R(1) = \{[Z(1)]^2 + (1.5A)^2\}^{1/2} \tag{9b}$$

$$Z(2) = HE + 0.6\ (A^2)/HE \tag{9c}$$

$$HE = 0.9HC\ (E^*/E\emptyset)^{1/3} \tag{9d}$$

$$R(2) = \{[Z(2)]^2 + (1.5A)^2\}^{1/2} \tag{9e}$$

$$Z(3) = (HE + N\ell) + 0.6(A^2)/(HE + N\ell) \tag{9f}$$

$$R(3) = \{[Z(3)]^2 + (1.5A)^2\}^{1/2} \tag{9g}$$

N = 10 for rigid bottom at finite depth,
N = 100 for infinite subgrade, and
PP = $\pi A^2 p$ (p = tire pressure).

Equations 7, 8, and 9-9g are used iteratively to determine the pavement modulus E* for any given combination of subgrade modulus (EØ), pavement thickness (HC), load (PP), tire pressure (p), and center deflection (DØ). This calculation is done automatically by the computer (see lines 1210 to 1610 in Figure 2).

DETERMINATION OF ASPHALT MODULUS E1

Guidelines to determine the asphalt modulus are given, among other sources, in several reports (12-16). The Shell methodology (15,16) was found to be practical; it is implemented here for low-cost roads. According to this methodology, the stiffness of the bitumen (asphalt cement) can be calculated according to the following equation quoted from Ullidtz and Peattie (11):

$$SB = 1.157 \times 10^{-6} \times T_s^{-0.368} \times e^{-PI} \times (TRB - TC)^5 \tag{10}$$

where

SB = stiffness of bitumen (kg/cm²); the term stiffness is used to denote the modulus or instantaneous relationship between the stress and the strain, which corresponds to particular values of temperature and of loading;
T_s = time of loading (sec);
TRB = softening point, ring and ball (ASTM) of bitumen (°C);
TC = temperature of the bitumen (°C); and
PI = penetration index of the bitumen, i.e.,

$$\begin{aligned} PI = {} & [20\ (TRB) + 500\ LOG\ (PEN) - 1{,}951.55] \\ & \div [(TRB) - 50\ LOG\ (PEN) + 120.15] \end{aligned} \tag{11}$$

where PEN is the bituminous penetration at 25°C.

For the analysis of pavements, the properties of the bitumen in the road must be used. It may be convenient to recover bitumen from the road and measure its penetration directly. The following approximate relationship (11) has been found to apply to many road bitumens:

$$\text{PEN in road} = 0.65 \times \text{original PEN} \tag{12}$$

It may also be convenient to measure TRB directly. Measurements of a wide range of road bitumens led to the development of the following empirical relationship (11):

$$TRB\ (^\circ C) = 99.13 - 26.35 \times LOG\ (PEN) \tag{13}$$

Equation 10 gives reasonable results when the loading time is between 0.01 and 0.1 sec, the PI is between -1 and +1, and the TRB - TC is between 20° and 60°C. The detailed procedure to calculate PI and SB is given in the program (see lines 2000 to 2040). For example, if TS = 0.01 and TC = 27°C (data line 5050), and if field penetration PEN = 50 and TRB = 55°C (data line 5090), then using Equation 11 (lines 2020 and 2030 in the program) gives PI = $2.3 \times 10^{-2} \approx 0$, and using Equation 10 (line 2040) gives SB ≈ 110 kg/cm².

The elastic modulus (E1) of the asphalt concrete (AC) mix is a function of the stiffness of the bitumen, the amount of mineral aggregate, and the air void percentage. E1 can be calculated according to the following equations (11):

$$E1 = SB \times \{1 + (2.5/N) \cdot [CV/(1 - CV)]\}^N \tag{14}$$

$$N = 0.83\ LOG\ (4 \times 10^5/SB) \qquad SB \text{ in kg/cm}^2 \tag{14a}$$

$$CV = \begin{cases} VG/(1 - VA) & \text{when } VA \leqslant 0.03 \\ VG/[(1 - VA) \times (0.97 + VA)] & \text{when } VA > 0.03 \end{cases} \tag{14b}$$

$$VG = [(1 - MB)/GS] \times (1 - VA)/\{(MB/GB) + [(1 - MB)/GS]\} \tag{14c}$$

where

GS = specific gravity of the mineral aggregate (see lines 5100 and 5110, GS = 2.65),
VA = percentage of air voids (see lines 5100, 5110, VA = 0.035), and
GB and MB = specific gravity and percentage of the bitumen, respectively (see lines 5120 and 5130, GB = 1.04 and MB = 0.06).

