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Authors of the Papers in This Record

Artman, David H., Jr., USAF, c/o Dept. of Civil Engineering, University of Illinois, Urbana, Ill. 61801
Berdahl, G., Alberta Transportation, 9630 106th Street, Edmonton, Alberta, Canada T5K 2B8
Carmichael, Robert F., III, ARE, Inc., 2600 Dellana Lane, Austin, Tex. 78746
Cheetham, A., Pavement Management Systems, Ltd., RR 1, Paris, Ontario, Canada N3L 3E1
Christison, T.J., Civil Engineering Dept., Alberta Research Council, Terrace Plaza, 4445 Calgary Trail South, Edmonton, Alberta, Canada T6H 5R7
Connor, Billy G., State of Alaska, Department of Transportation and Public Facilities, 2301 Peger Road, Fairbanks, Alaska 99701
Darter, Michael I., Dept. of Civil Engineering, University of Illinois, Urbana, Ill. 61801
Fernando, Emmanuel G., Dept. of Civil Engineering, The University of Texas at Austin, Austin, Tex. 78712-1076
Gutierrez de Velasco, Manuel, Dept. of Civil Engineering, The University of Texas at Austin, Austin, Tex. 78712-1076
Haas, Ralph, Pavement Management Systems, Ltd., RR 1, Paris, Ontario, Canada N3L 3E1
Hartgen, David T., New York State Department of Transportation, 1220 Washington Avenue, State Campus, Albany, N.Y. 12232
Hudson, W.R., Dept. of Civil Engineering, The University of Texas at Austin, Austin, Tex. 78712-1076
Karan, M.A., Pavement Management Systems, Ltd., RR 1, Paris, Ontario, Canada N3L 3E1
Liebman, Judith S., Mechanical and Industrial Engineering Dept., University of Illinois, Urbana, Ill. 61801
Longenecker, K., Idaho Transportation Department, 3311 West State Street, P.O. Box 7129, Boise, Idaho 83707
Lytton, Robert L., Texas Transportation Institute, Texas A&M University, College Station, Tex. 77843
Mason, John M., Jr., Texas Transportation Institute, Texas A&M University, College Station, Tex. 77843
McCullough, B.F., Dept. of Civil Engineering, The University of Texas at Austin, Austin, Tex. 78712-1076
McHattie, Robert L., State of Alaska, Department of Transportation and Public Facilities, 2301 Peger Road, Fairbanks, Alaska 99701
O'Grady, James B., Transportation Department of the City of Arvada, 8101 Ralston Road, Arvada, Colo. 80004
Scullion, Thomas, Texas Transportation Institute, Texas A&M University, College Station, Tex. 77843
Shufon, John J., New York State Department of Transportation, 1220 Washington Avenue, State Campus, Albany, N.Y. 12232
Stanley, A., Idaho Transportation Department, 3311 West State Street, P.O. Box 7129, Boise, Idaho 83707

Optimization of Long-Range Major Rehabilitation of Airfield Pavements

DAVID H. ARTMAN, JR., JUDITH S. LIEBMAN, AND MICHAEL I. DARTER

A procedure has been developed to optimize the planning for major rehabilitative measures for airfield pavements. The procedure is divided into two parts. First, at the project level, an optimum long-range pavement rehabilitative plan is developed for each individual pavement feature (project) of the airfield for each of several levels of funding. The optimum rehabilitative alternative is selected by maximizing the pavement performance as defined by the weighted pavement condition index (PCI) versus time curve. The decision process is modeled by using dynamic programming. The second part of the analysis steps up to the network level of optimization. The criteria for selecting the set of projects is done by maximizing the pavements' performance weighted by the relative value of each project in the network. The rehabilitative projects are selected by using Toyoda's heuristic for 0-1 integer linear programming. The results of the airfield analysis are the selection and timing of major rehabilitative activities. The consequence of many funding levels and any directed work are also determined. Lastly, the pavement engineer can justify an optimum level of funding for long-range planning purposes. The methodology can also be applied directly to highways, roads, and city streets to provide long-range plans for better pavement management.

The U.S. Air Force has long recognized the need for effective airfield pavement management. This need led to the extensive effort contracted to the U.S. Army Corps of Engineers Construction Engineering Research Laboratory (CERL). The work at CERL produced several pavement management aids [e.g., Pavement Condition Index (PCI), Airfield Pavement Management System (APMS), and PAVER]. These aids provided U.S. Air Force pavement engineers with extensive data storage and retrieval plus data manipulative and presentation capabilities. By using these aids pavement engineers interactively develop and compare alternative plans for maintenance and rehabilitation of their airfield. These plans are based on the current use and performance of the airfield pavement feature. Inexperienced engineers can draw on the many years of valuable pavement engineering experience built into the system as they design projects; experienced pavement engineers use the system as a tool for extensive analysis and detailed comparison. The systems greatly enhance an engineer's abilities for comparing many more alternative designs objectively.

Yet, with all this enhanced capability, engineers have still been constrained in that all efforts in designing and comparing alternatives are directed at the present condition of the pavement. When asked what the best plan for the pavements is for the next 20 years, engineers have to rely on their previous manual techniques, their engineering judgment, and their experiences.

Not only are long-range planning capabilities limited to current decision making for each pavement section, but engineers cannot optimize these types of long-range plans at the network level (for an entire airfield or group of airfields). Since optimization of the expenditure of funds for the network of pavement sections over a specified period of time cannot be achieved, pavement engineers cannot be assured of getting the maximum performance of the entire network for a specified level of funding.

The work described in this paper is addressed to

1. Optimize the selection and timing of major rehabilitative measures over a specified period of time at a given funding level for individual pavement features (the project level) and

2. Optimize the selection of these measures at the network level, also with limited funding and for a specified time period.

Simply put, with a limited amount of money, the objective is to let the air force maintain its pavements in the best condition possible and to predict the performance of the pavement network given a limited amount of money.

AIRFIELD PAVEMENT FEATURE OPTIMIZATION--PROJECT LEVEL

U.S. Air Force bases are divided into separate pavement sections, called features. The number of features per base can be as low as 50 or as high as 200. Each feature is a unique element of the airfield and has its own construction and maintenance history, current traffic use, and relative need or importance for aircraft operations. In order to optimize the rehabilitation of the entire network of features, the first task must be to develop strategies for each individual feature as input into the overall network analysis. Each of these individual analyses is an optimization problem in itself, which consists of selecting possible rehabilitative measures, timing their occurrence within a fixed funding limit, and concurrently maximizing the performance of the feature.

Performance Criteria

One of the initial developments by CERL for the U.S. Air Force was PCI (1,2). Well-established in the air force as the standard measure of airfield pavement distress, PCI is an objectively derived value from 0 to 100, highly correlated to the engineering judgment of many air force pavement engineers. The engineers based their subjective rating on their experiences in airfield pavement maintenance and the standards levied on them by aircraft operations (Figure 1). Some states are using PCI for airfield pavement rating (Illinois) on civilian airports.

In addition to the basic index, the PCI scale also has a relative utility. The utility of the PCI is the relative value of a particular unit point of PCI to another unit point of PCI elsewhere on the scale. To illustrate, the value of raising the PCI of a particular pavement feature from 50 to 60 is very different from that of raising the PCI from 90 to 100. With utility, this change or difference in value up and down the PCI scale can be evaluated (Figure 2). As the PCI gets larger, the utility of the unit PCI point diminishes.

The performance measure of a particular feature over time is defined as the area under the utility weighted PCI versus time plot [Figure 3 (3)]. The area is called the nonmonetary benefit or performance of a particular rehabilitation policy for the analysis period. The larger this area, the better the structural performance of the pavement section.

Part of CERL's work included the development of a PCI prediction model. The current models (4) predict the future PCI of a particular pavement feature within a range acceptable to the U.S. Air Force for use in planning, programming, and budgeting. These

Figure 1. Summary of steps for PCI determination.

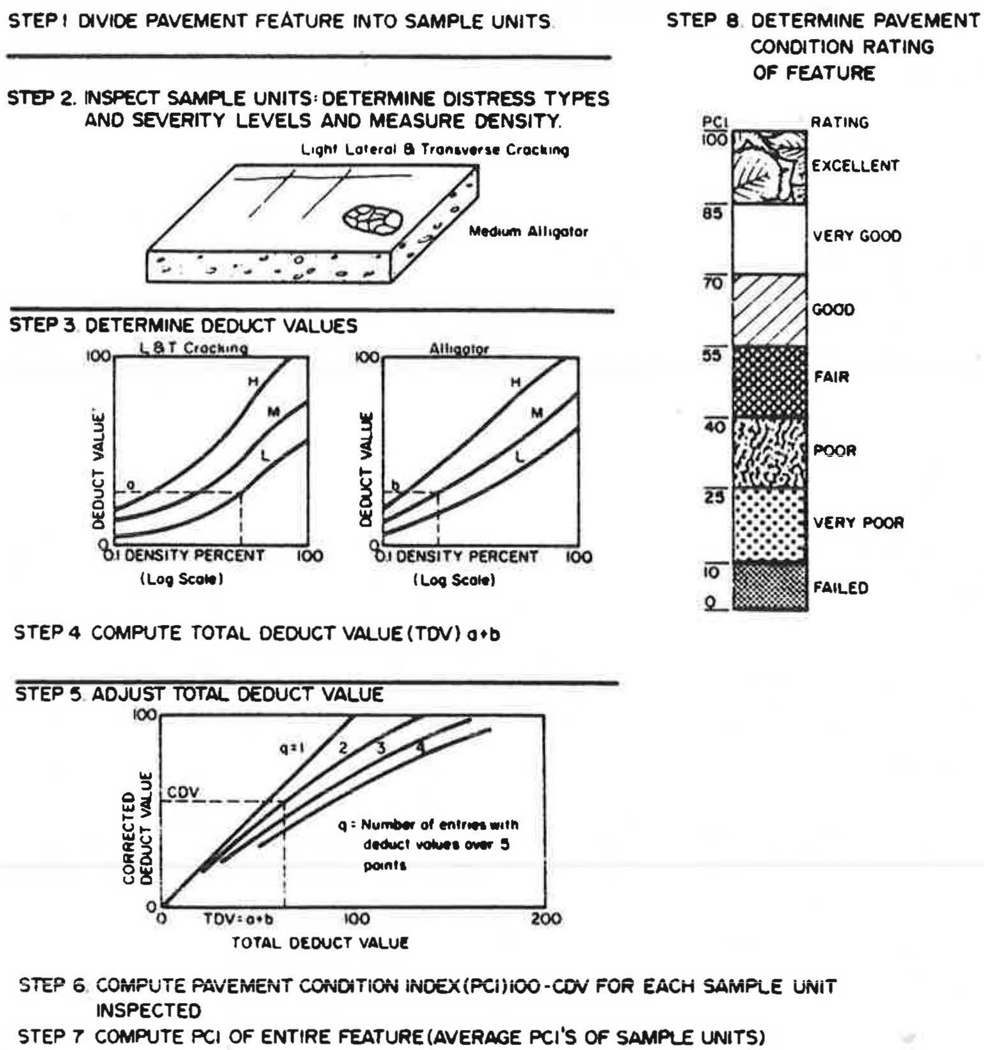


Figure 2. Utility weighted PCU versus time curve.

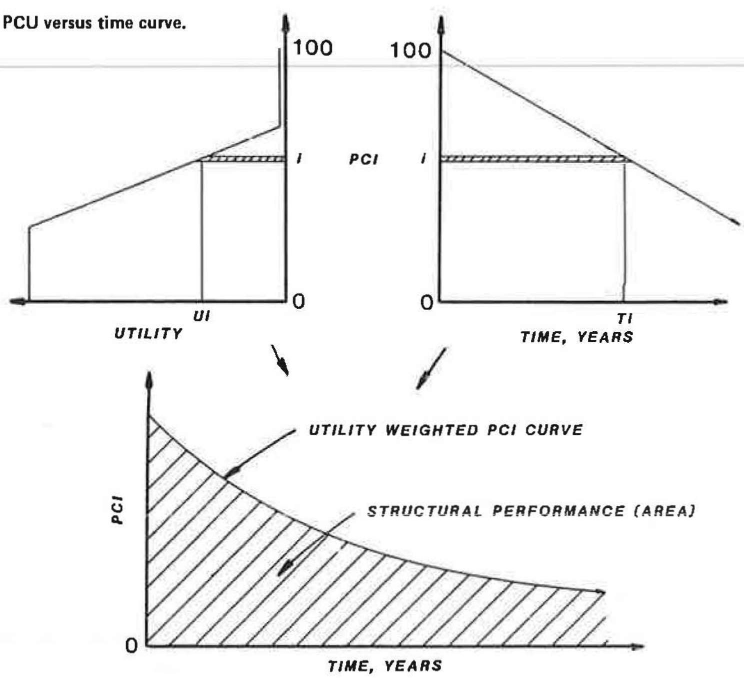


Figure 3. Performance of rehabilitative activity.

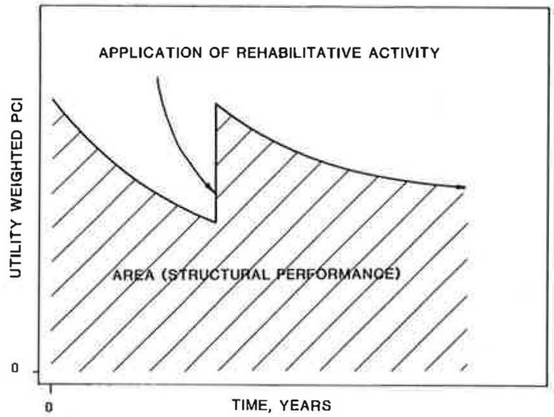
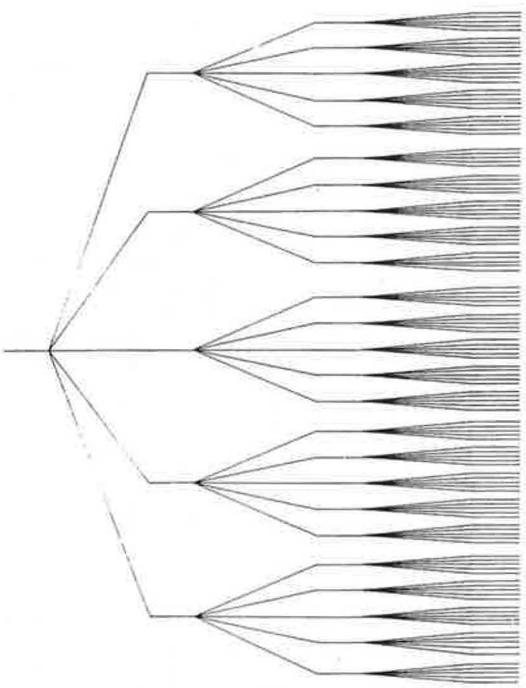


Figure 4. Decision tree of project decision process.



models lend themselves to the prediction of major rehabilitative activities (overlays and reconstruction) and their timing in light of the level of routine maintenance the air force was using during the development of the models.

Feature Decision Process

The decision process for selecting the best rehabilitative activity and its timing for a particular feature becomes an extensive decision tree when expanded over a period of time (20 years). If the decision process included 5 possible alternatives and were considered every 2 years for a period of 20 years, the number of possible combinations of decisions would be 5^{10}. Figure 4 depicts the complexity of the decision tree over just 3 decision periods.

With such a large number of possible combinations of decisions, even the most efficient computer in existence would take more than 10 days to enumerate all possible decision paths. However, because of the nature of this decision process, an algorithm called dynamic programming can be used to determine the optimal rehabilitative policy in a reasonable amount of computer time without enumerating all possibilities.

The term dynamic programming, first used by Bellman, denotes a mathematical method to solve a multistage decision process (5). When properly applied, dynamic programming reduces the problem size and still guarantees an optimal or best solution within the bounds of the models used. In this case, the 5^{10} possible combinations of decisions over 20 years reduces to 5 x 5 x 10 possible combinations of decisions for the same period.

The decision process of pavement rehabilitation at the feature (project) level is modeled as a series of staged decisions (every 2 years for 20 years). At each stage (decision point) all the feasible decisions (e.g., routine maintenance, reconstruction, and overlays) are applied to each entering state (previous combination of possible decisions). Only the decision that gives each of the entering states its maximum benefit or performance (area under the utility weighted PCI versus time plot) over the next decision period (2 years) is retained and passed on to the next stage (decision point). Figure 5 is the dynamic programming flow chart for the decision process in this procedure. This property of the dynamic programming algorithm permits reduction of the decision tree to a feasible size for computer solution. This is illustrated in Figure 6, with the dynamic programming methods applied to the decision tree shown in Figure 4.

Dynamic Programming Inputs

Basic pavement and aircraft data are necessary inputs into the dynamic programming algorithm. Those data are readily available at each U.S. Air Force base either in the form of reports (condition survey reports or pavement evaluation reports) or drawings (master plans). Reconstruction designs for each feature (project) are standard designs from current U.S. Air Force manuals. These designs are functions of the structural parameters of the existing feature (obtained from reports and drawings) and current (or anticipated) aircraft use. Costs for either reconstruction or overlays are based on current average pricing used for planning purposes. As these alternatives are selected as future decisions, their costs are adjusted for inflation.

Dynamic Programming Execution

The current dynamic programming algorithm averages approximately 3 central processing seconds on a CDC Cyber 175 computer for each feature (project) in the network. One feature (project) at a time, the input data are read in and the long-range rehabilitative plans are developed until all the features in the network have been analyzed.

The program is operated in an interactive mode and the feature information is read from a data input file. The results of all the features are output into a single data file.

Dynamic Programming Results

The output from the dynamic programming algorithm is a series of long-range rehabilitation policies for different levels of funding. The first policy is always the routine maintenance policy. That policy reflects the resulting performance (area under the utility weighted PCI versus time plot over 20 years) and distress condition if only routine maintenance

Figure 5. Dynamic programming model flow chart.

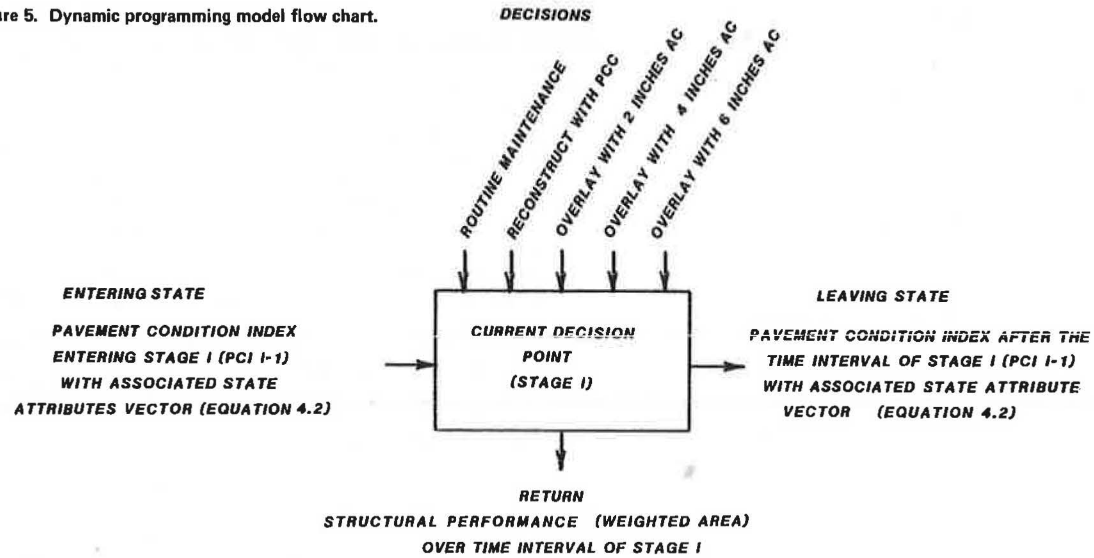
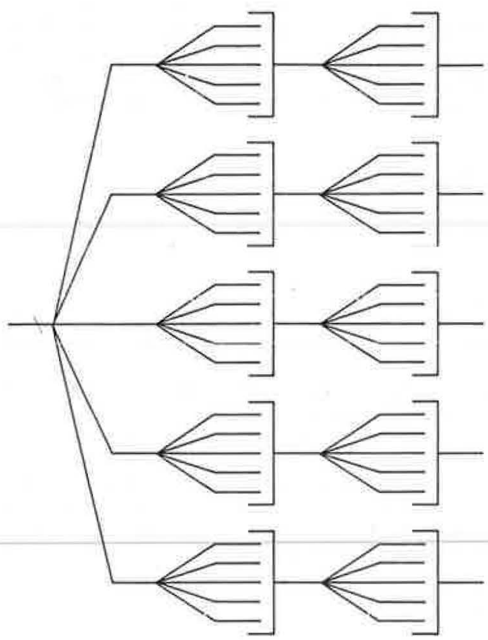


Figure 6. Dynamic programming method applied to project-level decision process.



is applied over the analysis period (20 years). Even though no expenditure of funds for major rehabilitation is made over the period, a benefit (performance) is still obtained. The other policies reflect rehabilitative plans that maximize the performance over the analysis period for different levels of funding. At each level of funding the optimal plan provides the type and timing of major rehabilitation for a feature.

Major rehabilitative activities include

1. Routine maintenance,
2. Reconstruction with portland cement concrete (PCC),
3. Overlay with 2 in. of asphalt concrete (AC),
4. Overlay with 4 in. of AC, and
5. Overlay with 6 in. of AC.

In addition, the output provides the increments of performance and the features' PCI as a function of time over the analysis period.

AIRFIELD OPTIMIZATION--NETWORK LEVEL

Air force airfield pavement networks suitable for network optimization can be defined in several ways. First, a network that encompasses all the airfields in the entire U.S. Air Force inventory would include several hundred airfields around the world. Another network could be defined to include only those air force bases within a single major air command [e.g., Strategic Air Command (SAC), Military Airlift Command (MAC), Tactical Air Command (TAC), or U.S. Air Forces in Europe (USAFE)]. This division of U.S. Air Force airfields would entail sizes from 6 to 20 airfields/network. Funds are allocated at both the air force and command levels for airfield pavement rehabilitation; therefore, either of these would be a logical division for network funding optimization. A third logical network level is the individual airfield itself. If the network is defined at the single airfield level, each base has the opportunity to plan long-range strategies that reflect its specific needs. The base engineers are the most familiar with their unique pavement problems and specific operational needs and are responsible for the performance of their airfield pavements; therefore, the airfield level of network optimization is also very important.

This paper addresses the airfield network optimization; however, the methods developed and explained here are directly applicable to higher levels (command and air force) of optimization. The objective of airfield network optimization is similar to individual feature (project-level) optimization: Within given funding restraints, maximize the total network performance over the analysis period.

Performance Criteria

The criteria used in optimizing the expenditure of limited funds can take several forms at the network level. First, the same criteria used at the feature-level optimization can be reflected at the network level. For all the feature plans submitted to the network optimization, maximization of the summation of the performance (area under the utility weighted PCI versus time plot) for all the selected feature plans, constrained by a limited funding level, is a feasible criterion. With this method engineers can obtain the maximum performance (per the defined criterion) of their networks for a specified level of funding.

However, obtaining the maximum performance (as previously defined) regardless of the value or need of a particular pavement section to the aircraft operations of an airfield might not provide the best pavement system over the analysis period. This is because all the features in an airfield have a relative worth as compared with each other for supporting the aircraft operations. For example, the taxiway used by B-52 bombers standing strategic alert has a higher value than the taxiway used for access to an engine test cell. The primary runway that supports fighter operations has a higher value than the ladder taxiway of an adjacent auxiliary runway. To keep the engine test cell taxiway or the auxiliary runway ladder taxiway in tip top shape but allow the alert bomber taxiway and primary fighter runway to deteriorate to high distress levels just because this might be easier or cheaper would not be in the best interest of the users. Hence, relative feature value must be incorporated when optimizing at the network level.

The relative value of a particular feature can be broken down as a function of the following variables:

1. Pavement type (e.g., runway, taxiway),
2. Pavement need (e.g., primary, secondary),
3. User aircraft type (e.g., bomber, fighter, cargo),
4. User aircraft mission (e.g., alert, training, operational), and
5. Number of user aircraft by type and mission.

Together these variables describe the relative worth of one particular pavement feature versus another. The features being compared can be on the same airfield or different airfields around the world. Table 1 gives the complete breakdown of each category. The relative numerical value assigned to each element is also included. These weights have been estimated based on 6 years of experience in air force pavement management, but they should be validated before implementation.

The worth of a single feature can be calculated by

$$\text{Worth} = \sum_{i=1}^{n} N_i*(PT_i + PN_i + AT_i + AM_i) \quad (1)$$

where

N_i = number of user aircraft by type and mission,
PT_i = pavement type coefficient for user aircraft,
PN_i = pavement need coefficient for user aircraft,
AT_i = user aircraft type coefficient,
AM_i = user aircraft mission coefficient,
i = counter of different user aircraft types, and
n = number of different aircraft types that use feature.

All the information necessary for the worth calculation is readily available at each U.S. Air Force base. If the relative worth of a single feature becomes the criterion used at the network level for optimizing the expenditure of funds, then the pavement engineer can maximize the user's needs. In this case the limited funds are spent without regard to getting the most pavement structural benefit for the expenditure. Because different long-range rehabilitative plans from the same feature have the same feature worth to the total network system, the network optimization will choose the cheapest rehabilitative plans within the specified network funds. As

Table 1. Relative weights of total feature worth calculation.

Characteristic				
Feature	Relative Weight	Criteria	Relative Weight Within Criteria	Relative Weight[a]
Aircraft mission	10	Alert	100	1,000
		Operational	50	500
		Training	10	100
Pavement need	5	Primary	100	500
		Secondary	20	100
		Auxiliary	5	25
		Transient	2	10
		None	1	5
Pavement type	2.5	Runway	100	250
		Taxiway	30	75
		Apron	10	25
Aircraft type	0.5	Bomber	100	50
		Command	50	25
		Tanker	30	15
		Cargo	20	10
		Fighter	10	5
		Transport	8	4
		Reconnaissant	6	3
		Trainer	4	2
		Experimental	2	1

[a]Product of characteristic and criteria weights.

discussed, criteria might not always be in the best interest of the pavement structure.

These two optimizing criteria represent the extreme limits, ranging from do the best for the pavement to do the best for the user. The correct optimization criteria rest somewhere in between. For the work in this paper, the features' total benefit is weighted by the features' worth parameter.

Optimization at the network level with the feature-worth weighted performance of each long-range rehabilitative plan takes into consideration both the importance of the feature to the user and the effect of the plan on the performance of the pavement. These are the criteria used in the examples in this paper.

Network Decision Process

The decision process for optimizing the selection of long-range plans can be modeled very simply as a 0-1 integer linear programming problem. The integer program algorithm selects the plans (one plan per feature) that provide the largest summation of feature worth and plan performance product. The problem is formulated as

$$\text{Maximize: } \sum_i \sum_j P_{ij}*FW_i*PP_{ij}$$

such that

$$\sum_i \sum_j P_{ij}*C_{ij} \leq \text{network funding limit}$$

and

$$\sum_i P_{ij} \leq 1 \quad \text{for all } j \text{ (limits one selected plan per feature)}$$

where

FW_i = feature worth of ith feature,
PP_{ij} = plan performance of jth policy of ith feature,
C_{ij} = cost of jth policy of ith feature, and
P_{ij} = jth policy of ith feature (the decision variables), equals 1 if selected or 0 if not selected.

Integer Program Inputs

For optimizing at the network level, the required inputs for all the long-range rehabilitative plans are

1. Cost of rehabilitative plan for each feature,
2. Performance of the rehabilitative plan,
3. Relative worth of the plan's feature, and
4. Funding limit for network expenditure.

All the inputs are either set by the pavement engineer at the time of execution or are output from the dynamic programming algorithm. Also, in addition to setting the funding limit at the time of execution, pavement engineers can choose the optimizing criteria. Choices for optimizing criteria (objective function) include

1. Structural performance of individual feature plans (area under utility weighted PCI versus time plot),
2. Relative feature worth (function of user aircraft type and mission plus type and need of feature to the user aircraft), and
3. Structural performance weighted by relative feature worth (performance times worth).

Integer Program Solution

Solutions to integer programming problems normally require checking (either directly or indirectly) every possible combination of solutions. Small problems (in our case a small problem is 10 features with 3 or 4 plans or long-range strategies each) are easily solvable on most computers. But, as the size of the problem approaches an air force base network with possibly 100 features with 3 to 4 plans each,

Figure 7. Selected project listing example.

```
**********************************
*                                *
*    SELECTED PROJECT LISTING    *
*                                *
*      NETWORK OPTIMIZATION      *
*                                *
**********************************
*                                *
*        DECISION  LEGEND        *
*                                *
*  R/M  = ROUTINE MAINTENANCE    *
*  R/C  = RECONSTRUCT            *
*  O/L2 = OVERLAY WITH 2" AC     *
*  O/L4 = OVERLAY WITH 4" AC     *
*  O/L6 = OVERLAY WITH 6" AC     *
*                                *
**********************************
```

NETWORK DESCRIPTION: EXAMPLE PROBLEM WITH OPTIMIZED WITH TFW*PERFORMANCE

NETWORK SPENDING LIMIT(PRESENT WORTH): 250000

AMOUNT SPENT(PRESENT WORTH): 246746

OBJECTIVE FUNCTION VALUE: 327956890

BASE	FEATURE	TOTAL COST	BENEFIT	WORTH
EXAMPLE	R03A	185435	2214	48170

TIME(YEARS)	0	2	4	6	8	10	12	14	16	18	20
PCI	80	78	76	74	72	70	98	96	95	93	91
DECISION	R/M	R/M	R/M	R/M	R/M	O/L4	R/M	R/M	R/M	R/M	R/M
COST($)	0	0	0	0	0	185435	0	0	0	0	0

BASE	FEATURE	TOTAL COST	BENEFIT	WORTH
EXAMPLE	T01A	23512	2049	41170

TIME(YEARS)	0	2	4	6	8	10	12	14	16	18	20
PCI	60	57	96	92	88	84	80	76	71	66	61
DECISION	R/M	O/L2	R/M	R/M	R/M	R/M	R/M	R/M	R/M	R/M	R/M
COST($)	0	23512	0	0	0	0	0	0	0	0	0

BASE	FEATURE	TOTAL COST	BENEFIT	WORTH
EXAMPLE	T03C	17951	2035	22810

TIME(YEARS)	0	2	4	6	8	10	12	14	16	18	20
PCI	75	73	71	68	95	89	84	79	74	68	63
DECISION	R/M	R/M	R/M	O/L2	R/M	R/M	R/M	R/M	R/M	R/M	R/M
COST($)	0	0	0	17951	0	0	0	0	0	0	0

BASE	FEATURE	TOTAL COST	BENEFIT	WORTH
EXAMPLE	T05A	19848	2199	41170

TIME(YEARS)	0	2	4	6	8	10	12	14	16	18	20
PCI	75	73	70	68	97	95	92	89	86	84	81
DECISION	R/M	R/M	R/M	O/L2	R/M	R/M	R/M	R/M	R/M	R/M	R/M
COST($)	0	0	0	19848	0	0	0	0	0	0	0

evaluation of each and every possible combination becomes infeasible. The number of possible combinations of network solutions exceeds 3^{20} with just 20 features and 3 plans per feature.

Total or implicit enumeration is infeasible; therefore, a heuristic must be used to solve the resulting large integer program optimization. A heuristic procedure (providing good but not necessarily optimal solutions) developed by Toyoda (6) has been used by Ahmed (7), and Phillips and Lytton (8) for their procedures in allocating state highway resources during a single period. This same procedure by Toyoda can be used for selecting the optimal set of policies developed for long-range rehabilitation. The procedure uses an effective gradient search to select the best feature plans. For the sample problems solved in this research the heuristic solved for more than 90 percent of the objective function value when compared with solving for a global solution by using normal linear programming.

Integer Program Execution and Results

The current integer programming algorithm uses 20 to 40 central processing seconds to solve each level of expenditure at the network level on a CDC Cyber 175 computer (for 10 features, averaging 3 plans each). The program is rerun for different levels of expenditure, which results in the optimal set of feature plans for each level of expenditure.

The output from the integer programming heuristic is a listing of selected feature plans giving the maximum objective function within the network funding limit. Each set of feature plans represents the best group of long-range rehabilitative plans at selected budget levels, and also strives to serve the user and pavement structure in the best way possible.

Analysis Results

On completion of both the feature- (project) and network-level optimizations, the output data are summarized and presented in a user-readable format. Three tables are formed for each level of funding at the network optimization level:

1. Selected project listing,
2. Features without project listing, and
3. Network summary.

Selected Project Listing

The selected project listing is a summary of all the long-range rehabilitative feature plans selected by the network optimization. Each listing represents the optimal group of plans that does not exceed the specified funding limit. The listing provides the amount of money spent out of the amount allotted plus a summary description of each selected long-range plan.

Figure 7 is a typical example of the selected project listing. After identifying the airfield and feature of the selected long-range plan, the total cost of the plan (present worth dollars), total benefit (performance, area under utility weighted PCI versus time plot), and total feature worth (relative value of feature to aircraft operations, function of aircraft mission and type, plus pavement type and need) are shown. The predicted PCI, selected decisions, and costs (present worth) are given as a function of time (years) from the present. The information is repeated for each selected long-range plan as shown in Figure 4. A separate listing is developed for each specified limit of network funding.

Features Without Projects

The features without projects listing (Figure 8) summarizes all the features in the network that did not receive funding for any of their long-range rehabilitative plans. Each feature is identified with its total benefit (performance) and relative value, plus the PCI as a function of time for the analysis period. Note that because an air force airfield pavement is considered in poor condition when its PCI reaches 40, the features that fall in this category during the analysis period (this case 20 years) are flagged and their PCIs are asterisked. Again, a separate listing is generated for each specified level of network funding.

Figure 8. Features without projects example.

FEATURES WITHOUT PROJECTS

NETWORK DESCRIPTION: EXAMPLE PROBLEM WITH OPTIMIZED WITH TFW*PERFORMANCE

NETWORK SPENDING LIMIT(PRESENT WORTH): 250000

AMOUNT SPENT(PRESENT WORTH): 246746

FEATURES FALLING BELOW PCI OF 40 ARE FLAGGED AND PCI'S ARE ASTERISKED

	BASE	FEATURE	BENEFIT	REL VALUE	PCI BY YEAR										
					0	2	4	6	8	10	12	14	16	18	20
<40	*EXAMPLE*	R01A	1127	48170	65	62	58	54	50	45	41	36*	31*	26*	21*
	EXAMPLE	R02C	1654	48170	75	73	71	68	66	64	62	60	58	55	53
<40	*EXAMPLE*	T02A	1128	41170	65	62	58	54	50	45	41	36*	31*	26*	21*
	EXAMPLE	T04C	2032	22810	85	84	82	81	80	78	77	76	75	73	72
	EXAMPLE	A01B	2007	16420	85	84	82	81	79	78	76	75	73	71	70
<40	*EXAMPLE*	A02B	1089	22750	60	57	53	50	46	43	39*	36*	32*	29*	25*

Figure 9. Network summary example.

NETWORK SUMMARY

NETWORK DESCRIPTION: EXAMPLE PROBLEM WITH OPTIMIZED WITH TFW*PERFORMANCE

NETWORK SPENDING LIMIT(PRESENT WORTH): 250000

AMOUNT SPENT(PRESENT WORTH): 246746

TIME(YEARS)	0	2	4	6	8	10	12	14	16	18	20
PCI(WEIGHTED)	71	69	71	68	71	67	68	65	62	58	55
COST(PRESENT WORTH)MILLIONS	0.00	.02	0.00	.04	0.00	.19	0.00	0.00	0.00	0.00	0.00
ACCUM COST,MILLIONS	0.00	.02	.02	.06	.06	.25	.25	.25	.25	.25	.25
NEW FEATURES W/PCI<=40	0	0	0	0	0	0	1	2	0	0	0
POOR FEATURES(PCI<=40)	0	0	0	0	0	0	1	3	3	3	3
BENEFIT	0	1844	1887	1817	1910	1827	1823	1736	1650	1565	1475
ACCUM BENEFIT	0	1844	3731	5548	7458	9285	11108	12844	14494	16059	17534

Network Summary

The last product of the network optimization procedure is the network summary. For each specified limit of funding the summary lists the following as a function of time:

1. Weighted network mean PCI,
2. Cost of rehabilitation (present worth),
3. Accumulated cost of rehabilitation,
4. New features that become poor (PCI = 40),
5. Total number of poor features in network,
6. Total summed benefits (performance) of all features, and
7. Accumulated summed feature benefits.

Figure 9 is a typical example of a network summary listing. The network PCI is the average of all features weighted by the relative worth of each feature. The cost is the amount of money spent in present worth values at each interval of time in the analysis period. The new features with PCI less than or equal to 40 represent the number of features that became poor during the respective decision points (time intervals) in the analysis period. The poor features listing gives the total number of features that have a PCI less than or equal to 40. The benefit is the increment of benefit or performance (area under utility weighted PCI versus time plot) summed for all the features for the individual time increments of the analysis period. Accumulated benefit is the summation of the individual time increment network benefits.

SAMPLE AIRFIELD APPLICATION

A small sample airfield was analyzed with this procedure. The sample airfield has only 10 features so the size of the problem does not hinder understanding the decision process (Figure 10). The inputs for each feature were extracted from real U.S. Air Force base features subjected to light load aircraft. All the feature data were run through the dynamic programming algorithm and these results were input into the Toyoda network analysis programs. All reports were generated for funding levels between \$0 and \$3 million present worth (at \$250,000 intervals). The resulting objective function values are plotted in Figure 11.

Figure 10. Layout of example airfield.

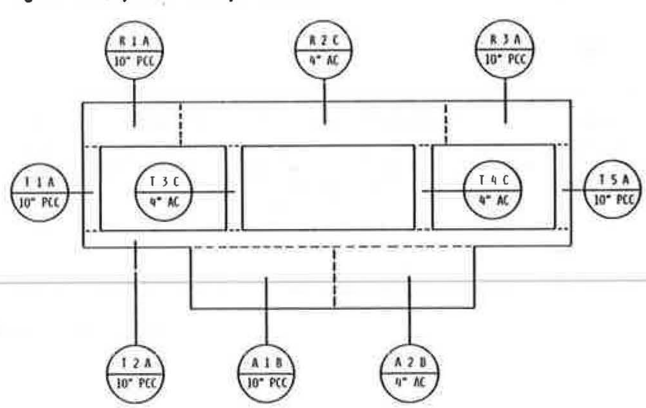
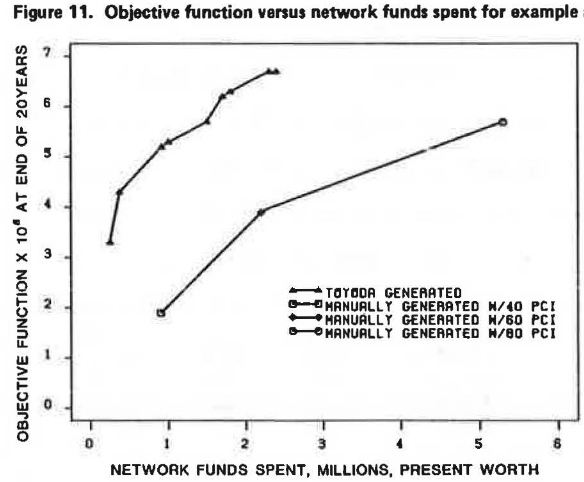


Figure 11. Objective function versus network funds spent for example airfield.



As a comparison, the same sample airfield was managed manually. By using three different condition levels (PCIs of 40, 60, and 80) a separate network analysis was completed. An activity (2-in. asphalt overlay) was scheduled for any feature on the sample airfield when its condition reached the preestablished minimum condition level (40, 60, or 80 PCI). The analyses were carried out for 20 years (the same as for the Toyoda analyses). For each of the three analyses, the objective function value was calculated and also plotted on Figure 11. Note that in each case, for the same amount of money spent, the methods developed in this research nearly double the objective function values obtained manually. Or, from another perspective, for any level of the objective function, the cost of the optimally selected projects was less than half the cost of the manually selected methods to achieve the same objective function.

Figure 12. Terminal PCI at 20 years versus network funds spent, McClellan Air Force Base.

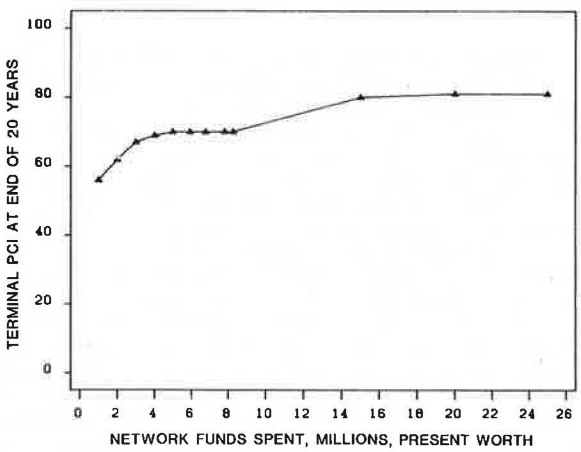


Figure 13. Network performance at 20 years versus network funds spent, McClellan Air Force Base.

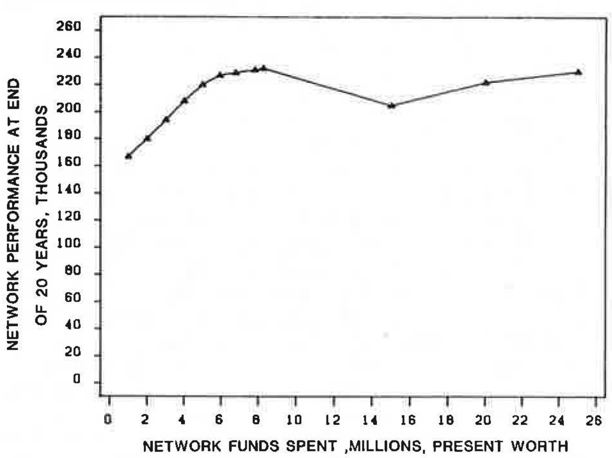
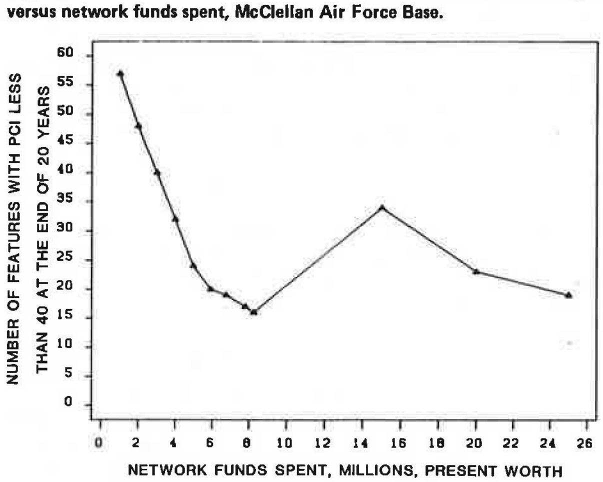


Figure 14. Number of features that have PCI less than 40 at end of 20 years versus network funds spent, McClellan Air Force Base.



Examination and comparison of the results (which feature activities were scheduled at various funding levels) for both methods revealed several reasons for the vast differences illustrated in Figure 11:

1. Activity assignment was optimized at the project level for the method derived in this research,
2. Manual network analysis selected projects regardless of the relative value of a feature to the user, and
3. Manual analysis does not take into consideration the structural benefit (performance) of a selected activity schedule (project).

This small example illustrates the value of the program developed in this research: it provides substantially better ways of spending the same money and maintaining an established condition for a reduced amount of money.

EXAMPLE NETWORK ANALYSIS

To illustrate the use of the programs developed, a U.S. Air Force base was selected, analyzed, and is presented as an example. The selected U.S. Air Force base is in California and serves all sizes of aircraft in the inventory. The airfield has 113 features, a mixture of both flexible and rigid pavements.

All of the airfield's features were loaded into the feature analysis dynamic programming algorithm. The resulting optimized long-range feature plans were automatically processed into a condensed data file and input into the network optimization program. The network was optimized for funding levels from $0 to $25 million (present worth). Reports were generated at all funding levels and used for the following discussion.

Pavement engineers can use the reports generated from the example described to show justification for increased funding or consequences of decreased funding of the long-range rehabilitative plans for their airfield. They can modify their analysis and make additional runs to compensate for directed work (ordered independent of the analysis). This modified analysis can show the consequence of the directed work on the network airfield. Figure 12 is a plot of the network PCI at the end of the analysis period as a function of network funding level. At approximately $5 million (present worth), the terminal network PCI reaches an asymptotic value close to 70. Figure 13 plots the network structural performance at the end of the analysis period. Note that it also reaches an asymptotic value at approximately $5 or $6 million. Figure 14 shows the number of features that fall below a PCI of 40 by the end of the analysis period as a function of network funding level. The amount of decrease of the total number of features in this category remains constant until approximately $5 million. At this point and beyond, additional monies are spent in previously funded features due to their relative value to the user. Figure 15 depicts the network objective function versus network funds spent. It is a smooth continuous curve as expected and reaches an asymptotic value of about $6 million.

With these findings air force pavement engineers can not only show justification for adjustments to a proposed plan of the long-range rehabilitative measures for an airfield, but they now have justification for setting the approximate optimum level of network funding for the analysis period. Looking at Figure 16, in this case $5 or $6 million present worth would maintain the airfield at a level most advantageous to both the sustained performance of the pavement structures and the sustained normal operations of aircraft.

Figure 15. Objective function versus network funds spent, McClellan Air Force Base.