For the specific case presented in Figure 2 (computer program), SB = 110.9, VG = 2.65, VA = 0.035, GB = 1.04, MB = 0.06, and E1 = 22 289 kg/cm².

The following concluding remarks summarize the determination of E1 (AC elastic modulus).

1. E1 is determined mainly from the engineering properties of the bitumen, the aggregate, and the AC mix.

2. E1 is used to determine E2 (granular material) based on the NDT methodology, as described in the following section. In the event that E1 is overestimated or underestimated, this will be reflected in the value of E2 in such a way that the total flexural stiffness (EH³) remains constant as back-calculated from the NDT data.

3. The methodology presented herein is applica-

ble to uncracked, sound AC layers greater than 1 in. thick. Cracked or thin asphalt should be considered as part of the granular layer.

DETERMINATION OF BASE MODULUS E2

The modulus of the base layer is determined according to Nijboer's equation (Equation 15), which is based on the theory of strength of materials (16):

$$E^*/E1 = [(H2/H1)^4 + 4(H2/H1)^3(E1/E2) + 6(H2/H1)^2(E1/E2) + 4(H2/H1)(E1/E2) + (E1/E2)^2] / \{(E1/E2)[(H2/H1) + (E1/E2)]\ [(H2/H1) + 1]^3\} \qquad (15)$$

Equation 15 is based on the assumption that the flexural rigidity (EI) of the two layers H1 and H2 is equal to the EI of the composite pavement E* and HC, or

EI (E1, H1 and E2, H2) = EI (E*, HC).

It is also assumed that there is full friction between the asphalt and the base layers. The input data are

- H1 = asphalt layer thickness,
- E1 = asphalt modulus,
- H2 = base thickness,
- E* = composite pavement modulus of the asphalt and base materials, and
- HC = H1 + H2, which is the total pavement thickness of asphalt and base materials.

The only unknown is the elastic modulus of the granular material (E2), which is determined iteratively by using the personal computer (see lines 2500 to 2610 in the computer program).

DETERMINATION OF OVERLAY THICKNESS DH

The required overlay thickness DH' is determined by using the following procedure:

$$DH' = H - HEQ \qquad (16)$$

where DH' is the required additional thickness of gravel material (subbase or base), and H is the required total pavement thickness, subbase (CBR = 30) + base (CBR = 80) + thin layer of asphalt (usually less than 2 in.). H can be determined by using any

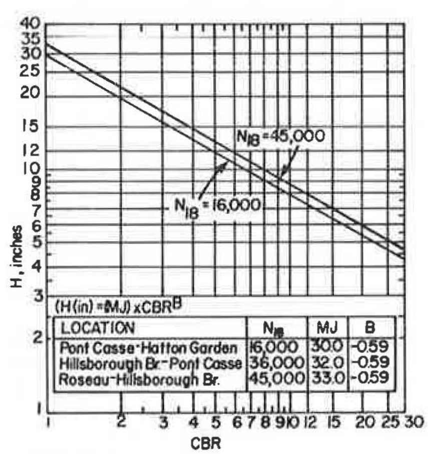


FIGURE 9 Thickness design curves for Roseau-Hatton Garden Road (*21*).

pavement design methodology for low-volume roads (17-21) that presents the relationship between the subgrade CBR, the projected traffic loading, and H. Figure 9 (21) shows the thickness design curves used for the rehabilitation program of the Roseau-Hatton Garden road in Dominica; the curves are based on the Transport and Road Research Laboratory (TRRL) method (21). As an example, the projected traffic loading of the road section from Pont Casse to Hatton Garden is 16,000 equivalent axle loads (EALs). In this case, the required thickness for CBR = 6 is H = 10.5 in. The mathematical relationship between H and CBR for the Roseau-Hatton Garden road [TRRL method (21)] is given in the following equation (see also Figure 9):

$$H = (MJ) * (CBR)^{-0.59} \qquad (17)$$

where H is the required thickness (in.) and MJ is a constant for a given traffic loading:

Road Section (in Dominica)	N18	MJ
Roseau-Hillsborough Bridge	16,000	30.0
Hillsborough Bridge-Pont Casse	36,000	32.0
Pont Casse-Hatton Garden	45,000	33.0

The existing pavement is a nonuniform granular waterbound macadam covered by a thin asphalt layer. The thickness of the granular material varies between 4 and 8 in., and the asphalt thickness is less than 1 in. The elastic modulus of the pavement (E*) varies mainly between 0.5 to 10 times the EØ of the subgrade. The lower values (E* = 0.5 to 2EØ) generally correspond to failed-to-poor sections.