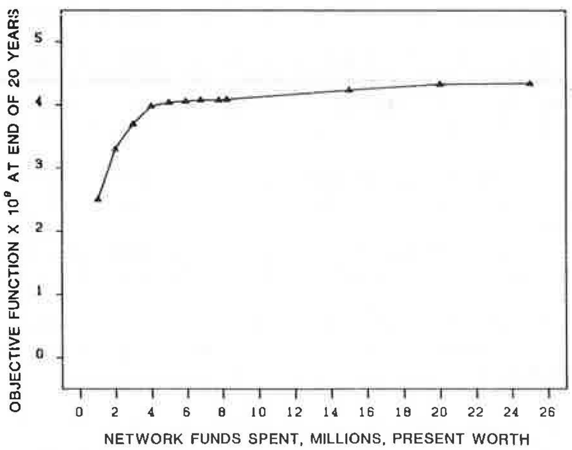


Figure 16. Network consequences versus network funds spent, McClellan Air Force Base.

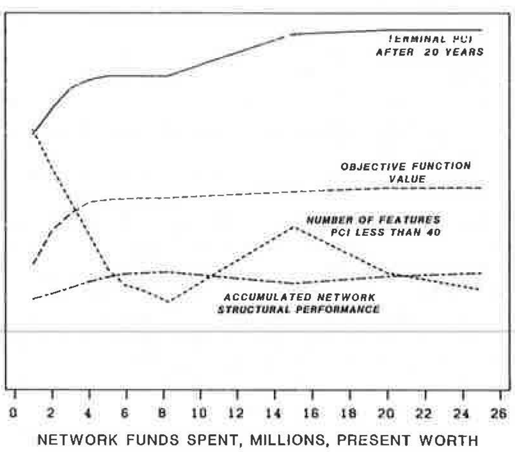
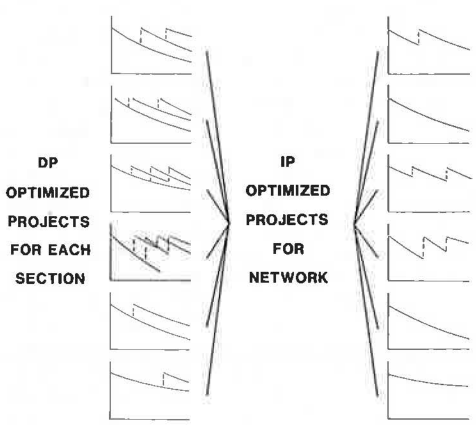


Figure 17. Graphical presentation of entire decision process.



SUMMARY

The programs and procedures developed during the course of the research described in this paper focus on the long-range planning of only major rehabilitative measures (routine maintenance, reconstruction, and overlays) of military airfields. The dynamic and integer programming procedures make it possible to solve large decision processes far beyond the abilities and comprehension of pavement engineers. This approach provides pavement engineers with the resources to consider many different alternative plans and rationally select the series of major rehabilitative measures that maximize the performance of the pavement structure (area under the utility weighted PCI versus time curve). The engineers now can comprehensively consider the possible decisions available for implementation as rehabilitative measures and can identify the unique plan that is best for each pavement feature at a specified funding level.

After developing the optimal long-range feature plans, airfield pavement engineers can use this information for the basis of long-range planning. Figure 17 shows the process from generating optimal projects for many features to selecting the best of these projects at the network level.

In the case of the example, the pavement system to be planned for was defined as a single airfield with 113 features with the dynamic program developing approximately 3 long-range feature plans each. The Toyoda algorithm was used to select among these feature plans. Air force pavement engineers can develop an optimal long-range plan for all the features of an airfield at specified funding levels.

Reports generated by this procedure of feature-level and network-level optimization describe what work is to be done and when to schedule it for all the features in the network. They reveal the consequence of this work not only on the features with planned projects, but also the impact on those features without any scheduled work. When directed to accomplish work not scheduled, the engineer can show the consequence of such work on the system as a whole and, if still directed, can reoptimize the use of the remainder of the funds. The report listings summarize the results of the long-range plan with respect to total network composite parameters of condition, cost, and performance. Furthermore, all of this information is generated at each funding level to be considered. Further details of this study may be found elsewhere (9).

ACKNOWLEDGMENT

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Development and Implementation of Alberta's Pavement Information and Needs System

M.A. KARAN, T.J. CHRISTISON, A. CHEETHAM, AND G. BERDAHL

Alberta Transportation initiated a project in November 1980 to develop and implement a pavement management system (PMS) for the province of Alberta, Canada. A comprehensive project plan was developed in the first phase of the project, which commenced in November 1980 and was completed in January 1981. Carried out as a preplanning project, the first phase identified six successive stages for the overall total PMS development and implementation project. Stage 1 of the project, the development and implementation of a pavement information and needs system (PINS), was initiated in May 1981 and scheduled to be completed in September 1982. A major element of PINS is a set of models that predict performance and various data processing and analysis components that take the individual field meansurements; calculate the performance measures in terms of pavement quality index, riding comfort index, structural adequacy index, and visual condition index; apply the performance prediction models; and identify both current and future needs. The major features of the PINS system and how the system fits into Alberta's overall PMS development and implementation are described. Specific attention is given to the details of performance prediction modeling and development of a pavement quality index concept.

Alberta Transportation is responsible for the management of a large network of provincial highways that consists of approximately 7,000 miles of paved primary highways and about 2,000 miles of paved secondary roads. In addition, approximately 200 miles of new pavement are added to the highway system annually. This represents a substantial investment of many millions of dollars. To preserve this investment and maintain an acceptable level of serviceability for the total highway network, an additional investment of approximately $50 million is required annually for the maintenance and rehabilitation of deteriorating highway sections.

The department's engineers and administrators are concerned that the rehabilitation and maintenance programs make the best possible use of available funds on an overall basis as well as ensure an equitable allocation between the regions in the province. To establish an objectively based rehabilitation program several questions must be answered:

1. What is the current status of the network?
2. What are the expected needs during the programming period?
3. What rehabilitation alternatives can be considered for sections that require action within the programming period?
4. What are the performance and cost implications associated with the possible rehabilitation alternatives?
5. What is the effect of delaying or advancing a rehabilitation project within the programming period?
6. What are the effects of maintenance on the rehabilitation alternative selection?
7. What is the optimum total program of work for each year in the programming period based on the previous questions for a given level of funding?
8. What are the effects of the funding level used on the network as a whole?
9. What level of funding is required to maintain or increase the average serviceability of the network during the programming period?

Pavement management is the process by which answers to these questions can be obtained; Alberta Transportation initiated a project in November 1980 to develop and implement a pavement management system (PMS) for the province of Alberta.

A comprehensive plan was developed in the first phase of the project, which started in November 1980 and was completed in January 1981 (1). Carried out as a preplanning project, the first phase identified six successive stages for the overall total PMS development and implementation project. These stages, which are briefly summarized in Figure 1, were designed specifically for Alberta Transportation's needs and requirements considering its goals and objectives, organizational structure, current

Figure 1. Proposed stages of project.

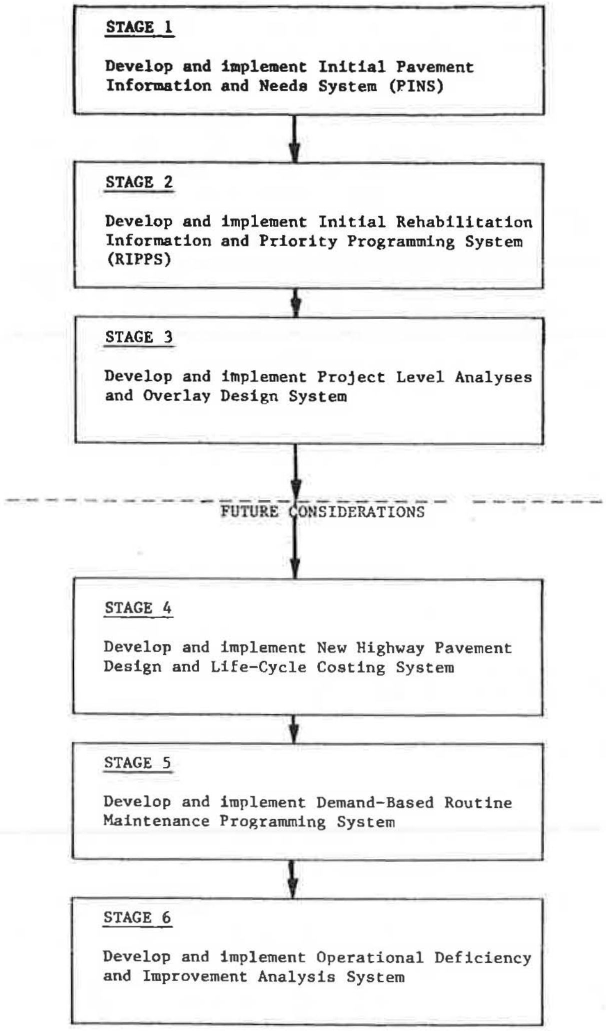
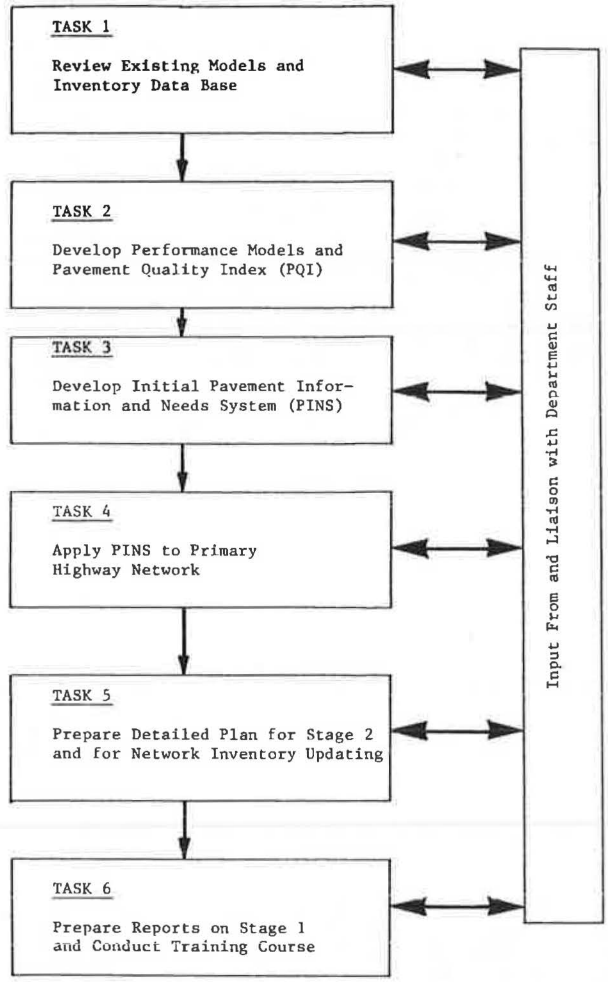


Figure 2. Major tasks in stage 1.



practices, staff and equipment resources, and financial constraints.

Stage 1 of the project, the development and implementation of a pavement information and needs system (PINS), was initiated in May 1981 and scheduled to be completed in September 1982. The main objective of this paper is to summarize the development of the PINS system with specific attention to the details of performance prediction modeling and development of a pavement quality index (PQI) concept.

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: PINS. The preplanning project produced a work plan for stage 1 using a series of tasks and subtasks. The end product of each task was also identified. The major tasks and their interrelationships are shown in Figure 2; the subtasks involved in stage 1 are shown in Figure 3.

TASK 1: REVIEW EXISTING MODELS AND INVENTORY DATA BASE

The first task undertaken in stage 1 was (a) to review Alberta Transportation's existing methodologies, information, and hardware and (b) to assess these in terms of the requirements of the overall system. The overall objective of this task was to make the best possible use of the province's existing data, hardware, and methods.

The review process revealed the following.

1. Some significant work had been done toward the development of performance prediction models. Although these models are not directly applicable to PINS, some valuable concepts were available.
2. The pavement management methodology that exists in Alberta Transportation is based on a comprehensive pavement sectional system, which provides structural, geometric, and performance data that are relatively detailed for use at the project level of pavement management (i.e., design).
3. A serious need exists for an objective, systematic, and computerized method for determining and programming pavement rehabilitation projects.
4. Alberta Transportation's computer facilities are suitable for the software packages that will be developed in the project.
5. The primary highway network of the province has been the subject of one of the most extensive (in terms of both time and information content) pavement inventories in North America. Historical field data are, therefore, available in a computerized data bank format to develop prediction models for various pavement parameters.
6. Alberta Transportation currently uses six Benkelman beams and one Dynaflect for measuring deflection, usually at a rate of 10 tests/mile,

Figure 3. Tasks and subtasks in stage 1.

TASK 1 Review Existing Models and Inventory Data Base

1.1 Review Existing Models, Software, Hardware

a) Review Existing Models for performance prediction,

b) Review Existing Pavement Management methods and software used in the department, and

c) Review the Department's Hardware.

1.2 Review the Inventory Procedures and Data Base Structure

a) Review the Inventory Procedures, and

b) Review the Data Base Structure.

1.3 Provide Comprehensive Detailed Planning for the Total Project

a) Determine the needs, and

b) Provide Detailed Planning.

1.4 Provide Report on Task 1

TASK 2 Develop Performance Models and Pavement Quality Index (PQI)

2.1 RCI prediction models for Granular Base Sections

a) Determine the form of models, and

b) Perform regression analyst to develop models.

2.2 Provide report on Task 2.1

2.3 RCI prediction models for soil-cement in full-depth

2.4 Deflection prediction models

2.5 VCR prediction models

2.6 PQI model

2.7 Evaluate models for Overlay Performance Prediction

2.8 Provide Report on Task 2

TASK 3 Develope an Initial PINS Program

3.1 Design and tailor PINS

3.2 Develop PINS

3.3 Evaluate by test application

3.4 Provide report on Task 3

TASK 4 Apply PINS to the Primary Highway Network

TASK 5 Prepare detail plan for Stage 2 and for Network Inventory Updating

5.1 Prepare detailed plan for Stage 2

5.2 Prepare detailed plan for inventory updating

TASK 6 Prepare Reports on Stage 1 and Conduct Training Courses

although this is increased to 26 tests/mile for sections that approach the terminal level. The department also uses two Portland Cement Association (PCA) car roadmeters for measuring roughness. Since 1976 the visual condition rating (VCR) procedures have been used by department personnel.

7. The department has an effective method of sectioning the highway system, which involves control sections defined by major intersections along a highway and these, in turn, are composed of homogeneous subsections. A total of 3,014 subsections are currently in the primary highway network.

TASK 2: DEVELOP PERFORMANCE MODELS AND PQI

Task 2 was directed toward the development of the pavement performance models required by the proposed PMS. Originally it was thought that approximately 27 performance models would be required. These models included three dependent variables [riding comfort index (RCI); deflection, and VCR] for each of three pavement types (granular base, soil-cement, and full-depth) in each of three climatic zones (southern, central, and northern). A PQI model was also needed as a means of combining RCI, deflection, and VCR into a single index for comparison of highway sections.

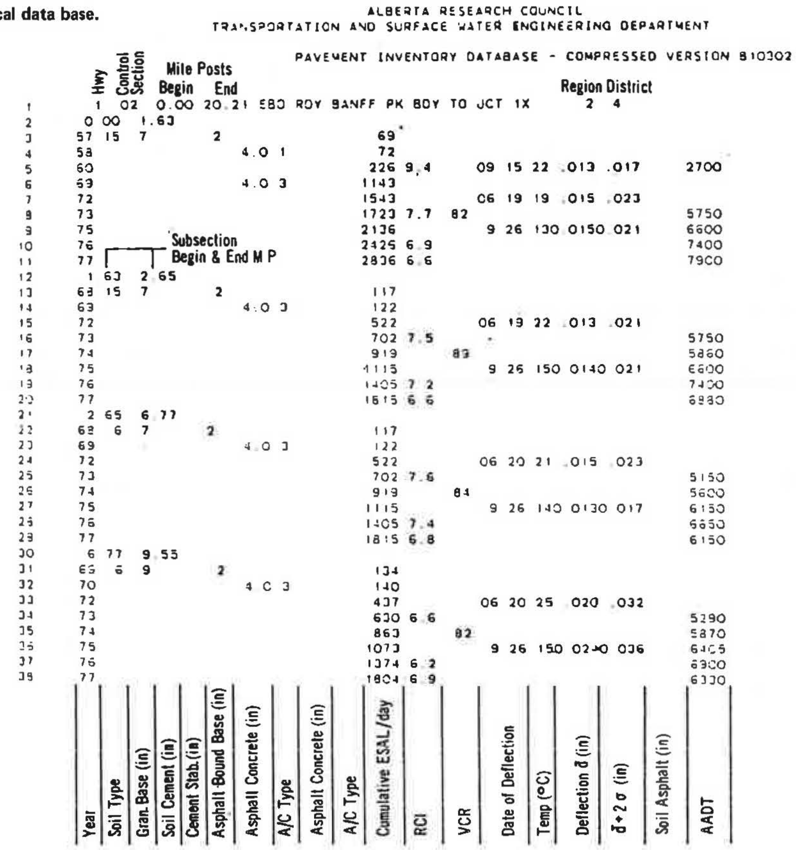
Performance Prediction Models

The performance models were expected to be recursive (i.e., the future RCI is a function of the present RCI), with terms that relate to age, traffic, soil type, and structural thickness used as independent or explanatory variables. The starting point for the model development was the department's performance data base, which had previously been computerized by the Alberta Research Council. This data base includes (for every section in the primary network) periodic measurements of RCI, Benkelman beam rebounds, VCR, and structural composition, traffic, soils, and rehabilitation data (see Figure 4). For some sections the data base dates as far back as the 1950s.

Separate models were thought to be necessary for different types of pavement structure; therefore, the first step involved extraction of the RCI, deflection, and VCRs and all relevant data [e.g., soil type, layer thicknesses, cumulative equivalent single axle load (ESAL)/day, and year] for each type of structure from the historical data base. During this process certain conversions were performed to make the data compatible with the types of models required. For example, years were converted to ages, with an age of zero corresponding to the year of surfacing, and cumulative ESAL/day for the age of zero were also set to zero. The resulting data files were then screened to eliminate extremely short sections (0.5 mile or less) and sections for which all data points are similar.

To analyze the possible effects of soil type, the soil types given in the data base were divided into three classes (good, fair, and poor). Two indicator variables were then defined--soil D1 and soil D2. Similarly, to test for possible effects of climate, two indicator variables (climate D1 and climate D2)

Figure 4. Sample of historical data base.



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                    ALBERTA RESEARCH COUNCIL
  TRANSPORTATION AND SURFACE WATER ENGINEERING DEPARTMENT

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 6  69                 4.0 3    1143
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13  63 15  7      2              117
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19  76                          1405 7.2                           7400
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25  73                           702 7.6                           5150
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35  74                           863     82                        5870
36  75                          1073         9 26 150 0240 036     6405
37  76                          1374 6.2                           6900
38  77                          1804 6.9                           6330
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were defined. The 15 districts in the province were separated into three approximate climatic zones (south, central, and north), and the indicator variables were assigned values that corresponded to the climatic zones and districts.

To compare the effects of structural layer thicknesses within structure types, the variable Equivalent Granular Thickness (EGT) was defined by using granular equivalency factors in the Roads and Transportation Association of Canada Pavement Management Guide (2).

RCI Prediction Models

Separate analyses were conducted for different pavement types by using explanatory variables related to age, traffic, layer thicknesses, soil type, and climate. As many as nine models (one for each combination of three climatic zones and three pavement types) were thought to be required. However, the results of the analyses indicated that only two models were required to predict RCI performance adequately for evaluation purposes under the conditions that exist in Alberta. The two models include one for granular base sections and one for the other structure types (full depth, soil cement, and cement stabilized). Both models are recursive in nature (i.e., RCI now is a function of the previous RCI). The analyses also showed that age was the major influence on RCI performance.

The major findings of the development analyses of the RCI model are summarized as follows.

1. Reliable RCI predictions cannot be obtained without the use of a recursive model in which the RCI at time t is a function of a previous RCI at time t - 1.

2. Regression analyses showed that the traffic, structural thickness, and soil type do not affect RCI performance significantly.

3. Climate has an effect on RCI performance only when a full-depth section is constructed in the northern climatic zone, in which case performance decreases significantly.

4. Although these parameters (traffic, soil, structural thickness, and climate) do not play a major role in affecting RCI performance, this does not mean that they have no effect on overall pavement performance.

5. The granular base sections perform significantly better (with respect to RCI) than the other structure types.

6. Two RCI prediction models were developed for use in the PINS system that require only Δ AGE and a starting value of RCI. The accuracy of the predictions are therefore dependent on the accuracy of the starting value. These two models include one for granular base sections and another for all other structure types. The value for Δ AGE that should be used in both models is 4 years. For predictions between the 4-year intervals linear interpolation should be used.

The two models discussed in detail by Cheetham and Christison (3) are described below:

$$\begin{aligned}RCI = &-5.998\,75 + 6.870\,09 \times LOG_e\,(RCI_B) - 0.162\,42 \times LOG_e\,(AGE^2 + 1)\\ &+ 0.184\,98 \times AGE - 0.084\,27 \times AGE \times LOG_e\,(RCI_B)\\ &- 0.092\,60 \times \Delta AGE \end{aligned} \tag{1}$$

with R^2 = 0.838 and a standard error of estimate = 0.38.

For soil-cement, full-depth, and cement-stabilized pavements:

$$RCI = -4.288 + 5.802 \times LOG_e (RCI_B) - 0.1744 \times \Delta AGE - 0.1846 \times FDN \quad (2)$$

with $R^2 = 0.845$ and a standard error of estimate = 0.29

where

RCI_B = previous RCI,
AGE = present age of pavement,
Δ AGE = 4 years, and
FDN = 1 for full-depth sections in the northern climatic zone
= 0 otherwise.

Deflection Prediction Models

Similar analysis conducted for predicting average pavement deflection as measured by Benkelman beam resulted in three models for the three major types of pavement in Alberta. Unlike the RCI models, it was found that separate models were needed for predicting average deflection for soil-cement, cement-stabilized, and full-depth pavements. However, as with RCI models, the effect of climatic zones was not significant and, therefore one model for each major pavement type was sufficient for predicting average deflection in the context of PINS.

The models for the different pavement structure types are as follows:

For granular base pavements,

$$LOG_e \bar{d} = 0.428\,47 + 0.916\,46 \times LOG_e \bar{d}_B + 0.041\,04 \times SD_2 \ldots \quad \text{with } R^2 = 0.87 \quad (3)$$

where $\bar{d}_B$ is the previous mean deflection and SD_2 is the soil district parameter.

For soil-cement and cement-stabilized pavements,

$$LOG_e \bar{d} = [0.688\,4 + 0.926\,38 \times LOG_e \bar{d}_B + 0.115\,44 \times (AGE + 1)] \div [(AGE_B + 1) + 0.025\,14 \times LOG_e \times \text{cumulative daily ESALS} \ldots] \quad \text{with } R^2 = 0.86 \quad (4)$$

where AGE is the present age of pavement and AGE_B is the previous age of pavement.

For full-depth pavements,

$$\bar{d} = 1.728\,41 + 1.209\,73 \times \bar{d}_B \ldots \quad \text{with } R^2 = 0.86 \quad (5)$$

After the development of deflection-prediction models, a structural adequacy index (SAI) concept was necessary to convert deflections into a more meaningful engineering measure that would indicate directly the ability of the pavement structure to withstand the traffic loadings. The SAI concept provides a means of converting the deflection (measured or predicted) to a scale of 0 to 10 (with 10 being perfect) and thus enables one to know the structural condition from a single number.

The SAI models were derived from the measured pavement deflection and an empirical relationship involving a maximum tolerable deflection (MTD) and the traffic volumes.

The SAI models developed are as follows:

For granular base pavements,

$$\text{Log } SAI = 1.222\,51 + 0.003\,2\,(SAL + 1.65)^{1.39} - 0.012\,538\,\bar{d} - 0.000\,157\,\bar{d}\,(SAL + 1.10)^{1.44} \quad (6)$$

When $\bar{d} \leq 18$, set SAI = 10.

For full-depth pavements,

$$\text{Log } SAI = 1.269\,62 + 0.000\,267\,(SAL + 7.6)^{2.086} - 0.011\,885\,\bar{d} - 9.88 * 10^{-6}\, d(SAL + 7.6)^{2.14} \quad (7)$$

When $\bar{d} \leq 13.5$, set SAI = 10.

where

$\bar{d}$ = mean fall rebound as measured by Benkelman beam x 10^3,
SAL = cumulative ESAL/10^5, and
MTD = deflection at 1980 cumulated ESALs that correspond to SAI = 3.0 for each of the three types of pavement.

VCR Prediction Models

The performance of a pavement in terms of its surface distress was modeled and predicted by using a visual condition index (VCI) concept, which is calculated from VCR and is based on a scale of 0 to 100 by dividing by 10. This was done to make the surface condition scores compatible with the RCI and SAIs, which are based on a scale of 0 to 10.

Extensive regression analysis conducted by using the VCR data in the data bank revealed no need to develop nine models (combinations of three pavement types and three climatic zones) as originally expected. One model for each pavement type was sufficient for predicting VCIs in the context of PINS evaluation.

The VCI prediction models developed are as follows:

For granular base pavements,

$$VCI = 1/10 \times (8.959\,66 + 0.875\,1 \times VCR_B - 3.046\,95 \times CD_2 - 2.921\,35 \times LOG_e (AGE + 1) \ldots \quad \text{with } R^2 = 0.74 \quad (8)$$

where

VCR_B = previous visual condition rating,
CD_2 = climatic district indicator, and
AGE = present age of the pavement.

For soil-cement and cement-stabilized pavements,

$$VCI = 1/10 \times (33.094 + 0.006\,67 \times VCR_B^2 - 1.252\,8 \times LOG_e (AGE^2 + 1) \ldots \quad \text{with } R^2 = 0.8 \quad (9)$$

For full-depth pavements,

$$VCI = 1/10 \times \exp\left\{-0.645\,84 + 1.122\,23 \times LOG_e VCR_B - 0.059\,73 \times CD_2\right\} \ldots \quad \text{with } R^2 = 0.73 \quad (10)$$

Development of PQI

The three performance prediction models just described aid in determining future needs for rehabilitation in terms of the individual parameters. Needs are thus determined for RCI that relate to the roughness of the pavement as it affects the highway user, for VCI in terms of the amount and severity of surface distress, and for SAI as the structural ability to withstand the expected traffic loadings.

The individual prediction of each of the preceding performance parameters discussed allows the determination of rehabilitation needs based on each parameter; however, the ability to determine needs based on the overall quality is also necessary. The use of PQI allows RCI, VCI, and SAI to be combined into a single number that represents the overall quality of the pavement. PQI encompasses all of these aspects of the pavement performance and provides a single index for comparing the performance of pavement sections and their relative rehabilitation needs.

The PQI model was developed considering the overall performances of different pavement sections for which RCI, SAI, and VCI were known. The overall performance was defined by a subjective panel rating procedure. Forty pavement sections were selected for the PQI rating sessions. The sections were selected to cover a wide range of the three basic performance parameters (RCI, VCI, and SAI) for each of the three major pavement types (granular base, full depth, and soil cement).

Two station wagons of similar ride and size (a 1978 Plymouth and a 1977 Ford) were used in October 1981 to carry two panels of four raters each on a visual inspection and ride on 5 of the 40 sections. During the following 2 days the remainder of the sections were rated but only by six raters. On the last day replicate ratings were made on five sections by five raters.

The panel members were trained before the rating sessions. The purpose of the project was explained and the pavement quality concept discussed. A sample rater's guide used in the training can be found elsewhere (4). Pavement quality rating forms were provided for each section, a sample of which is shown in Figure 5. These forms also contained information on traffic and deflection magnitudes as well as RCI, VCI, and SAI.

PQI rating data were first analyzed to check for systematic errors. Leniency error, halo effects, and central tendency effects, which are the most common types of systematic errors in rating procedures, were found to be insignificant. Therefore, no adjustment of the data was necessary, and the raw data were used in subsequent analyses.

Analysis of variance (ANOVA) techniques were then used to test for sources of variation in the data. A panel comparison was made that tested between panels and among each panel as sources of variation. However, neither of these factors was found to be significant as a source of variation. Next a location comparison ANOVA was conducted on the data. This ANOVA tested the effects of drivers versus others and among others for location comparisons. Again, these effects were found to be insignificant as sources of variation.

In both the panel comparison and location comparison, the only truly significant source of variation was due to sections. The replicated sections were then analyzed to determine the short-term repeatability of the PQI ratings. The replication ANOVA indicated that the raters could repeat their ratings reasonably well. However, this should not be taken as a generalization because the replications were done within a short time period (i.e., 2 days). The details of the preceding data analyses are given by Cheetham and Karan (4).

No systematic errors were found; therefore, the raw data were used in regression analyses to develop a PQI model. Several transformations of the data were evaluated; however, the final model that resulted from the analyses is

$$PQI = 1.1607 + 0.0596 \cdot RCI \cdot VCI + 0.5264 \cdot RCI \cdot \log_{10} SAI \qquad (11)$$

This model has a standard error of estimate of 0.79 ($R^2 = 0.76$).

The regression analyses that result in this model are discussed elsewhere (4).

Figure 5. Sample PQI rating form.

SECTION NUMBER:

LOCATION: 21:22 MP 5.36 (8.63 km) to MP 6.36 (10.23 km)

MILEAGE TIE: MP 0.0 = Jct. 53

PAVEMENT TYPE: GB

LAYER THICKNESS: 4 AC, 2 ABB, 6 BASE

Age: 18 DTF: 4.8 ESAL ($x10^5$): 2.4

d: 0.039 MTD: 0.056 RCI: 5.5

VCI: 6.5 SAI: 5.2

PAVEMENT QUALITY

Excellent (Pavement Like New)

Good (Many years of service life)

Fair (Close to or needing rehabilitation)

Poor (Should have been rehabilitated in the last couple of years)

Extremely Poor (Should have been rehabilitated many years ago)

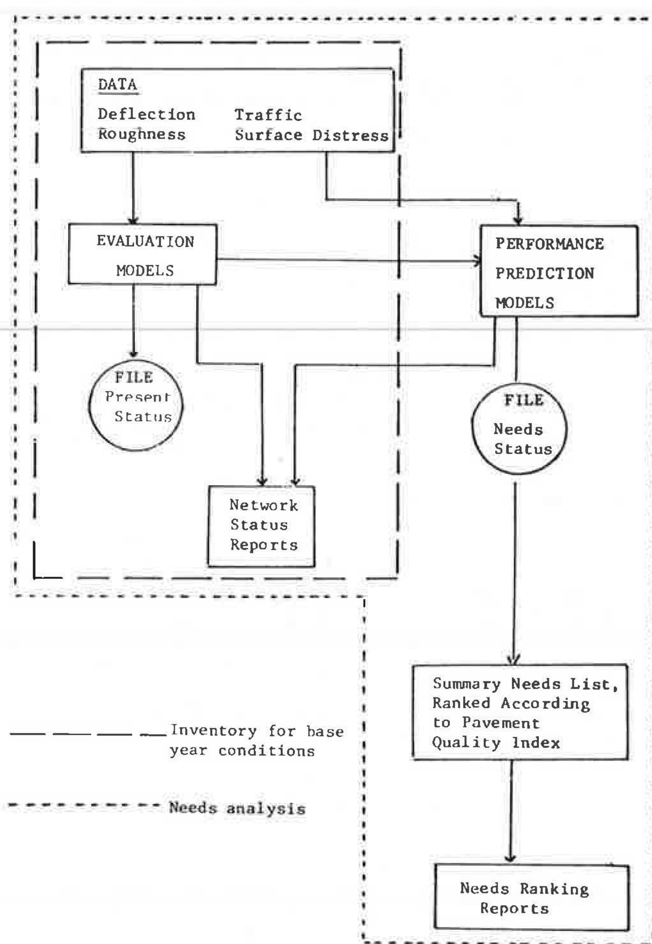
Is Pavement of Acceptable Quality? ____Yes

____No

____Undecided

COMMENTS: ____________

Figure 6. General structure of PINS.



An analysis of the acceptance of the pavement quality was also conducted based on the acceptable-unacceptable responses of the rater for each section. This analysis showed that, based on the raters involved, a minimum acceptable level of PQI is about 4.7. This is not to be taken as an absolute level but is an indication of the rater's responses. The details of the minimum acceptable PQI analysis are also given elsewhere (4).

TASK 3: DEVELOP INITIAL PINS

The main function of the PINS program is to process pavement management data (i.e., deflection, RCI, VCR, and traffic) from the pavement data base and to generate for immediate and future use of department personnel the following:

1. Current status of the network in terms of PQI and its components of structural adequacy, SAI, ride quality, RCI, and visual condition, VCI;
2. Remaining service life (in structural and serviceability terms) of each section in the network, based on the performance prediction models 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 current status of the highway network and improvement needs for each region in Alberta.

The PINS programs developed for Alberta has the capability of determining the current status of a section in terms of its RCI, SAI, VCI, and PQI parameters (see Figure 6). These analyses can be conducted for every section in the network, in a region, or on a highway. Once the analyses are completed for every section the program produces detailed output for every section as well as a status report for the network, region, or highway.

The next step in the analyses is to predict performance for each performance parameter (i.e., RCI, SAI, VCI, and PQI). As with analysis of the current status, performance prediction and needs analyses can be conducted for every section in the network, in a region, or on a highway. The program produces graphical outputs (i.e., performance curves) for every section as well as the year in which a parameter will reach its minimum acceptable level. A sample output of this type is shown in Figure 7.

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 RQI) 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 is produced by PINS. Figures 8 and 9 show sample ranking lists based on RCI and PQI, respectively. Similarly, a three-dimensional histogram similar to the one shown in Figure 10 is also produced so that comparisons can be made of regions, districts, or highways in Alberta.

Figure 7. Sample sectional PINS output.

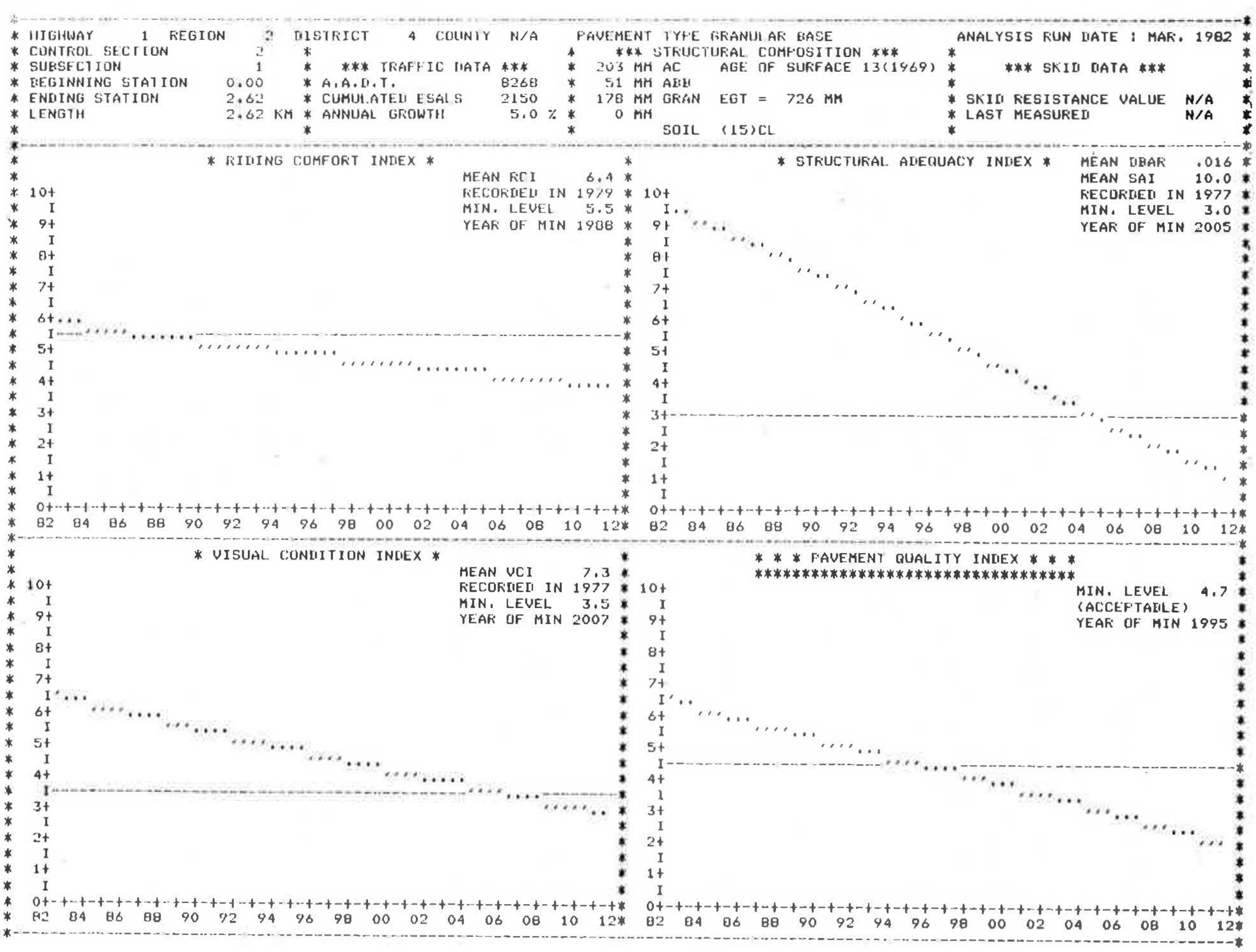


Figure 8. Sample RCI ranking list produced by PINS.

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* PAVEMENT INFORMATION AND NEEDS SYSTEM *

* SECTIONAL SUMMARY TABLE *

* RCI RANKING FOR REGION 2 *

* *

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	3	10-08	ECL DRUMHELLER TO EAST COUL	1	0.00	5.79	0.0	1.9	2.7	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	SCL 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	3	21-14	JCT 9 TO JCT 27	21	39.47	41.05	2.9	1.4	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	1.9	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	2	7.96	9.17	3.2	4.3	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	25.26	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	1A-06	JCT 940 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	1.03	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	1.4	3.5	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	3.6	4.4	2.2	2.8	N/A

Figure 9. Sample PQI ranking list produced by PINS.

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* PAVEMENT INFORMATION AND NEEDS SYSTEM *

* SECTIONAL SUMMARY TABLE *

* PQI RANKING FOR REGION 2 *

* *

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	3.1	4.3	1.2	N/A
2	3	10-08	ECL DRUMHELLER TO EAST COUL	1	0.00	5.79	0.1	2.3	3.0	1.2	N/A
3	3	10-08	ECL DRUMHELLER TO EAST COUL	3	6.92	10.23	0.4	2.8	3.2	1.3	N/A
4	4	1A-06	JCT 940 TO 16 AVE NW CALGAR	1	0.00	3.35	4.6	1.2	0.1	1.5	N/A
5	4	2A-12	JCT 2 TO DIDSBURY	3	9.48	9.75	4.3	0.9	1.1	1.5	N/A
6	3	21-14	JCT 9 TO JCT 27	21	39.47	41.05	4.1	2.2	0.1	1.7	N/A
7	3	21-14	JCT 9 TO JCT 27	22	41.05	42.64	4.0	2.5	0.5	1.7	N/A
8	3	21-14	JCT 9 TO JCT 27	20	38.54	39.47	4.2	2.7	0.3	1.8	N/A
9	3	21-16	JCT 27 TO JCT 42	1	0.00	2.48	4.2	2.6	0.4	1.8	N/A
10	3	21-16	JCT 27 TO JCT 42	2	2.48	3.07	4.1	2.6	0.7	1.8	N/A
11	4	1-14	JCT 21 TO JCT 956	1	0.00	11.44	2.8	2.7	1.9	2.0	N/A
12	4	1A-06	JCT 940 TO 16 AVE NW CALGAR	3	12.16	13.63	4.4	3.3	1.0	2.0	N/A
13	4	2A-04	JCT 2 TO JCT 23	1	0.00	18.78	3.2	1.1	2.4	2.0	N/A
14	3	21-14	JCT 9 TO JCT 27	23	42.64	44.05	4.0	3.4	0.3	2.0	N/A
15	3	21-14	JCT 9 TO JCT 27	19	36.36	38.54	4.7	2.5	1.2	2.1	N/A
16	3	10-08	ECL DRUMHELLER TO EAST COUL	6	14.50	21.27	2.3	2.9	3.5	2.2	N/A
17	4	2A-10	S.CROSSFIELD TO N.CROSSFIEL	1	0.00	2.04	3.6	1.8	2.5	2.3	N/A
18	4	1A-04	JCT 1X TO JCT 940	2	27.34	29.64	4.2	2.4	2.0	2.4	N/A
19	4	1A-06	JCT 940 TO 16 AVE NW CALGAR	2	3.35	12.16	4.6	1.6	2.2	2.4	N/A
20	3	21-16	JCT 27 TO JCT 42	4	4.09	14.53	5.0	2.5	1.5	2.4	N/A
21	4	22-16	JCT 1 TO CREMONA	7	9.28	10.93	4.2	5.1	0.7	2.4	N/A
22	4	2A-04	JCT 2 TO JCT 23	3	20.74	21.54	5.0	4.7	0.4	2.5	N/A
23	3	21-16	JCT 27 TO JCT 42	3	3.07	4.09	4.9	2.9	1.6	2.5	N/A
24	4	9-04	JCT 21 TO SCL DRUMHELLER	1	0.00	0.39	3.2	1.7	4.6	2.6	N/A
25	4	22-16	JCT 1 TO CREMONA	4	7.88	8.09	4.6	5.3	0.4	2.5	N/A
26	4	22-16	JCT 1 TO CREMONA	5	8.09	8.50	4.5	5.1	1.1	2.6	N/A
27	4	1-08 EBD	JCT 22 TO WCL CALGARY	1	0.00	3.81	4.8	4.4	1.3	2.7	N/A
28	4	1-08 EBD	JCT 22 TO WCL CALGARY	2	3.81	7.63	4.7	4.9	1.2	2.7	N/A
29	4	1-08 EBD	JCT 22 TO WCL CALGARY	3	7.63	13.07	4.8	5.2	1.1	2.7	N/A
30	4	1-10 EBD	BLACKFT TR(CAL) TO JCT 9	6	19.28	26.61	4.4	4.9	1.3	2.7	N/A
31	4	1A-06	JCT 940 TO 16 AVE NW CALGAR	8	24.63	29.53	4.7	5.2	0.9	2.7	N/A
32	3	21-14	JCT 9 TO JCT 27	5	9.98	13.95	4.3	4.6	1.5	2.7	N/A
33	4	22-16	JCT 1 TO CREMONA	2	0.63	3.02	4.9	5.3	0.8	2.7	N/A
34	4	22-16	JCT 1 TO CREMONA	3	3.02	7.88	5.0	5.3	0.7	2.7	N/A
35	4	27-06	ECL SUNDRE TO JCT 2	3	25.20	26.26	4.3	5.8	0.4	2.7	N/A
36	3	56-10	JCT 9 TO S.JCT 589	3	12.97	13.35	4.2	4.3	1.7	2.7	N/A
37	4	1-06 EBD	E.MORLEY TO JCT 22	3	16.56	20.42	4.4	5.4	1.2	2.8	N/A

Figure 10. Sample three-dimensional histogram produced by PINS.

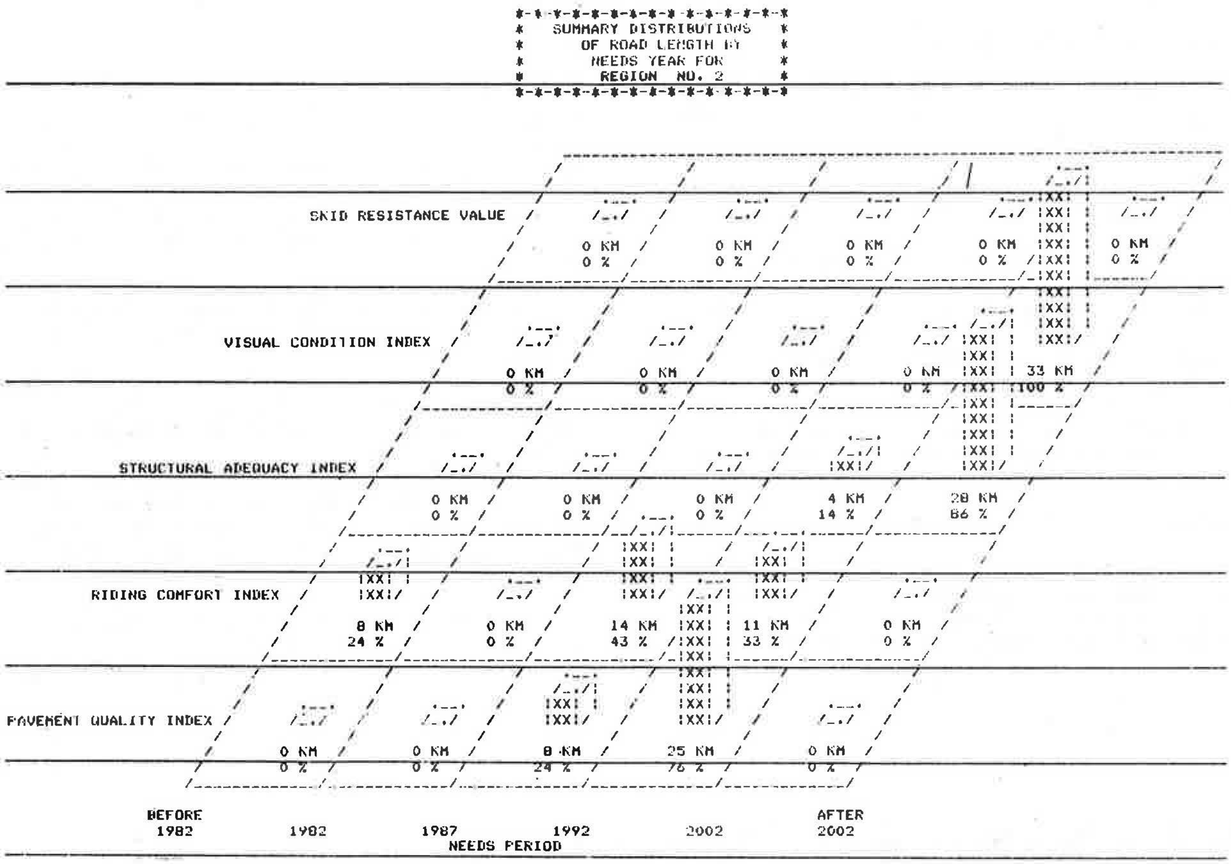

Figure 11. Sample needs list produced by PINS.