For cases such as Dominica, where there is a large variability between the back-calculated E*/EØ ratios, it is necessary to bring the different sections to the same comparative basis. This is done by introducing the flexural stiffness concept. Flexural stiffness is a function of the thickness of the pavement and its modulus of elasticity and Poisson's ratio. Poisson's ratio is assumed to be constant for low-cost roads and to vary between 0.35 and 0.45. If the existing pavement with elastic modulus E* and thickness HC is equivalent to a new pavement with elastic modulus Ep and thickness HEQ, the following relationship between the flexural strength of the existing and the new pavement holds:

$$(E^*)\,(HC)^3 = (Ep)\,(HEQ)^3 \qquad (18)$$

HEQ in Equations 16 and 18 is the equivalent thickness of the new pavement.

The pavement structure of low-cost roads is constructed mainly from granular material such as subbase or base. The thickness of the AC is usually less than 2 in. In these cases the elastic modulus of the pavement structure (Ep) is derived from the elastic modulus of the granular material and must lie between 2 and 4 times EØ (11). In Dominica, the relationship of Ep = 3EØ was used to determine HEQ and DH', as defined in Equations 16 and 18. In simpler terms, if E* is found equal to 3 times EØ, then HEQ is equal to HC. Finally, if E* is greater than 3 times EØ, then HEQ is greater than HC. The value of HEQ gives credit to the flexural strength of the existing pavement.

DH' determined according to Equations 16-18 is in inches of granular material. When asphalt is used to overlay the pavement, DH' should be divided by an equivalency factor. This factor varies mainly between 1.5 and 4.0. According to the FAA (22), 1 in. of AC is equivalent to 1.5 in. of high quality base. According to the Asphalt Institute (3) and the Transportation Research Board (21), 1 in. of asphalt might be equivalent to 2 to 3 in. of granular material. The AASHTO (22) practice is that 1 in. of asphalt is equivalent to 3.2 in. of unstabilized base course or 4 in. of subbase. In the case of Dominica, it was found that the AC produced from the local aggregates has high strength and durability values. The Marshall stability is more than 2,000 lb, flow is 10 to 15, and immersion compression retained greater than 85 percent. Therefore, for this project in Dominica, where a high quality AC is designed, 1 in. of asphalt is equivalent to 3.5 in. of granular material, or the DH in thickness of AC is as follows:

$$DH = (H - HEQ)/3.5 = [(MJ)*(CBR)^{-0.59} - HEQ]/3.5 \quad (19)$$

The methodology for determining DH is described between lines 2900 and 3300 of the computer program.

SUMMARY

The computerized rational methodology of road rehabilitation presented in this paper was implemented in upgrading 50 km of a low-volume road in Dominica. A pocket computer (Sharp PC-1500) with 8-K RAM was used in the field to determine the minimum required AC overlay to carry 16,000 to 45,000 EALs. The following engineering parameters were calculated (see calculation example in Figure 2):

1. R50 (r50), the offset distance R at which the deflection ratio DR/DØ = 0.5 (see Figure 4);
2. LØ(ℓ), the characteristic length (Hogg model);
3. EØ, the subgrade elastic modulus;
4. CBR, the subgrade CBR;
5. E*, the combined modulus of the asphalt and the base layers;
6. E1 and E2, the modulus of the asphalt and the base, respectively;
7. HEQ, the equivalent thickness of a new pavement with E* = 3EØ; and
8. DH, the required AC overlay thickness.

The calculations of the AC overlay are done with a finite subgrade at a depth of h = 10ℓ (see Figure 4) or at infinity, h = ∞. The finite subgrade case is more often implemented and always results in lower values of the subgrade modulus and higher values of the pavement modulus in comparison with the infinite subgrade model. The AC overlay thick- ness DH is not sensitive to the subgrade depth. The use of the computer enables all the calculations, including the overlay thickness, to be completed in about 1 min. Therefore the rehabilitation or the overlay design can be completed while conducting the NDT. In Dominica, the NDT and the strengthening design of 50 km of rural roads were carried out simultaneously and completed in 1 week.

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