* PAVEMENT INFORMATION AND NEEDS SYSTEM *

NEEDS IDENTIFICATION LIST
BY VCI
FOR HIGHWAY 1
FOR ALL SECTIONS OF THIS HIGHWAY

DISTRICT	HWY - CONTROL SECTION	CONTROL SECTION DESCRIPTION	INV. SECTION	BEGIN KM.	END KM.	PAVEMENT QUALITY INDEX	RIDING COMFORT INDEX	STRUCTURAL ADEQUACY INDEX	NEEDS ** * VISUAL CONDITION INDEX	SKID RESISTANCE VALUE
4	1- 2	EBD BANFF PK BDY TO JCT 1X	1	0.00	2.62	2.9(1995)	4.2(1988)	2.3(2005)	3.5(2007)	N/A
4	1- 2	EBD BANFF PK BDY TO JCT 1X	2	2.62	4.26	2.5(1997)	4.1(1989)	1.7(2006)	3.5(2010)	N/A
4	1- 2	EBD BANFF PK BDY TO JCT 1X	3	4.26	10.89	2.7(1999)	4.2(1992)	2.1(2008)	3.5(2010)	N/A
4	1- 2	EBD BANFF PK BDY TO JCT 1X	4	10.89	15.37	2.1(1988)	3.5(1982)	1.3(1999)	3.5(2007)	N/A
4	1- 2	EBD BANFF PK BDY TO JCT 1X	5	15.37	18.60	2.7(1992)	3.6(1982)	2.6(2006)	3.4(2007)	N/A
4	1- 2	EBD BANFF PK BDY TO JCT 1X	6	18.60	21.79	2.5(1997)	3.9(1989)	1.8(2007)	3.4(2011)	N/A
4	1- 2	EBD BANFF PK BDY TO JCT 1X	7	21.79	32.52	2.8(2000)	4.0(1993)	2.4(2010)	3.5(2011)	N/A

Needs tables are also produced for each performance parameter and for each year in the programming period. Figure 11 shows a sample needs table.

In summary, the PINS program developed for Alberta analyzes the data base first to determine the present status and second to predict performance and 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 and graphical format for every section. Network summary information is also produced in tabular and graphical formats.

TASK 4: APPLY PINS TO PRIMARY HIGHWAYS IN ALBERTA

PINS has been applied to all of the primary highways in each of the six regions in Alberta. These results, which are described in detail by Kerr and Karan (5), included the predicted performance over a 30-year period (1982 to 2012) for each inventory section; the existing status of the primary highways in each region in terms of PQI, RCI, SAI, and VCI; and the needs lists for selected periods of time, again for each of the four evaluation indices.

Alberta Transportation's headquarters and regional personnel have gone through the results in detail and assessed their reasonableness. Extensive field trips and discussions indicated that the results are reasonable and that they provided useful information for pavement management purposes. A few comments were made about program structure and formats to make the overall program performance more efficient and results more directly useful to the department's engineers. In overall terms, however, the PINS was generally accepted as a valuable tool within the department.

TASKS 5 AND 6: CONTINUING WORK PLAN

The following tasks were conducted to complete stage 1 by September 1982 and provide a good base for stage 2:

1. Refine PINS program based on the feedback received from Alberta Transportation in terms of input and output formats,
2. Prepare model and system documentation,
3. Prepare user manual,
4. Install PINS program on Alberta Transportation's computer facilities,
5. Conduct training courses for the users of PINS at Alberta Transportation,
6. Prepare a detailed work plan for stage 2, and
7. Carry out the actual work in stage 2 according to the plan prepared in step 6 above and the preplanning report.

ACKNOWLEDGMENT

We wish to acknowledge Brian Kerr and Alan Sadowsky of Pavement Management Systems, Ltd.; and Robert White, George Nicol, and Marshall Korchinski of Alberta Transportation; and Brian Shields of Alberta Research Council for their invaluable assistance in this project.

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Predicting Reductions in Service Life of Surface-Treated Pavements under Oil Field Traffic

THOMAS SCULLION, JOHN M. MASON, JR., AND ROBERT L. LYTTON

One adverse effect of the recent oil field boom in Texas has been the accelerated physical deterioration of many of the thin pavements that service the oil fields. To study this problem the Texas State Department of Highways and Public Transportation sponsored a research project the ultimate aim of which is to quantify the additional costs to the highway department associated with the drilling of a single well and the total costs for any impacted area. One key phase of the study has been the development of pavement distress and performance equations for thin pavements that relate pavement damage to traffic loading. These equations have been developed by regression analysis using pavement condition data collected during a seven-year period on thin pavements in Texas. Initial results demonstrate that these regression equations are better predictors of long-term pavement performance than the AASHTO equation. A case study is presented to outline how these predictions were used to calculate reductions in pavement life and increases in life-cycle costs associated with the oil field development. This study predicted that the oil field development reduced the remaining life of a typical thin pavement from 46 to 16 months and increased the rehabilitation costs tenfold from $0.50 to more than $5.00/yd^2.

During the late 1970s and early 1980s, several Texas counties experienced a rapid expansion of oil field exploration and development work. A majority of the pavements in these rural areas are surface-treated pavements, which typically have a 6-in. flexible base. These pavements were not designed to carry the high intensity of loads associated with oil field traffic, and subsequently many severe pavement failures occurred. The Texas State Department of Highways and Public Transportation (TSDHPT) found that the oil-related activity caused considerable additional demand for their maintenance and rehabilitation funds. This aroused an interest in providing a means of accurately predicting the additional life-cycle costs incurred. Questions such as the following became the subject of a research project (1) with the Texas Transportation Institute: What traffic loads are associated with the development of an oil well? How much damage do they do? What additional costs are associated with the drilling of a single well? What are the total costs associated with an impacted area? The long-term objectives of this project are as follows.

1. Identify the type and duration of loads associated with the development of a single oil well. Convert these loads into 80-kN (18-kip) equivalent single-axle-loads (ESALs).
2. Develop a procedure to predict the reduction in pavement life and increases in rehabilitation costs associated with these oil-related loads.
3. Perform a life-cycle cost analysis to identify total additional costs associated with the development of a single well and total costs for an oil-impacted area.

The first objective has been met and is reported elsewhere (1). This paper concentrates on describing the development of the predictive procedure used for calculating the reductions in pavement life associated with oil field traffic and presents the initial results of a life-cycle cost analysis.

Typical surface-treated pavements consist of a single or double surface treatment over a 6- to 8-in. flexible base course, and carry an average daily traffic (ADT) of less than 750 vehicles per day. Little has been published on the long-term performance of these thin pavements. Discussions with the states' maintenance personnel and analysis of available data made evident that many of these pavements, under normal conditions, only require regular seal coats at five- to nine-year intervals to prolong pavement life and treatments applied at the onset of moderate levels of pavement distress (i.e., surface cracking or raveling). Localized and full reconstruction are applied when the pavements show significant levels of load-associated distress; i.e., rutting, alligator cracking, and reduced riding quality [present serviceability index (PSI)]. This is frequently found in pavements that have carried traffic loads that are heavier than anticipated.

The approach taken in this study to predict the reduction in pavement life caused by oil field traffic is as follows:

1. Develop pavement performance equations for PSI and distress from inspection data collected over a seven-year period on in-service surface-treated pavements in Texas;
2. Use these equations to predict distress levels induced in typical pavements under both intended use and intended use plus oil field traffic;
3. Define pavement damage in terms of a pavement score that is a composite index that combines distress and loss of serviceability; and
4. Define pavement failure (in terms of pavement score) at a level compatible with the TSDHPT's current rating system for these thin pavements.

The problems associated with oil field exploration and development are not unique but are similar in many respects to the impact of other load-intensive commercially important hauls such as coal, timber, grain, cotton, and beef.

DEFINITION OF PAVEMENT DAMAGE AND DAMAGE FUNCTIONS

Damage was defined at the AASHTO Road Test to be a normalized score between 0 and 1; when the pavement reached a terminal condition the damage was 1. A damage function is an equation that describes how the damage proceeds from its initial value to its terminal value and beyond. In the AASHO Road Test (2, pp. 307-322) the damage function was assumed to be of the form

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

where

g = the damage,
N = the number of 18-kip (80-kN) ESALs,
ρ = a constant that equals the number of 18-kip (80-kN) ESALs when g=1, and
β = a power that dictates the curvature of the damage function.

In the AASHTO Road Test, damage was defined as

$$g = (P_i - P)/(P_i - P_t) \quad (2)$$

where

P_i = initial serviceability index,
P_t = terminal serviceability index, and
P = present serviceability index.

Values of ρ and β were found for each pavement section by regression of the logarithm of damage against the logarithm of 18-kip (80-kN) ESALs. Further regression analysis determined how ρ and β depended on design and load variables.

This analysis led to the development of the AASHTO flexible pavement design system, which was first published as an interim design guide in 1961 and issued as a revised edition in 1972 (3). The design equation used in this system relates the number of 80-kN ESAL repetitions required to reach a predefined terminal serviceability level (P_t) for any given pavement structure, climatic condition, and subgrade soil. The AASHTO design equation is recommended for flexible pavements that have a minimum asphalt surfacing thickness of 2 in. (3, p. 21); therefore, the AASHTO equation does not give reasonable predictions of pavement life for the thin surface-treated pavements under investigation in this study. With a structural number of approximately 1 to 1.5, the AASHTO equation predicts a life for Texas pavements of less than 5,000 18-kip (80-kN) ESALs. This is considerably less than has been observed on in-service thin pavements in Texas.

For these reasons new performance equations were thought necessary for thin flexible pavements in Texas. These equations can then be used to predict reductions in pavement life caused by oil field traffic.

TEXAS FLEXIBLE PAVEMENT PERFORMANCE EQUATIONS

Flexible Pavement Data Base

As the AASHTO Road Test drew to a close one of the strongest recommendations made by the test staff was that satellite studies should be made in other parts of the country to determine with some objectivity the real effects of subgrade and climate.

Texas participated in these studies with the establishment of a flexible pavement data base (4) that contains detailed data on more than 400 sections of pavement. The sections were chosen by a stratified random selection process that gave a reasonably uniform distribution of pavement type, age, materials, layer thickness, soil types, and climate. Of these 400 sections, 132 are on thin surface-treated pavements on farm-to-market-type routes. These thin pavement sections were chosen for analysis in this study. They typically carry between 100 and 750 vehicles per day and were constructed with granular base courses that range in thickness from 4 to 10 in. All of these sections originally had a single- or double-seal surfacing, and many have received additional reseals.

Data collection on these sections started in 1972 when each section's full construction, maintenance, and traffic history was compiled. PSI, distress, and skid surveys have been made periodically on all sections since 1973. In most cases five or six separate observations have been made since the survey began. A complete listing of data collected on one of the thin pavement sections is shown in Figure 1. This section was reconstructed in 1969 with a 6-in. flexible base and surface treatment. Distress and riding quality (PSI) surveys were completed in the years 1973-1980. In 1975 the average daily traffic was 685 vehicles (both directions) and in the period between 1969 and 1979 the section has carried almost 23,000 80-kN ESALs.

Pavement Performance Equations

When a distress survey is conducted the following eight types of distress are observed: alligator cracking, transverse cracking, longitudinal crack-

Figure 1. Data from section 193 of Texas flexible pavement data base.

```
LOCATION
   SECTION ID NO: 1938
   DISTRICT NO:       19
   COUNTY NO/NAME:  32/CAMP
   CONTROL-SECTION: 1019- 1
   HIGHWAY:          FM 556
   MILE POINTS:       3.990 -   5.990
   LANE:              R

   FROM POST 4 TO POST 6
```

```
ENVIRONMENTAL - 20 YEAR SUMMARY (1955-1974)

                       JAN FEB MAR APR MAY JUN JUL AUG SEP OCT NOV DEC  AVG
 THORNTHWAITE INDEX:    -   -   -   -   -   -   -   -   -   -   -   -  47.4
 MEAN TEMPERATURE:     43  47  54  64  71  77  82  81  75  65  54  46  63.5
 PRECIPITATION:       2.8 3.4 3.5 6.6 4.5 4.0 2.5 2.6 4.7 3.7 4.1 4.0 46.2
 WET F-T CYCLES:        1   1   0   0   0   0   0   0   0   0   0   0    4
 TOTAL F-T CYCLES:     12   8   3   0   0   0   0   0   0   0   3   9   37
 DIST TEMP CONSTANT:    -   -   -   -   -   -   -   -   -   -   -   -  24.6
 SOLAR RADIATION:
```

```
PAVEMENT CONDITION SURVEY
  PVMT RATING  80    77    76    75    74    73
     PRS       82    67    92    73    88    90
     RUTT            1 SL  1 SL
     FLUSH           1 MO  2 SL        1 SL  1 SL
     CORR                        1 SL
     RAVEL           1 SL
     ALLG CR   1 SL  1 SL
     LONG CR   1 SL  1 SL
     TRANS CR  1 SL  1 SL
     CRACKS    NS    NS
     PATCHING  1 F   1 F   1 G   2 F   2 F   1 F
     FAIL/MI    0     0     0     1     0     0
 *** *** *** *** *** *** *** *** *** *** ***
RATING SCORES
  SHOULDER     70    75    50    75    70    70
  ROADSIDE     85    77    77    70    75    75
  DRAINAGE     57    70    83    70    70    77
  TRAFFIC SER  62    76    76    80    76    86
 *** *** *** *** *** *** *** *** *** *** ***
  PVMT URGENCY  3     3     1     3     3     4
```

```
TRAFFIC
  1980 ADT:                       645
  1980 PERCENT TRUCKS:            8.0
   8/69-12/82 VEHICLES:       1453156
   8/69-12/82 18K AXLES:       112941
```

```
SKID NUMBER
  DATE     AVG   LOW   HIGH
 12/76      23    18    40
  4/74      27    22    29
  4/74      40    34    45
  5/72      27     0     0
  5/71      32     0     0
```

```
SERVICEABILITY INDEX
  YR  MEAN  STD DEV   N    CV    LOW  HIGH
  80  1.89    .273   10  14.4  1.60  2.40
  77  2.40    .411   10  17.1  1.80  3.10
  76  1.28    .572   10  44.7   .50  2.20
  75  1.85    .486   10  26.3  1.30  2.80
  74  2.51    .557   10  22.2  1.70  3.70
  73  2.46    .403   10  16.4  2.10  3.50
```

```
DYNAFLECT DEFLECTION BASIN

                      W1               SCI
   DATE      MEAN   STD DEV   MEAN   STD DEV
  8/ 2/76   1.392    .562     .544     .218

                   PAVEMENT SUBGRADE
 STIFF. COEFF.:      .891      .254
```

```
STRUCTURAL SECTION          WORK                                          AGG.              APPL THICKNESS
  LAYER DESCRIPTION JOB    TYPE    DATE     AGG / SOIL TYPE    ITEM CL   TYPE GR RATE   ADMIX PCNT RATE CENT EDGE TTC  LL   PI
    1     S ST       12 WID-REC  8/69 SILICEOUS              320             3                          .6   .6
    2     B FLEX     12 WID-REC  8/69                                                                   6.0  .0
    3    SG SAND     12 WID-REC  8/69 SC,CL                                                                       4.5 30.2 17.3
```

Table 1. Area and severity ratings for flexible pavements.

Area			Severity		
Percentage of Area	Rating	Score	Description	Rating	Score
0 - 1	0	0.005	None	0	0.005
1 - 15	1	0.080	Slight	1	0.167
16 - 30	2	0.230	Moderate	2	0.333
>30	3	0.500	Severe	3	0.500

ing, rutting, raveling, flushing (or bleeding) failures (potholes), and patching. Each of these is rated for its area and severity of distress according to the distress identification manual prepared for Texas (5).

The area covered by the distress is estimated on all distress types except failures. For longitudinal and transverse cracking the linear length of cracks and number of cracks per station are used to obtain an area rating. The possible severity ratings are described by Epps and others (5); for instance, rutting severity depends on rut depth and cracking severity depends on crack width. In order to develop distress equations, the area and severity ratings are converted to a decimal score (0 to 1.0) as given in Table 1.

For this study a different form of damage function was assumed that produces a sigmoidal (S-shaped) curve; as shown in Figure 2 this shape appears to reproduce long-term pavement distress and performance better than does the assumed form of the AASHTO Road Test damage function (6-8). The assumed form of the damage function for Texas flexible pavements is

$$g = \exp - (\rho/N)^{\beta} \tag{3}$$

where

g = normalized damage,
N = as defined in Table 2, and
ρ,β = constants for each pavement section.

A full description of the analysis undertaken to produce the pavement performance equations used in this study is not presented here; however, the procedure and typical equations have been published elsewhere (9). An overview of the procedure is as follows.

The ρ and β values for each section are calculated from the observed distress and serviceability index histories. A plot of the growth in area of rutting, alligator cracking, and longitudinal cracking from section 320 of the flexible pavement data base is shown in Figure 3. The best curve of the form shown in Equation 3 is fitted through the pavement condition data and the values of ρ and β are calculated.

Regression analysis, using SAS (10) stepwise regression, was then performed to explain the variations of ρ and β between sections of the same pavement type. The determined final regression equations are of the form:

$$\rho = \text{f(climate, base thickness, subgrade properties, and so on)} \quad (4)$$

A sample equation is given below for rutting area.

$$\rho = [-0.103\ 5 + 0.005\ 49(\text{AVT}) + 0.006\ 7(\text{D}) - 0.001\ 5(\text{LL}) + 0.001\ 62(\text{PI}) + 0.000\ 77(\text{FTC})] \times 10^6 \quad \text{with } R^2 = 0.38 \quad (5)$$

and

$$\beta = 1.54 + 0.0169(\text{TI}) - 0.072(\text{D}) \quad \text{with } R^2 = 0.47 \quad (6)$$

Figure 2. Comparisons of present serviceability predictions made with Texas regression equation and AASHTO equation against actual performance data.

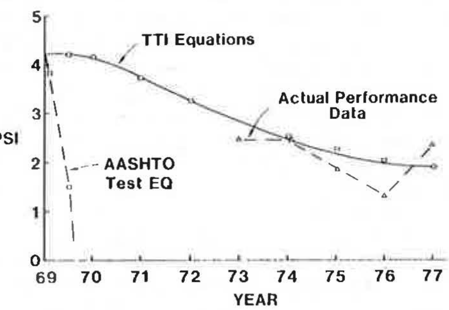


Figure 3. Plots of growth in area of various distress types of typical thin pavement section.

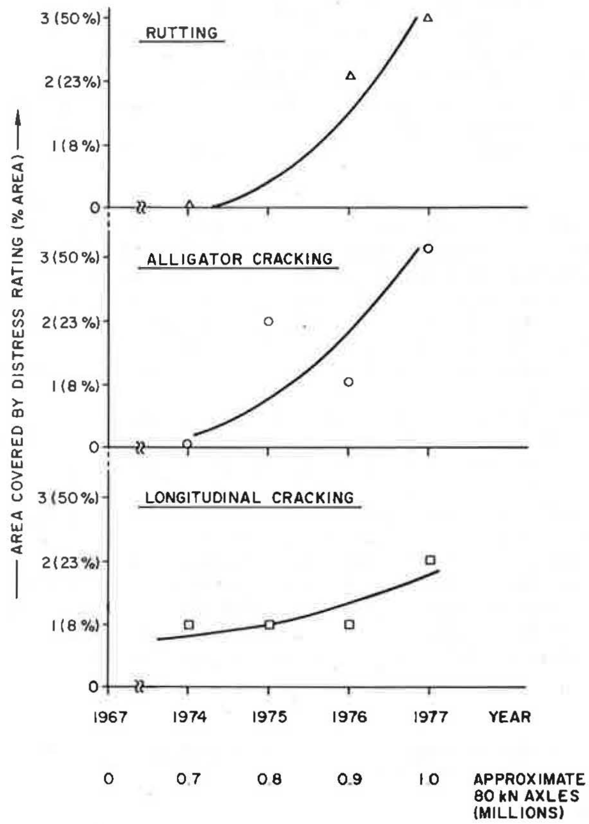


Table 2. Regression constants obtained for ρ and β equations by type of distress.

Distress Type	Equation Parameter	A	B	C	D	E	F	G	H	Mean	R^2
	ρ	−0.173	0.006 87	−0.006 32	0.013 3	0.000 75	0	0.001 53	−0.0214		0.36
	P_f					← Use mean →				0.83	
Rutting	Area ρ	−0.103	0.005 49	0	0.006 70	−0.001 5	0.001 62	0.000 77	0		0.38
	Area β	1.54	0	0.016 9	−0.072 0	0	0	0	0		0.47
	Severity ρ	−0.0678	0.003 20	0	0.005 66	−0.000 31	0	0.000 48	0		0.33
	Severity β					← Use mean →				1.78	
Raveling	Area ρ	1.03	0	0.014 6	0	0	0	0.006 4	−0.609		0.37
	Area β					Use mean				1.28	
	Severity ρ	0.62	0	0.012 9	0	0	0	0.006 6	−0.449		0.32
	Severity β					← Use mean →				1.40	
Flushing	Area ρ	0.488	0	0.013	0	0	0	0.003 45	−0.213		0.28
	Area β					← Use mean →				1.27	
	Severity ρ	−0.14	0.031	0.010 3	0	0	0	0.005 4	−0.201		0.32
	Severity β					← Use mean →				1.50	
Alligator cracking	Area ρ	−0.179	0.012 1	0	0.004	−0.001 1	0	0.001 53	0		0.52
	Area β	1.867	0	−0.009	0.144	0	0	0	−0.572		0.41
	Severity ρ	−0.22	0.012	0.000 33	0.002 7	−0.000 58	0	0.001 7	0		0.35
	Severity β	2.91	0.099	0	0.013	0	0	0	−1.567		0.34
Longitudinal cracking	Area ρ	−63.1	4.52	0.541	7.41	0	0	1.11	0		0.56
	Area β					← Use mean →					
	Severity ρ	−120	6.77	1.14	4.78	0	0	1.32	0		0.47
	Severity β					← Use mean →					
Transverse cracking	Area ρ	−66.4	0	2.156	10.1	0	0	0.718	0		0.33
	Area β	2.06	0	0	0.073 4	−0.06	0.061	−0.003 7	0		0.45
	Severity ρ	96.3	−1.04	1.07	0	0	0	−0.318	0		0.45
	Severity β	1.10	0	0	0	0.16	−0.24	−0.015	0		0.40
Patching	Area ρ	0.008	0.002 5	0.000 22	0.001 7	0	−0.001 2	0	0		0.36
	Area β					← Use mean →				1.75	
	Severity ρ	−0.04	0.003 5	0	0.003	−0.000 4	0	0.000 39	0		0.23
	Severity β	−0.16	0.050	0	0.090	−0.069	0.082	0.027	0		0.49

Notes: D = exp $(-\rho/N)^{**}\beta$ assumed form of distress curve where D is normalized damage function and
N = 10^{-6} x 80 kN axle rep because reconstruction for PSI
= 10^{-6} x 80 kN axle load rep because maintenance for rutting, alligator cracking, and patching
= 10^{-6} x accumulative ADT because maintenance for raveling and flushing
= number of months because maintenance for transverse and longitudinal cracking.

The ρ and β equations are of the form, ρ = Constant A + B (avg temp − 50°F) + C (Thornthwaite index + 50) + D (thickness of base) + E (liquid limit) + F (plasticity index) − G (freeze-thaw cycles) + H (dynaflect max. deflection).

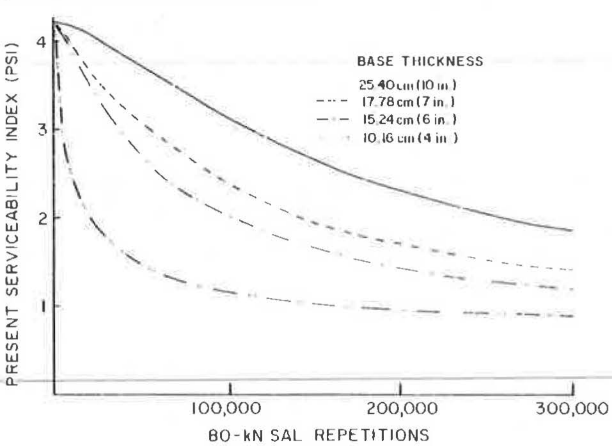
where

AVT = average district temperature, F - 50,
D = thickness of flexible base course,
LL = liquid limit of subgrade soil,
PI = plasticity index of subgrade soil,
FTC = average number of annual air freeze-thaw cycles, and
TI = Thornthwaite (moisture) index + 50.

Equations for ρ and β such as the preceding have been generated for each of the eight distress types and PSI. A complete listing is given in Table 2. The correlation coefficients (R^2) of these equations in general range from 0.30 to 0.60. For a few distress types, particularly raveling and flushing, no acceptable models were found. In these instances the mean values of ρ and β were used for predictive purposes.

Like other pavement distress predictive models reported in the literature, the models used in this study generally have low R^2 values. The cause of these low R^2 values can be traced to several sources, including subjectivity of rating and nonavailability of some important variables. To justify the use of these models two approaches were taken. First, their predictions of pavement performance were compared with actual performance (see Figure 2 and the discussion that follows). Second, a team of experienced field engineers was asked to audit the equation's pavement life predictions. Predictions, such as those shown in Figures 4 and 5, were shown to a panel of experienced engineers. They concluded that these predictions appeared reasonable for this type of pavement under the specified loading and environmental conditions.

Figure 4. Predicted PSI versus 80-kN axle load repetitions for surface-treated pavements of different base thicknesses.



Comparison of Equation Predictions with Actual Performance

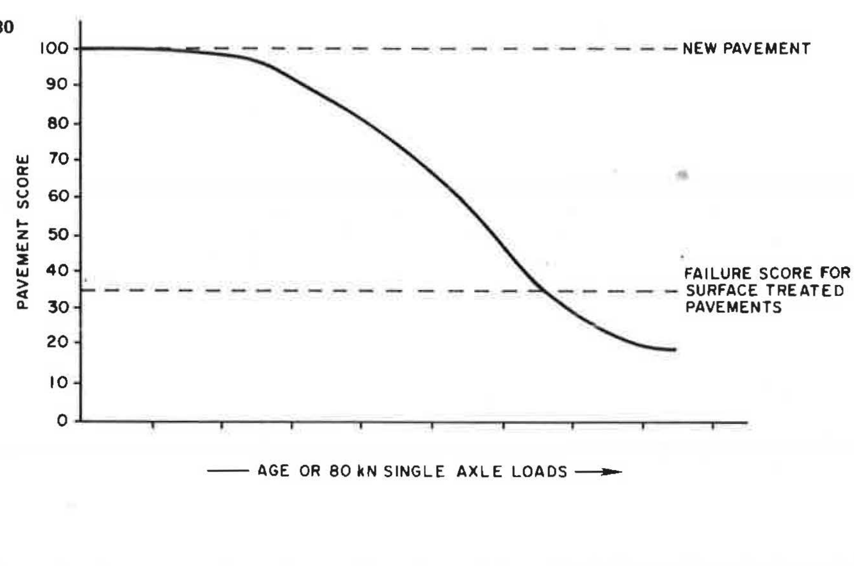
Several runs were made to test the validity of predicting pavement performance with these regression equations. Such a prediction using the PSI equation is shown in Figure 2. This is for Texas FM-556 in district 19, which is the section in the Texas Transportation Institute (TTI) flexible pavement data base shown in Figure 1. This section was reconstructed in 1969 and PSI measurements were made in 1974-1977. As can be seen from Figure 2, the Texas regression equations fit the observed data much better than did the regression equations developed at the AASHTO Road Test. This pavement had a structural number of approximately 1.0 and the AASHTO equation predicted a life until PSI = 1.5 of 5,000 80-kN axles, which under the actual traffic levels would be achieved in the first six months of service.

Further sensitivity analysis of the PSI equation is shown in Figure 4, where the effect of base thickness on PSI is predicted. These curves were generated from data collected on in-service pavements under normal traffic loads. The characteristic leveling off of the PSI curve is due partly to the application of routine maintenance by the state's personnel and partly to the nonlinearity of the relation between PSI and roughness. Pavements that have a low PSI, if they are not scheduled for major repair, frequently receive regular maintenance (e.g., patching and crack seal), which prevents further deterioration. In practice, few of the thin pavements in the data base were found to have a PSI, as measured by the Mays ride meter, of less than 1.5.

PREDICTIONS OF PAVEMENT LIFE

In the AASHTO Road Test damage was defined in terms of reduction in PSI. Damage was made more general in this study by applying it to distress as well as to a loss of serviceability index. Pavement condition (damage) was expressed in terms of a composite index that combines distress with loss in serviceability to produce a pavement score. Several states and agencies, including Arizona, Florida, Utah, and the U.S. Air Force, are using such a composite index

Figure 5. Expected pavement score versus age (or 80 kN) for surface-treated pavements.



(11). In general, these indices are used to determine which pavement sections are most in need of rehabilitation; the section that has the lowest score is the one most in need of repair.

Texas also uses this pavement score approach (12). A pavement utility score (range 0-1) is calculated by using the following equation and the final pavement score is equal to this utility score x 100.

$$\text{Pavement utility score} = U_{RIDE}^{a_1} \times U_{DIST}^{a_2} \tag{7}$$

where

U_{RIDE} = riding quality utility score of range 0-1,
U_{DIST} = visual distress ability score of range 0-1, and
a_1, a_2 = weighting factors on each utility score.

The visual distress utility score is further defined as

$$U_{DIST} = (U_{rut})^{b_1} (U_{ravel})^{b_2} (U_{flush})^{b_3} (U_{failures})^{b_4} (U_{allig})^{b_5} (U_{long})^{b_6} (U_{tran})^{b_7} \tag{8}$$

where each U_i value is determined from the visual inspection data and has a range of from 0 to 1.0 and the b_i values are weighting factors whose values depend on climatic factors such as rainfall and freeze-thaw cycles (12).

By using the Texas definition of pavement score, if any single utility value becomes low, the pavement utility score will be low. For instance, if the highway's ride value falls to a critical level, then the pavement score will drop to a failure level. Alternatively, a pavement score may reach failure by a combination of distress types but still maintain a high PSI. In Texas new pavements have a pavement score of 100 and (as shown in Figure 5) for surface-treated pavements the failure level is defined to be a pavement score of 35.

With the Texas pavement evaluation system (13), this pavement score can be used to determine which strategy should be used to rehabilitate those pavements below the minimum score. This is done by examining the principal causes of a low pavement score. For surface type distresses (i.e., transverse cracking, raveling, and flushing) a seal coat would be recommended. For other load-associated distress types (i.e., severe rutting, alligator cracking, and failures or loss in PSI) a sectional or full reconstruction may be recommended.

Predictions of Pavement Score from Pavement Distress Equation

A computer program was written to incorporate the Texas pavement distress equations and pavement score concepts discussed previously. The inputs required to make predictions of pavement performance are as follows:

1. Average daily traffic,
2. Percentage trucks,
3. Flexible base thickness,
4. Subgrade Atterberg limits (PI, LL) obtained from construction records or county soil reports,
5. Section maximum Dynaflect deflection obtained from field observation or elastic layered analysis, and
6. Texas county number--for each of the 254 Texas counties the program has stored the relevant climatic data, such as rainfall and average temperatures.

The program uses the input traffic data to calculate the expected 80-kN loading for the analysis period (20 years). It then uses the distress equations to predict pavement condition and, hence, pavement score for each year in the analysis period. When the pavement score reaches the failure level of 35, the number of months to failure is computed. Once failure has occurred which distress types have caused the reduction in pavement life and consequently which rehabilitation strategy would be most appropriate can be determined.

An example of pavement score predictions is shown in Figure 6. The highway was assumed to be in Burleson County, Texas, with its typical soil and climatic conditions. It carried an ADT of 400 vehicles per day (200 in each direction), 5 percent of which were trucks. Predictions have been made that assume a 4-, 6-, and 8-in. flexible base layer. The results from this figure are tabulated below.

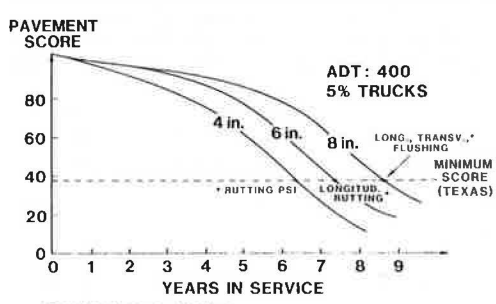
Base Thickness (in.)	Time to Failure for First Performance Period (years)	Predicted Distresses that Cause Major Reductions in Pavement Score
4	6.3	Rutting PSI
6	7.3	Longitudinal cracking Rutting
8	8.6	Longitudinal cracking Transverse cracking Flushing

This table gives the causes of pavement failure with the 4-in. pavement, which were predicted to be primarily load associated. This pavement would presumably require a sectional or full reconstruction. In contrast, the thick 8-in. pavement was predicted to fail by mainly nonload-associated distress types, such as transverse cracking. This 8-in. pavement would presumably only require a minimum treatment such as a seal coat to extend its life. Thus the developed computer program can be used to predict not only decreases in pavement life but also increases in the cost of pavement rehabilitation. Both the timing and the cost of rehabilitation strategies are essential inputs to any life-cycle cost analysis, as will be demonstrated in the following case study.

Predictions of Reduction in Pavement Life Associated with Oil Field Traffic

The traffic pattern associated with the drilling of a single oil well is shown in Figure 7. These data were recorded with an air tube-activated camera at the entrance to an oil well drilling site. The techniques employed and conclusions reached in that phase of this study are reported elsewhere (1).

Figure 6. Pavement score predictions versus years in service for surface-treated pavements of varying base thicknesses in Burleson County, Texas.



*Predicted cause of failure

Figure 7. Traffic pattern associated with drilling of single oil well.

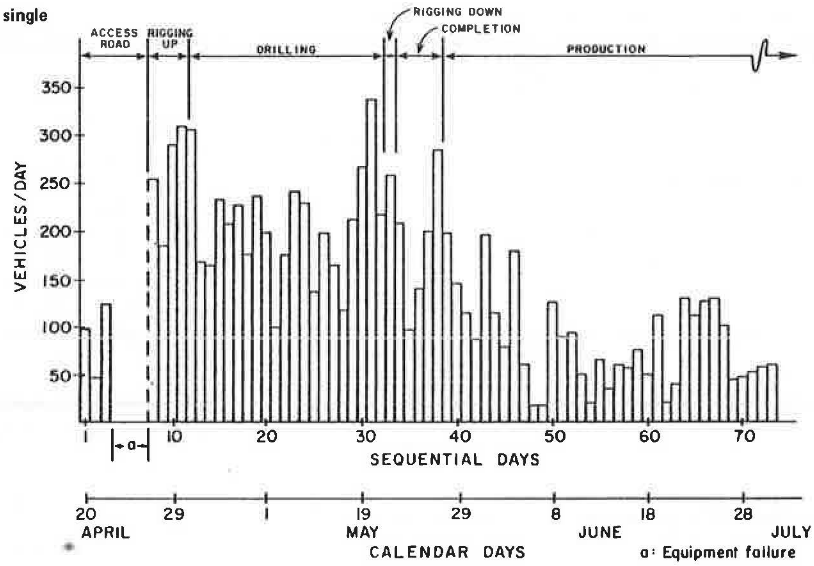


Table 3. Cumulative ADT and 80-kN ESAL repetitions for intended use and intended use plus oil wells starting in month 36.

Pavement Age (months)	Intended Use Analysis		Intended + 10 Wells Traffic	
	ADT	80-kN	ADT	80-kN
1	7,500	376	7,500	376
12	92,091	4,622	92,091	4,622
36	290,648	14,588	335,648	17,298
60	510,040	25,600	945,040	43,584
120	1,164,594	58,452	1,764,593	82,446

When the number of 80-kN axles associated with the drilling of a single oil well is known, it is possible to calculate the increase in axle loadings appropriate for any level of drilling activity. By using the computer program described it is possible to calculate the reduction in pavement life associated with the oil field development. This technique is demonstrated in the following case study.

Site Conditions

A severely impacted oil field area in Burleson County was chosen for this study. The climatic and subgrade parameters used as input to the program are listed as follows:

Item	Parameter
Mean annual temperature	67°F
Thornthwaite (moisture) index	2.10
Mean annual air freeze-thaw cycles	35.5
Subgrade liquid limit	42
Plasticity index	23

A typical base thickness for Burleson County is 6 in. and, from data collected on similar sites, a Dynaflect maximum deflection of 1.55 mils is appropriate.

For the purpose of this analysis the highway was assumed to carry an ADT of 500 vehicles per day, 5 percent of which were trucks, and a growth rate of 5 percent per year.

Traffic Analysis

The first phase of the analysis included a calculation of the intended use traffic levels (ADT, 80-kN axles) during the analysis period (20 years). Also, traffic levels were calculated by assuming that the highway under investigation was impacted with oil field development traffic after 36 months. In this example three levels of drilling activity were investigated--5, 10, and 20 wells. A sample of the predictions of traffic level are given in Table 3.

Pavement Performance

The traffic levels presented in Table 3 were used with the previously described regression equations to predict the pavements' PSI and distress levels under the conditions of intended use and intended use plus oil field traffic. Pavement score calculations were performed and the time to failure under each of the loading conditions was computed. The results are shown in Figure 8 and have been tabulated in Table 4.

As would be expected, the increased oil field traffic drastically reduces the time to failure of these thin pavements. Under the oil field traffic associated with 20 wells, the highway's life was reduced from 82 to 52 months. When the oil field traffic was impacted in month 36, the highway still had a perfect score of 100. In just over 1 year this score was reduced to the failure level, at which point the highway will require total reconstruction.

Rehabilitation Costs

The analysis of PSI levels and distress levels at failure indicates that, under intended-use traffic, the primary causes of the pavement score reaching failure level are surface distress types (e.g., transverse cracking, raveling, or flushing). Under the oil field traffic, with its high intensity of heavy traffic, load-associated distress (e.g., rutting and alligator cracking) become the primary causes of pavement failure.

These results are not surprising. It is common to find many thin pavements that only require regular reseals to prolong their lives, whereas when these pavements carry much heavier than anticipated traffic, rapid pavement deterioration can result. The implication of this for our study is that failure under intended-use traffic will only require a seal coat to prolong pavement life, whereas under the traffic associated with 20 oil wells, full re-

Figure 8. Prediction of reductions in pavement life associated with different levels of oil field activity.

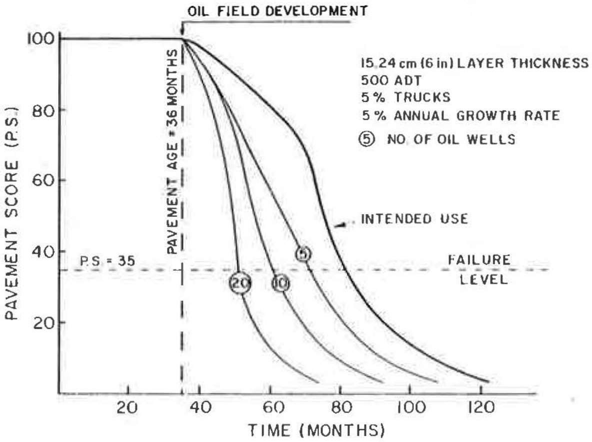


Table 4. Predicted pavement life for various levels of oil field activity.

Traffic Level	Time of Failure (months)	Reduction in Life (months)
Intended + 0 wells	82	0
Intended + 5 wells	73	9
Intended + 10 wells	61	21
Intended + 20 wells	52	30

construction is necessary. These costs (obtained from recent completion plans) are summarized in the table below.

Traffic Level	Time to Failure (months)	Rehabilitation Treatment	Rehabilitation Cost ($/yd²)
Intended traffic	82	Seal coat	0.50
Intended traffic + 20 wells	52	Rework of base + 2 in. of base + surface treatment	5.20

Thus, as has been observed in many cases of oil field impact, much higher rehabilitation costs are incurred earlier in the pavements' life. Both of these costs are inputs to the final life-cycle cost analysis.

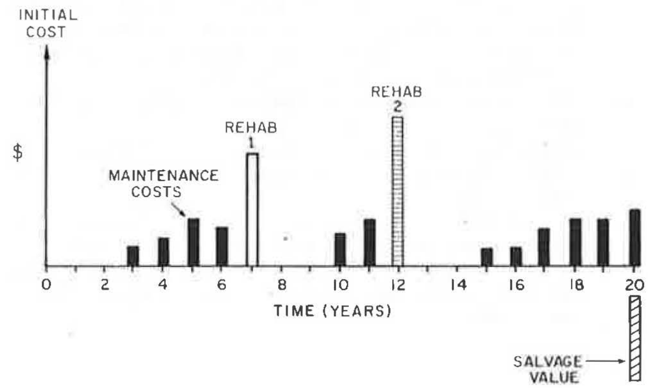
CONCLUSIONS

In order to study the effects of heavy oil field traffic on surface-treated pavements, the following approach was taken:

1. Pavement distress and performance equations were developed by regression analysis from data collected on thin pavements in Texas,
2. A traffic analysis was performed to calculate the increase in 80-kN ESALs attributable to the oil field traffic, and
3. Predictions were made of pavement life under intended use traffic and intended use plus oil field traffic (pavement life is defined by a composite index that includes serviceability index and distress types).

By using this approach, large decreases in pavement life associated with the oil field traffic are predicted (see Table 4). The long-term objective of this study is to develop for TSDHPT a procedure for

Figure 9. Schematic of components of life-cycle cost analysis.



calculating the additional life-cycle costs (14) incurred on thin pavements by oil field traffic. A schematic of anticipated life-cycle cost is shown in Figure 9.

The work in the current phase has concentrated on predicting the timing and cost of pavement rehabilitation under both intended use and intended use plus oil field traffic. Future work will involve quantification of the additional life-cycle costs for an oil-impacted area.

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Development of a Prioritization Procedure for the Network Level Pavement Management System

EMMANUEL G. FERNANDO AND W.R. HUDSON

Over the years funding for maintenance, rehabilitation, restoration, and resurfacing activities has not kept pace with the needs of highway agencies. Consequently, development of a system for managing the pavement network and, in particular, for assisting highway agencies in the efficient allocation of their resources to make the best possible use of the limited funds available has become more necessary. An integral component of any pavement management system is a procedure for establishing priority listings for rehabilitation and maintenance activities. The material reported here documents efforts made to formulate a procedure for establishing priority order by using a method that will lead to a more realistic and rational way of establishing candidate projects for priority programming at the network-level pavement management system. The method presented is based on a factorial design that involves a set of candidate decision variables such as distress and present serviceability index. For this reason it has been termed the rational factorial rating method. Application of the method to the formulation of a priority-setting procedure is discussed, together with the results obtained. The method may provide a better understanding of how decisions on priorities are made in practice.

The development of systematic procedures for scheduling maintenance and rehabilitation activities is one of the major concerns of state and federal highway agencies today. This is primarily because, over the years, funding for maintenance, rehabilitation, restoration, and resurfacing activities has not kept pace with the needs of highway agencies throughout the United States. Many of these agencies now have a backlog of projects. The problem is further compounded by the reduced buying power of the U.S. dollar because of inflation. Consequently, the amount of work that can be accomplished with a given amount of money has been reduced significantly.

The problems that confront highway engineers today demand good management of existing road networks and have led to increased interest in the development and implementation of pavement management systems (PMS) methodology. Basic features of an implemented pavement management system are shown in Figure 1 (1). As can be seen from the figure, pavement management operates at two levels--the network level and the project level. Activities at the network level are mainly the responsibility of administrators and are primarily connected with the establishment of decisions that cover large groups of projects or an entire highway network. On the other hand activities at the project level are concerned with more specific technical management decisions for individual projects.

At the network-level PMS, inventory data are used to assess the status and needs of the highway network as a whole, and decisions are made about which rehabilitation and maintenance projects to include in the coming work program and which ones to defer for another year. The selection of candidate projects for rehabilitation and maintenance work is handled through a priority analysis in which inventory data are used to assess the adequacy of pavement sections versus a set of decision criteria. To quantify the degree of adequacy or acceptability and to facilitate comparisons among pavement sections, scores are generally calculated for each pavement section by using a procedure established within the particular agency involved. The scores so obtained can then be used for establishing priority listings for rehabilitation and maintenance work.

The development of a variable for establishing priorities is therefore a necessary ingredient in the pavement management process, and highway agencies have set up various procedures for determining priority-ordered indices. Procedures used by several highway agencies are documented elsewhere (2). In most cases a combined rating or score is used to express the overall condition of the pavement in terms of a combination of selected attributes.

Several approaches have been used to combine various attributes into a single score for priority ranking of rehabilitation and maintenance projects. For example, a common procedure involves the establishment of sufficiency or deficiency ratings for various categories of selected pavement attributes. In addition, the application of utility theory for formulating a joint index has been reported for Arizona and Texas (3,4).

In this paper a method for formulating an index for priority ranking of rehabilitation projects is presented. The method, known as the rational factorial rating method, can provide a suitable medium for quantifying the opinions of highway engineers

Figure 1. Key features of pavement management system implemented.

NETWORK	PROJECT
• INVENTORY DATA BASE	• DESIGN METHOD - TESTS, MODELS, CRITERIA - ECON. ANAL. & OPTIMIZATION
• ASSESSMENT OF STATUS AND NEEDS	• CONSTR. CONTROL PROCED.
• PRIORITY PROGRAMS OF CAPITAL & MAINT. WORK	• MAINT. STANDARDS
• FUNDING LEVEL EVALUATION	• AS-BUILT & MAINT. DATA FILES

DOCUMENTATION OF METHODS AND PROCEDURES

regarding the establishment of rehabilitation priorities. The development of the method and its application to the problem of formulating a joint index are discussed in the following section.

DEVELOPMENT OF RATIONAL FACTORIAL RATING METHOD

As indicated previously, an important component of any PMS is a procedure for establishing the priority order of rehabilitation and maintenance activities. In practice, one method of formulating a priority index would be to select a number of pavement sections that are representative of a wide range of field conditions and have a panel of engineers rate them on the basis of a selected set of attributes. The ratings obtained, together with physical measurements made on the pavement sections, can then be used to arrive at an equation for priority ranking of pavements. This approach is somewhat similar to the procedure used in developing the present serviceability index (PSI) at the AASHO Road Test.

However, as an alternative to having a panel of engineers go out in the field to selected pavement sections, it may also be possible to quantify the opinions of these engineers through a series of structured questions. These questions could consist of several scenarios that describe pavement sections under conditions in which they might exist in the field. These conditions could be defined by combinations of levels of selected pavement attributes arranged in some kind of a factorial design. By asking highway engineers to indicate a rating of rehabilitation need for each of the pavement sections described to them, it would be possible to study (and perhaps gain a better understanding of) how highway engineers establish rehabilitation priorities in practice. Efforts would therefore be made to arrive at a specific factorial design through which the ideas presented could be made concrete.

Development of Initial Factorial Design

Two factorial designs were used in the study reported here. The initial factorial design is shown in Figure 2. Each of the cells in the figure (identified with a particular number) can be treated as a pavement section experiencing a particular and unique set of conditions of pavement distress, PSI, traffic level, and environment-related factors. By asking highway engineers to rate each cell on the basis of the priority they would assign to each, it would be possible to estimate how they establish rehabilitation priorities in practice. It would not be practical, however, to solicit the opinions of each pavement expert for all of the cells shown in the figure. Doing so would impose a heavy burden on each respondent and might be confusing. This concern led to the selection of a fractional factorial design based on a half-replicate of the full factorial (see Figure 2). With this plan a pavement engineer is asked to provide priority ratings only for certain selected combinations of the variables considered. In Figure 2 these are the cells marked with an X.

A fractional factorial will not give as much information as a full factorial. The design shown in Figure 2 enables one to estimate the main effects of each variable and certain two-factor interactions. This information was considered adequate for the purposes of the study. In addition, distress, PSI, traffic, freeze-thaw, and rainfall were included as variables in the design and the distress factor was fixed at three levels and the remaining ones were fixed at two levels. The distress factor was fixed at three levels in order to check for nonlinearity

Figure 2. Initial factorial design.

Pavement Distress Evaluation			Significant		Moderate		Minimal	
PSI								
Traffic Level*			2.4	3.5	2.4	3.5	2.4	3.5
Wet	Freeze Thaw	High	101 X	102	103	104 X	105 X	106
		Low	107	108 X	109 X	110	111	112 X
	No Freeze Thaw	High	113	114 X	115 X	116	117	118 X
		Low	119 X	120	121	122 X	123 X	124
Dry	Freeze Thaw	High	125	126 X	127 X	128	129	130 X
		Low	131 X	132	133	134 X	135 X	136
	No Freeze Thaw	High	137 X	138	139	140 X	141 X	142
		Low	143	144 X	145 X	146	147	148 X

* Low≈ 6000 ADT
High≈ 100,000 ADT

of responses that may be associated with this variable.

After the factorial design was selected a rating form was prepared for soliciting the opinions of pavement engineers. A sample page from the rating form is shown in Figure 3 [a copy of the complete version of the rating form may be found elsewhere (5)]. The pavement sections included in the form represent the cells marked with an X in Figure 2. A half-replicate of a 3×2^4 factorial (24 sections) was considered too much for a respondent to compare and rate at one time, and the decision was made to divide the half-replicate into two blocks that consist of 12 pavement sections each. Each pavement expert consulted was asked to assign priority ratings on a scale of 1.0 to 10.0, with 1.0 indicating a pavement section that has a high rehabilitation priority and 10.0 indicating a pavement section that has a low rehabilitation priority. In addition, the respondents were instructed to assign ratings to 12 pavement sections at a time. Whenever possible, the respondents were given a break between rating sessions.

The division of the half-replicate into two blocks may have an effect on the responses provided by the participants, and this possibility was recognized by the investigators. In particular, the responses for pavement sections included in one block may turn out to be different from those in the other block. Any difference in the responses between blocks would be hard to explain. However, the division of the half-replicate into two blocks was thought to not really have a significant effect, and this was later confirmed when the responses obtained were analyzed.

Development of Second Factorial Design

In addition to the development of the research design discussed previously a second factorial design was made to investigate whether the type of pavement has significant influence on the way rehabilitation priorities are established. The second design is essentially a modification of the initial research design. An examination of the responses to the original design showed no significant nonlinearity of responses associated with the distress variable. Consequently, the changing of this variable from three levels in the initial factorial design to only two levels seemed reasonable. With this modification, and with the inclusion of pavement type as another factor, a second research plan was established, a half-replicate of a 2^6 factorial design (32 sections).

The second fractional factorial design is illustrated in Figure 4 and a sample page from the rating form prepared for this design is shown in Figure 5. Instructions for filling out the second rating form are essentially the same as those for the initial rating form. In the second factorial design, however, the 32 pavement sections were divided into 4 blocks of 8 sections each. This was done because to have a respondent rate the pavement sections all at the same time would be too much of a burden. Consequently, participants were told to rate only 8 sections at a time, and, whenever possible, a break was given after half of the total number of sections were rated.

EVALUATION OF PRIORITY RATINGS OBTAINED THROUGH APPLICATION OF RATIONAL FACTORIAL RATING METHOD

Determination of Significant Variables

Summary statistics calculated for the priority ratings for the initial factorial design are given in Table 1. To determine which of the variables had a significant influence on the responses obtained a regression equation was calculated of the average responses given in Table 1 as a function of the independent variables included in the study. The regression equation obtained is expressed as Equation

Figure 3. Sample rating sheet for first factorial design.

Indicate your major work area by putting an X in the appropriate space below:

____Administrative ____Construction Date: ________________

____Design ____Materials & Testing

____Maintenance ____Others (specify ____________________)

Assign ratings for the following pavement sections:

Section No.	Environmental Condition	Traffic Level*	PSI	Pavement Distress Evaluation	Rating of Rehabilitation Need	Should Pavement Section Be Considered a Candidate for Rehabilitation?
123	Wet, No Freeze Thaw	Low	2.4	Minimal Distress	________	YES:____NO:____
130	Dry, Freeze Thaw	High	3.5	Minimal Distress	________	YES:____NO:____
137	Dry, No Freeze Thaw	High	2.4	Significant Distress	________	YES:____NO:____
122	Wet, No Freeze Thaw	Low	3.5	Moderate Distress	________	YES:____NO:____
135	Dry, Freeze Thaw	Low	2.4	Minimal Distress	________	YES:____NO:____
118	Wet, No Freeze Thaw	High	3.5	Minimal Distress	________	YES:____NO:____
108	Wet, Freeze Thaw	Low	3.5	Significant Distress	________	YES:____NO:____
134	Dry, Freeze Thaw	Low	3.5	Moderate Distress	________	YES:____NO:____
101	Wet, Freeze Thaw	High	2.4	Significant Distress	________	YES:____NO:____
127	Dry, Freeze Thaw	High	2.4	Moderate Distress	________	YES:____NO:____
144	Dry, No Freeze Thaw	Low	3.5	Significant Distress	________	YES:____NO:____
115	Wet, No Freeze Thaw	High	2.4	Moderate Distress	________	YES:____NO:____

*Low ∿ 6000 ADT; High ∿ 100,000 ADT

1. This equation has a value of R^2 = 97.1 percent and a standard error of estimate (SEE) equal to 0.31.

$$Y = 5.26 + 0.46X_1 + 0.396X_2 + 0.601X_3 + 0.749X_4 + 1.66X_5 - 0.0568X_6 - 0.0036X_7 \quad (1)$$

The dependent variable (Y) in Equation 1 represents the predicted priority rating. The first five independent variables (X_1 to X_5) represent, respectively, the following variables:

1. Rainfall,
2. Freeze-thaw,
3. Traffic,
4. PSI, and
5. Distress (linear component).

Values of these independent variables were coded in the following manner in the analysis. A value of +1 was assigned when a variable was at its best level, and a value of -1 was used when it was at its worst level. For the distress factor that was fixed at three levels, a value of zero was used to indicate a pavement that has a moderate degree of distress.

The remaining two variables are explained as follows. Variable X_6 is used to represent the quadratic component of the distress factor. As mentioned previously this factor was set at three levels to verify whether or not there is a significant nonlinearity of responses that may be associated with this variable. The X_6 factor in Equation 1 is therefore used to verify that a significant nonlinearity does exist, and values for

Figure 4. Second factorial design.

Pavement Type			Rigid				Flexible			
Pavement Distress Evaluation			Significant		Minimal		Significant		Minimal	
PSI			2.4	3.5	2.4	3.5	2.4	3.5	2.4	3.5
Traffic Level*										
Wet	Freeze Thaw	High	201 X	202	203	204 X	205	206 X	207 X	208
		Low	209	210 X	211 X	212	213 X	214	215	216 X
	No Freeze Thaw	High	217	218 X	219 X	220	221 X	222	223	224 X
		Low	225 X	226	227	228 X	229	230 X	231 X	232
Dry	Freeze Thaw	High	233	234 X	235 X	236	237 X	238	239	240 X
		Low	241 X	242	243	244 X	245	246 X	247 X	248
	No Freeze Thaw	High	249 X	250	251	252 X	253	254 X	255 X	256
		Low	257	258 X	259 X	260	261 X	262	263	264 X

*Low ≈ 6000 ADT
High ≈ 100,000 ADT

Figure 5. Sample rating sheet for second factorial design.

Date:__________

INDICATE YOUR MAJOR WORK AREA BY PUTTING AN X IN THE APPROPRIATE SPACE BELOW:

___Administrative ___Construction
___Design ___Materials and Testing
___Maintenance ___Others (specify________________)

ASSIGN RATINGS FOR THE FOLLOWING PAVEMENT SECTIONS:

Section No.	Pavement Type	Environmental Condition	Traffic Level*	PSI	Pavement Distress Evaluation	Rating of Rehabilitation Need	Should Pavement Section be Considered a Candidate for Rehabilitation?
258	Rigid	Dry, No Freeze Thaw	Low	3.5	Signif. Distress	______	YES:___ NO:___
213	Flexible	Wet, Freeze Thaw	Low	2.4	Signif. Distress	______	YES:___ NO:___
206	Flexible	Wet, Freeze Thaw	High	3.5	Signif. Distress	______	YES:___ NO:___
231	Flexible	Wet, No Freeze	Low	2.4	Minimal Distress	______	YES:___ NO:___
235	Rigid	Dry, Freeze Thaw	High	2.4	Minimal Distress	______	YES:___ NO:___
249	Rigid	Dry, No Freeze Thaw	High	2.4	Signif. Distress	______	YES:___ NO:___
224	Flexible	Wet, No Freeze Thaw	High	3.5	Minimal Distress	______	YES:___ NO:___
244	Rigid	Dry, Freeze Thaw	Low	3.5	Minimal Distress	______	YES:___ NO:___

* Low ∿ 6000 ADT; High ∿ 100,000 ADT

Table 1. Means and standard deviations of priority ratings, and confidence interval estimates of mean ratings for initial factorial design.

Section No.	Mean Rating	SD	Confidence Interval Estimate of Mean Rating (95% Level)
123	6.61	1.78	6.08-7.14
130	7.65	1.55	7.18-8.11
137	3.39	1.36	2.99-3.80
122	6.42	1.63	5.93-6.91
135	6.52	1.79	5.98-7.05
118	7.54	1.47	7.10-7.98
108	4.19	1.47	3.75-4.63
134	6.53	1.32	6.13-6.92
101	1.40	0.84	1.15-1.65
127	3.85	1.29	3.66-4.05
144	5.30	1.65	4.81-5.80
115	3.67	1.19	3.32-4.03
105	4.44	1.92	3.86-5.01
131	3.94	1.52	3.49-4.39
104	4.16	2.07	3.54-4.77
145	5.93	1.68	5.42-6.43
140	6.21	1.50	5.76-6.66
148	9.28	0.98	8.99-9.57
119	3.71	1.33	3.31-4.11
141	6.26	1.72	5.74-6.77
112	7.50	1.58	7.03-7.97
114	3.55	1.62	3.06-4.03
109	4.41	1.49	3.96-4.85
126	3.79	1.70	3.28-4.30

this variable were coded in the following manner. A value of -1 was assigned to correspond to the minimal and significant levels of distress, and a value of +2 was assigned to correspond to the moderate level of distress. In addition X_7 is the factor used to check whether responses differed significantly between the component blocks in which the design was divided. For this factor, a value of +1 was used to represent pavement sections that belong to the first block, and a value of -1 was used to represent pavement sections that belong to the second block.

To illustrate how well the equation fits the ratings obtained, a plot of the residuals versus the predicted priority ratings is shown in Figure 6. The plot does not indicate any dependence of the residuals on the magnitudes of the equation values. In addition, examination of the figure does not reveal any outlier in the data obtained.

A test for the strength of the functional relationships between each independent variable and the dependent variable was made to determine the significant factors. In connection with this, t-statistics (Table 2) for the coefficients in Equation 1 were calculated and compared with a value of t = ±2.12, which corresponds to a 95 percent confidence level and (24 - 8) = 16 degrees of freedom (d.f.).

Table 2. Computed t-statistics for coefficients of Equation 1.

Variable	Coefficient	t-Statistic
Rainfall, X_1	0.460	7.22
Freeze-thaw, X_2	0.396	6.21
Traffic, X_3	0.601	9.43
PSI, X_4	0.749	11.75
Distress		
Linear component, X_5	1.656	21.22
Quadratic component, X_6	-0.057	-1.26
Block effect, X_7	-0.004	-0.06

The results indicate that the variables rainfall, freeze-thaw, traffic, PSI, and distress have a significant influence on the establishment of priorities for rehabilitation work. In addition, the analysis indicates no significant nonlinearity of responses associated with the distress variable. In other words, the priority ratings obtained vary more or less linearly with the degree of distress. Finally, the results indicate that the division of the factorial into blocks had no significant effect on the responses obtained.

Estimation of Interaction Between Variables

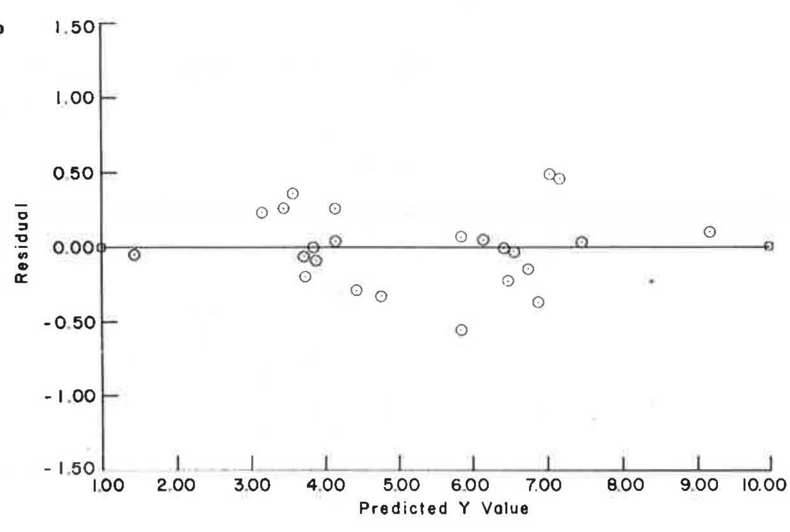
Responses were also evaluated to check for interactions among the variables included in the factorial design. In the analysis only two-factor interactions were considered. Three-factor and higher-order interactions were used for estimating the residuals.

To facilitate the discussion of the analysis, each of the main variables was represented by a letter as shown below:

Variable	Symbol
Rainfall	A
Freeze-thaw	B
Traffic	C
PSI	D
Distress	E

To determine whether any significant interactions occurred among variables, a regression equation of

Figure 6. Plot illustrating goodness-of-fit of Equation 1 to average priority ratings obtained for first factorial design.



the average responses as a function of the main variables and selected two-factor interactions was calculated. These two-factor interactions were AC, AD, AE, BC, BD, BE, CE, and DE. AB and CD were not included in the analysis because these interactions were confused in the fractional factorial design.

The strength of the relationship between each two-factor interaction and the dependent variable was evaluated by using a t-test to determine which of the two-factor interactions were significant. In connection with this, t-statistics were computed for the coefficients of the interaction terms (see Table 3) and were compared with the value of t = ±2.228 for (24 - 14) = 10 d.f. and a 95 percent confidence level. The results indicate that a significant interaction exists between PSI and distress. This interaction is illustrated in Figure 7, where the average responses were plotted as a function of the two variables. As can be seen in the figure, lines fitted to the data are not quite parallel, which indicates that an interaction does exist. An increase in the level of the distress factor at PSI = 2.4 does not produce quite the same change in the responses as at PSI = 3.5.

Note that, for each level of distress and PSI, one observation seems to plot quite differently from the rest. These points represent ratings for pavement sections where the values of the other variables--rainfall, freeze-thaw, and traffic (which are not accounted for in Figure 7)--are either all at their best levels or all at their worst levels. For this reason, such observations seem to plot either much higher or much lower than the other observations that correspond to a particular level of distress and PSI.

Although the DE interaction turned out to be statistically significant in the analysis, the sum of squares associated with this interaction is only about 1.2 percent of the total sum of squares. In contrast, the five main variables taken together already account for about 97.7 percent of the total variation. As such, although the DE interaction is statistically significant, a sufficient amount of the total variation in the priority ratings can already be explained by the five main variables. Consequently, the DE interaction may be ignored for practical purposes.

Evaluation of Influence of Pavement Type

A regression equation of the average priority ratings for the second factorial design was calculated in order to verify whether pavement type has a significant influence on the ratings obtained. For the analysis a value of -1 was used as the code for rigid pavements, and a value of +1 was used for flexible pavements. Values for the other five main variables were coded in the same way as was done for the analysis of priority ratings for the initial factorial design.

The coefficients of the variables in the computed regression equation are given in Table 4. The last three variables in the table are used to verify whether the division of the second factorial design into four blocks had any significant effect on the responses obtained.

Tests of significance for the coefficients of the independent variables were made, and the calculated t-statistics for the coefficients are given in Table 4. Comparison of the values of each of these statistics with the value of t = ±2.074 for (32 - 10) = 22 d.f. and 95 percent confidence level shows that only the first five main variables are significant. The results do not indicate differences in the responses obtained between blocks. This indicates that the division of the second factorial design

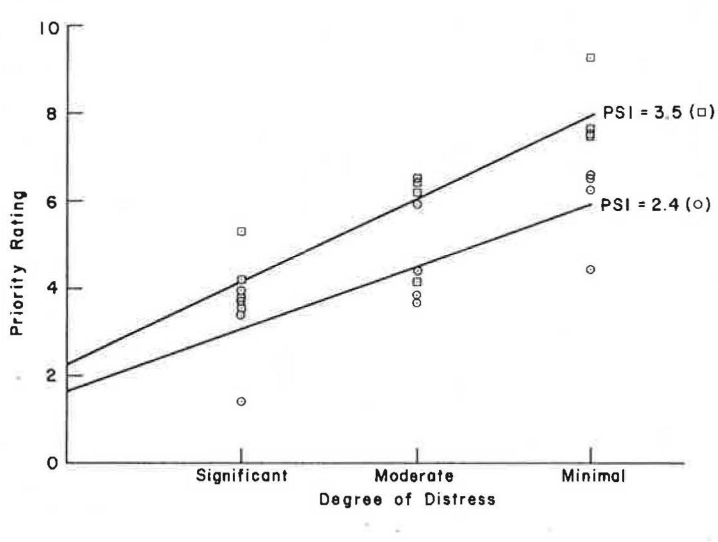
Table 3. Computed t-statistics for two-factor interactions.

Two-factor Interaction	Coefficient	t-Statistic
AC	-0.0731	-1.47
AD	0.0068	0.14
AE	0.0021	0.04
BC	-0.0508	-1.02
BD	0.0033	0.07
BE	0.0600	1.04
CE	-0.0620	-1.08
DE	0.2350	4.09

Table 4. Computed t-statistics for coefficients of variables in regression equation for second factorial design.

Variable	Coefficient	t-Statistic
Rainfall, X_1	0.389	5.94
Freeze-thaw, X_2	0.236	3.60
Traffic, X_3	0.735	11.22
PSI, X_4	0.872	13.31
Distress, X_5	1.370	20.91
Pavement type, X_6	-0.079	-1.21
X_7	-0.040	-0.61
X_8	0.060	0.91
X_9	-0.020	-0.30

Figure 7. Graph illustrating dependence of effect of distress on PSI.



into four blocks was not arbitrary and that any other division would have yielded similar results.

In addition, the analysis seems to indicate that, when decisions on priorities are made, consideration of whether a pavement is flexible or rigid is probably not as important as consideration of the other main variables, such as the degree of distress of a pavement section, its PSI, and the volume of traffic passing over. Another interpretation may be that, given a flexible pavement and a rigid pavement under similar conditions, a highway engineer would feel the same about the rehabilitation need for both pavements.

Comparison of Results of First and Second Factorial Designs

To check for similarity in the results of the first and second factorial designs, a comparison of the standardized regression equations calculated for both surveys was made. For the analysis pairwise comparisons of the beta coefficients were made by using a t-test. For each pair of coefficients the t-statistic expressed as Equation 2 was calculated:

$$t = (\beta_{i,1} - \beta_{i,2})/(\sigma_{\beta_{i,1} - \beta_{i,2}}) \tag{2}$$

where

$\beta_{i,1}$ = beta coefficient for the ith independent variable in the standardized regression equation for the first survey,

$\beta_{i,2}$ = beta coefficient for the ith independent variable in the standardized regression equation for the second survey, and

$\sigma_{\beta_{i,1} - \beta_{i,2}}$ = standard error of the difference between the beta coefficients.

In the analysis, an estimate of the standard error was made by using a pooled estimate of variance. The t-statistics calculated using Equation 2 are summarized in Table 5. By comparing the computed t-statistics with the value of $t \simeq \pm 2.017$ corresponding to a 95 percent confidence level and 44 d.f., it is seen that the beta coefficients of the regression equations are not significantly different from each other. Consequently, the analysis seems to indicate that the first and second surveys yielded similar results.

Table 5. Computed t-statistics for pairwise comparisons of beta coefficients of standardized regression equations for first and second factorial designs.

Variable	$\beta_{i,1}$	$\beta_{i,2}$	t
Rainfall, X_1	0.252	0.205	0.97
Freeze-thaw, X_2	0.217	0.124	1.93
Traffic, X_3	0.329	0.387	-1.20
PSI, X_4	0.410	0.459	-1.02
Distress, X_5	0.757	0.722	0.72

PRIORITY ORDERING PROCEDURE FOR SIMPLIFIED NETWORK-LEVEL PMS

Based on the results of the preceding analyses, the variables distress, PSI, traffic, rainfall, and freeze-thaw form an adequate set of criteria to use for a network-level priority ordering procedure. The results of the two surveys conducted are quite similar; therefore, the responses obtained from either one may be used to establish an equation for calculating priority ratings as a function of the five variables. In the analysis the average priority ratings from the first survey were regressed with the values of the independent variables shown below:

Variable	Best Level	Worst Level
Rainfall (X_1)	5 in./year	40 in./year
Freeze-thaw (X_2)	0 cycles/year	60 cycles/year
Traffic (X_3)	100 ADT	100,000 ADT
PSI (X_4)	4.0	2.0
Distress (X_5)	+1.0	-1.0

The regression equation obtained from the analysis is

$$Y = 5.4 - 0.0263X_1 - 0.0132X_2 - 0.40\mathrm{Log}_{10}X_3 + 0.749X_4 + 1.66X_5 \tag{3}$$

R^2 = 97.8 percent SEE = 0.31

The values used for the first four variables are thought to be representative of the conditions that might exist in the field. For the distress variable the categories listed in Table 6 may be used as a guide in the determination of the appropriate numerical values to use for various degrees of distress.

The categories for the distress variable are those that were used in the initial survey form. Although the categories are expressed in terms of word descriptions, it is also possible to use some quantified distress score should this be desired. A uniform set of verbal descriptions that characterizes qualitatively the degree of distress may, however, serve as a practical guide for evaluating pavement condition. In particular, because pavement condition is expressed in terms that are familiar to a field engineer, the use of a uniform set of word descriptions may be related to easily. In addition, verbal descriptions may help to guide the choice of maintenance or rehabilitation treatments. As such,

Table 6. Suggested categories for distress variable in Equation 3.

Category	Level of Distress	Numerical Value
Rigid pavements	Minimal-5 or fewer failures per mile, some minor spalling, little or no pumping at edges and longitudinal joints	+1.0
	Moderate-6-13 failures per mile, fair percentage of minor spalling in pavement section, some severe spalling, moderate pumping at edges and longitudinal joints	0.0
	Significant-14 or more failures per mile, fair to substantial amounts of severe spalling, moderate to extensive pumping at edges and longitudinal joints	-1.0
Flexible pavements	Minimal-Slight cracking, little or no rutting, and slight alligatoring in a few areas	+1.0
	Moderate-Intermittent moderate cracking with some spalling, frequent slight cracking, and intermittent slight or moderate alligatoring and rutting	0.0
	Significant-Extensive moderate cracking and rutting, frequent moderate alligatoring	-1.0

Table 7. Hypothetical data used in example and calculated priority-ordering indices.

Project Number	Amount of Rainfall (in./year)	Amount of Freeze-Thaw (cycles/year)	Traffic Level (000s ADT)	PSI	Distress Rating	Priority Index
8-2340-ND	30	25	20	3.4	+1.0	6.77
10-1029-EB	15	20	30	3.1	0.0	5.27
2-3471-WB	25	35	60	3.2	0.5	5.60
9-6189-NB	20	40	40	2.7	-1.0	2.87
12-5309-SB	29	28	50	3.6	+1.0	6.74
14-3070-WB	15	20	75	2.5	-0.05	3.83
7-6571-NB	35	36	38	3.0	0.0	4.42
3-6352-EB	23	18	20	2.5	-1.0	3.05

Table 8. Priority rankings of pavement sections based on calculated priority-ordering indices.

Project Number	Priority Index	Ranking
9-6189-NB	2.87	1
3-6352-EB	3.05	2
14-3070-WB	3.83	3
7-6571-NB	4.42	4
10-1029-EB	5.27	5
2-3471-WB	5.60	6
12-5309-SB	6.74	7
8-2340-NB	6.77	8

their use, in conjunction with quantified measures of distress, is also worth consideration.

Sample Application of Priority-Ordering Procedure

To illustrate how the procedure is used in establishing priorities for rehabilitation work, consider the hypothetical pavement sections given in Table 7. For simplicity, the sections listed are assumed to be rigid pavements in the example. Each section is represented by a project number, and data on the amount of rainfall, amount of freeze-thaw, traffic level, PSI, and distress rating are given for each pavement section. The categories listed in Table 6 for the distress variable may be used as a guide for defining the distress ratings given in Table 7.

By using Equation 3 a priority-ordered index is calculated for each of the sections listed in Table 7. The computed indices can then be used for assigning priority rankings to the given sections (Table 8).

The results show that the section that has project number 9-6189-NB has the highest priority for rehabilitation work, followed by section 3-6352-EB. Although both of these sections have the same distress ratings, section 9-6189-NB has twice as much traffic as the other section, and it is in an area with a greater amount of freeze-thaw cycling. As such, a higher priority ranking was assigned to section 9-6189-NB.

The results obtained for sections 8-2340-NB and 12-5309-SB are also worth considering. Both of these sections have the same distress ratings and are located in areas that have similar environmental characteristics. In addition, the PSIs for both sections are not that different from each other. However, section 12-5309-SB carries a significantly larger volume of traffic than section 8-2340-NB. Therefore, a higher priority ranking was assigned to section 12-5309-SB.

SUMMARY

A method for formulating a priority-ordering index has been presented. The method is based on a factorial design involving a set of candidate decision variables such as distress and PSI. For this reason, it has been termed the rational factorial rating method.

The application of this method to the formulation of a priority-ordering index has also been presented. Numerous pavement engineers were consulted, and the responses obtained were analyzed. In addition, an equation was developed that can be used for priority programming in a simple network-level PMS.

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Pavement Management—One City's Experience

ROBERT F. CARMICHAEL, III AND JAMES B. O'GRADY

The development of a pavement evaluation and management system (PMS) by the city of Arvada, Colorado, is presented. The approaches and techniques used are presented to give the reader an understanding of the system's development and use. A computerized network-level system for ordering priorities was developed and placed on city computers. A visual condition survey rating was made not only of the predominantly hot mix asphalt concrete street network but also of other associated facilities that the city must maintain, such as curbs and gutters, sidewalks, and crosspans. Several of the computer-sorting routines are summarized to show the types of reports that engineers and maintenance personnel will have available from PMS. PMS was developed in such a manner as to allow city of Arvada personnel to maintain and improve the system in-house in future years. PMS techniques can be developed and made useful at the local level to order the priorities among projects and make budget forecasts of needs.

The city of Arvada, Colorado, began a study with ARE Inc., of Austin, Texas, in January 1982, for the development of a pavement management system (PMS). The initial PMS was developed at the network level and used to evaluate the entire 320-mile city street network. This evaluation allowed the determination of present conditions and the estimation of rehabilitation needs and costs. The computer programs used to summarize the field information are stored in the city of Arvada computer system for future use. City of Arvada employees collected the field data under the training and direction of ARE Inc., engineers. This process will ensure Arvada of a PMS that the city can continue to use and update in the future. This study is the first phase of an overall project by the city to upgrade the street network to the best possible condition, considering current funding levels. The evaluation provided enough detail on the condition of pavements, sidewalks, crosspans, curbs, and gutters to allow the preparation of budgets and work programs for rehabilitation work (both contract and in-house work). A crosspan is a concrete trough that crosses a street, typically at intersections.

The study scope included the following:

1. Visual inspection of all network streets to estimate pavement condition, curb and gutter condition, sidewalk condition, visible drainage problems, and ride quality;
2. Analyses of the visual condition data; selection of necessary rehabilitation actions such as patching, overlays, removal and replacement, and recycling; and estimation of the cost of rehabilitation;
3. Development of a computerized system to handle PMS data;
4. Development of preventive maintenance plans--information to help schedule preventive maintenance and types of preventive maintenance that may be feasible and efficient; and
5. Selection of a tentative network rehabilitation program for 1982 and for future years.

DISCUSSIONS WITH CITY PERSONNEL

Before beginning the field inventory and in the initial stages of the inventory, meetings were held between ARE Inc., and the city of Arvada personnel to discuss study objectives, field items to be inventoried, and data collection methods. The discussions focused on the specific pavement distress types that are of concern in Arvada, such as alligator cracking, transverse cracking, potholes, surface wear, and rutting, and the degree of detail to be collected on each. Information needs on sidewalks, curbs and gutters, drainage, and crosspans were also reviewed.

To provide input to the selection of rehabilitation methods, preferred and nonpreferred types of rehabilitation were reviewed. For example, it was learned that emulsified rubberized chip seals had been commonly used, but slurry seals were not so common and heater scarification had not been used. Information such as this provided some guidelines and direction in the selection of rehabilitation alternatives for those streets that needed improvement. As a result of these meetings data collection procedures were established and forms and computer output formats were developed that were best suited to the needs of Arvada engineers and the goals of the study.

CONDITION SURVEY FORM

The condition survey form used for the evaluation and inventory on Arvada streets is presented in Figure 1. This form resulted after discussions among Arvada and ARE Inc., staff and the preparation and review of several draft forms. The form identifies (a) the street being surveyed and its limits, (b) section identification numbers, (c) specific pavement distresses (both extent and severity), (d) ride quality, and (e) the condition of the crosspans, curbs and gutters, and sidewalks. A detailed description of form use and the criteria for rating the extent and severity of each item was placed into a condition survey users' manual. No physical measurements were made, just subjective visual ratings. Ride quality was estimated by using the AASHO Road Test derived present serviceability rating procedure (1).

A training session was held to instruct city raters in the use of the survey form and to ensure consistency among the various raters. Trial ratings were held by Arvada personnel with supervision by an ARE Inc., engineer before the actual inventory. The condition surveys were conducted by several teams, comprising two surveyors working together. One team member was from the Arvada asphalt paving crew and the other team member worked on the portland cement concrete crew. Each team member rated those roadway elements with which he or she was most familiar (i.e., the asphalt concrete pavement or the portland cement concrete curbs, gutters, crosspans, and sidewalks).

Before the collection of the field data ARE Inc., selected data collection routes. With the routing information ARE Inc., was able to provide the city of Arvada with copies of the condition survey form (Figure 1) that had the street identification number, street name, section limits, and maintenance area already added by the computer.

To provide for an organized system for conducting the inventory, the data forms were separated into four sets, one for each maintenance area within the city. Each of the four trained crews collected data on approximately 25 percent of the streets. All of the information on the condition survey form was entered in the field except for the four columns previously added by the computer. Collection of the field data on the 320-mile system required approxi-

Figure 1. Visual condition survey data collection form.

PAVEMENT MANAGEMENT SYSTEM
NETWORK LEVEL INFORMATION

PAVEMENT CONDITION SURVEY
CITY OF ARVADA, COLORADO

DATE: ____________
SURVEYOR: ____________

BLOCK IDENTIFICATION	STREET NAME and LIMITS	MAINTENANCE AREA 1, 2, 3, or 4	SECTION Number Between 1 and 48	PRESENCE OF CUL-DE-SAC OR DEAD END	ALLIGATOR CRACKING				LONGITUDINAL CRACKING				TRANSVERSE CRACKING				FULL DEPTH PATCH			RIDE QUALITY	CROWN	SHOVING	RUTTING	PONDING	SURFACE WEAR	UTILITY TRENCH	POTHOLES	CG&S MONOLITHIC CONSTRUCTION # of Sides 0, 1, 2	DETACHED CURB & GUTTER # of Sides 0, 1, 2	DETACHED SIDEWALK # of Sides 0, 1, 2	CROSSPANS				CURB & GUTTER				SIDEWALK		
					1) 1-10% 2) 11-25 3) 26-50 4) >50				1) 1-10% 2) 11-25 3) 26-50 4) >50				1) 1-10% 2) 11-25 3) 26-50 4) >50				1) 1-10% 2) 11-25 3) 26-50 4) >50			1) Very Poor 2) Poor 3) Fair 4) Good 5) Very Good	1) Flat 2) Inverted 3) Excessive	1) Minor 2) Severe	1) Minor 2) Severe	1) Minor 2) Severe	1) Minor 2) Severe	1) Good 2) Poor	1) Few 2) Many				NUMBER OF CROSSPANS	Number to		1) Yes	1) 1-10% 2) 11-25 3) 26-50 4) >50			1) Yes	1) 1-10% 2) 11-25 3) 26-50 4) >50		
					N.O.	Slight <1/8"	Moderate <1/4"	Severe >1/4"	N.O.	% of Length Cracked	Amount Sealed %	Amount Unsealed or Reflected, %	N.O.	Spacing 1)>50 ft 2)>20 ft 3)<20 ft	Amount Sealed %	Amount Unsealed or Reflected, %	N.O.	Poor	Good													Seal	Replace	Drainage Problem	Seal	Replace	Mudjack	Drainage Problem	Replace	Mudjack	
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mately 2 weeks. Records show that Arvada Streets Division personnel logged 595 hours on the PMS project. This is approximately 15 weeks, or slightly less than 2 weeks for each of the eight raters. This also includes the time spent to learn the rating procedure, to rate some sections jointly, to test rate other sections independently, and to discuss the results to foster consistency. The ratings of the four teams were found to be acceptably consistent.

MASTER DATA FORM

A master data form, shown in Figure 2, was also developed. Data are added in the office from historical records, including section length, section width, cul-de-sac diameters, additional square footage, street classification, and average daily traffic (ADT). Data entry and verification for master form data and condition survey form data required approximately 300 hours of key punching. This includes ARE Inc., staff time to code in the routing information for the preprinted rating forms. The Arvada Traffic Division logged 90 hours to provide traffic data inputs and to administer the project, including meeting time. The Arvada Management Information Section totaled 287 hours on meetings, developing data entry and editing interface systems specific for the Arvada computer system, and working with ARE Inc., systems analysts.

INFORMATION PRESENTATION

The results of the study are contained in a series of computer reports, which can be produced by the Arvada computer. The list below contains the different reports that resulted from phase 1 work and shows the different orderings these reports can produce in order to highlight various aspects of interest.

1. Street identification and description listing;
2. Pavement condition survey data listing--(a) order by severity of patching, (b) order by severity of alligator cracking, (c) order by length of longitudinal and transverse cracking, and (d) in street sequence;
3. Crosspan, curb and gutter, and sidewalk data listing;
4. Miscellaneous pavement distresses listing order by severity of each of the seven miscellaneous distresses [(a) crown shape, (b) shoving, (c) rutting, (d) ponding, (e) surface wear repair, (f) utility trench, and (g) potholes] and by street sequence;

Figure 2. Master data file data collection form.

PAVEMENT MANAGEMENT SYSTEM **MASTER DATA FILE** **DATE:** ______________
NETWORK LEVEL INFORMATION **CITY OF ARVADA, COLORADO** **SURVEYOR:** ______________

BLOCK IDENTIFICATION	STREET NAME and LIMITS	MAINTENANCE AREA 1, 2, 3, or 4	SECTION Number Between 1 and 48	LENGTH ℄ to ℄ of Intersection, ft.	WIDTH Gutter Lip to Lip, ft.	CUL-DE-SAC Diameter of if Present, ft.	ADDITIONAL BLOCK SQUARE FOOTAGE	CLASSIFICATION	ADT (100'S)	YEAR OF CONSTRUCTION / LAST RECONSTRUCTION (19__)	YEAR OF LAST MAJOR REHABILITATION (19__)	YEAR OF LAST SEAL (19__)	NUMBER OF MANHOLES	NUMBER OF WATER VALVES	ANNUAL MAINTENANCE COSTS ($100 / Section)	FUTURE MDF INFORMATION
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5. Condition survey rating score listing--order by classification and then per classification by rating scores from worst to best and in street sequence;
6. Rehabilitation recommendation report--(a) order by classification and per classification order by the rehabilitation prioritization equation, (b) in street sequence, and (c) order by type of recommended rehabilitation;
7. Ride quality report--order by ride qualities from worst to better;
8. Crosspan report--order by severity, where a drainage problem is considered the most severe, replacement is the next severity, and sealing is of least severity; and
9. Curb and gutter and sidewalk report (each of these listings are broken up so that drainage problems appear first; monolithic construction appears first within drainage problems and detached construction appears second)--(a) order by replace curb and gutter, (b) order by mudjack curb and gutter, (c) order by seal curb and gutter, (d) order by replace sidewalk, and (e) order by mudjack sidewalk.

Reports 1 through 4 contain street descriptions and condition survey information collected by the city of Arvada in 1982. The ordering of each of these reports in street sequence gives the city easy access to the condition of any given block in the city. Street sequence means that the listing of streets is in alphabetical order with all the blocks that make up the street listed in order of occurrence. This ordering is based on block number rather than the alphabetical name of intersecting streets. This is helpful when trying to respond to complaints by citizens about given blocks.

PAVEMENT CONDITION RATING SCORE

The condition rating score for each pavement section is based on the extent and severity of each distress that was observed. The concept used is one that has been developed over a period of time based on pavement studies done by many different agencies and the experience of the ARE Inc., staff. The pavement rating score (PRS) is computed by deducting points from an initial score of 100 based on the extent and severity of the different types of pavement distress and ride quality by using the following equation:

$$PRS = 100 - D \qquad (1)$$

where D is the summation of the individual deduct values for the various distresses observed.

The deduct values were established based on the importance of each distress type to the overall performance of a pavement. Although the study team determined a unique set of deduct values for the

Arvada study, similar values for the Texas (2), Washington State (3), and Palo Alto, California, systems (4) were reviewed for guidance. Deduct values were set accordingly and are given in Tables 1-5. They are coded into the computer program for computing the condition rating scores.

REHABILITATION RECOMMENDATIONS AND COST ESTIMATES

Table 6 gives the comprehensive rehabilitation procedures and their costs, which were established to cover the pavement distress types that had been noted in the field inventory. The costs were based on information obtained from the 1981 construction estimates submitted to Arvada and industry information for new methods. City of Arvada and ARE Inc., engineers set the rehabilitation alternatives to be considered.

The recommended rehabilitation for each pavement section is based on the extent and severity of the existing distresses present and the amount of traffic carried by the facility. The rehabilitation selection procedure is a computerized decision tree similar in concept to that presented in the California Department of Transportation PMS (5). Such a selection process is acceptable for network-level estimates, where the engineer desires only a ball-

Table 1. Deduct values for alligator cracking for calculation of condition rating score.

Extent (percentage of area)	Severity: Slight AASHO Class 1	Moderate AASHO Class 2	Severe AASHO Class 3
1-10	3	7	10
11-25	5	13	16
26-50	10	19	23
50+	15	25	30

Table 2. Deduct values for longitudinal cracking for calculation of condition rating score.

Extent (percentage of length)	Severity[a]: Sealed	Unsealed or Reflected
1-10	1	2
11-25	2	5
26-50	4	8
50+	6	12

[a]If a street section has been rated to have both sealed and unsealed cracks, and if one type of crack condition (sealed or unsealed) is two or three levels of extent higher, then all cracks are assumed to be that type. For example, if +50 percent of the streets' cracks are unsealed and only 1 to 10 percent of the streets' cracks are sealed, then all cracks are assumed to be unsealed for estimating purposes.

Table 3. Deduct values for transverse cracking for calculation of condition rating score.

Extent, Spacing	Percentage of Length	Severity: Sealed	Unsealed or Reflected
Slight, greater than 50 ft	1-10	1	1
	11-25	1	2
	26-50	2	3
	50+	2	4
Moderate, 20 to 50 ft	1-10	2	4
	11-25	2	6
	26-50	3	8
	50+	5	11
Severe, less than 20 ft	1-10	3	6
	11-25	3	9
	26-50	4	12
	50+	7	15

Table 4. Deduct values for full depth patching for calculation of condition rating score.

Extent (percentage of area)	Severity: Good	Poor
1-10	0	5
11-25	5	10
26-50	10	20
50+	15	30

Table 5. Deduct values for different distresses for calculation of condition rating score.

Condition	Deduct Value	Condition	Deduct Value
Ride quality		Surface wear	
1 Very poor	30	0 None	0
2 Poor	15	1 Minor localized wear	2
3 Fair	3	2 Severe major stripping	7
4 Good	0		
5 Very good	0		
Crown		Utility trench	
0 OK	0	0 None	0
1 Flat, none	0	1 Good	0
2 Inverted	10	2 Poor	10
3 Excessive	0		
Shoving		Potholes, skin patches	
0 None	0	0 None	0
1 Limited	3	1 Few, 5	5
2 Severe	10	2 Many, 5	20
Rutting		Ponding	
0 None	0	0 None	0
1 Minor, 0.5 in.	5	1 Minor, 100SF	2
2 Severe, 0.5 in.	15	2 Severe, 100 SF	10

Table 6. Rehabilitation techniques.

Technique	Cost ($/ft^2)[a]
1. Remove and replace (reconstruction) an average thickness of 3 in. of asphaltic concrete and 5 in. of base	1.29
2. Repair distressed area with full depth patching; cover with fabric (geotextile) and a 1.5 in. asphalt concrete overlay	0.48
3. Repair distressed areas with full depth patching; cover with fabric (geotextile) and a 1 in. asphalt concrete overlay	0.38
4. Repair distressed areas with full depth patching; and a 1.5 in. asphalt concrete overlay	0.29
5. 1.5 in. asphalt concrete overlay	0.29
6. Heater scarification, add rejuvenating agent; and 1 in. new asphalt concrete material	0.33
7. Repair distressed areas with full depth patching and chip seal with CRS-2R, emulsified rubberized asphalt	0.10
8. Stress absorbing membrane interlayer (SAMI) and 1 in. asphalt concrete overlay	0.40
9. Repair distressed areas with full depth patching and slurry seal	0.11
10. Permanent full depth patching of distressed areas	1.73
11. Skin patching with fabric of distressed areas	0.17
12. No current rehabilitation required	0.00
13. Wedge cutting to improve drainage[b]	0.52
14. Milling[c]	1.04

[a]Costs are based on 1981 Arvada bid tabulations and current industry information plus a 10 percent contingency factor. These costs do not include the costs of full depth patching because patching is a function of the extent of potholes, poor patches, and alligator cracking; thus, techniques 4 and 5 have the same unit cost. Patching costs are added for techniques 2, 3, 4, 7, and 9 and its unit cost is equal to technique number 10.

[b]Wedge cutting is used in conjunction with techniques 2, 3, 4, 5, and 8 when street is noted to have poor drainage.

[c]Milling is used in conjunction with techniques 2, 3, 4, 5, and 8 where there is an excessive crown.

park estimate of needs rather than a precise selection of the final rehabilitation alternative. For example, a pavement that has severe alligator cracking is not likely to be improved by a slurry. A portion of the flow-chart logic by which the appropriate rehabilitation technique was chosen for each pavement section is shown in Figure 3. The specific rehabilitation techniques that correspond to the numbers on the flow chart are those listed in Table 6 along with the cost data used for computing the rehabilitation cost for each pavement section. The rehabilitation recommendations estimated to be needed by this flow-chart technique are for those actions that, based on engineering experience and judgment, have a minimum risk of premature distress. Less-expensive treatments may be possible once specific sites are studied in detail; however, for budget preparation the information provided served well.

SUMMARY OF CURRENT NETWORK CONDITION

Figure 4 contains the condition rating score information for principal collector streets in Arvada. Such histograms were prepared for each of the five street classifications and show the percentages of blocks that were in each range of the condition rating score. Sections are grouped into segments that have scores within a 10-unit interval. For example, as shown in Figure 4, for principal collector streets 25.1 percent of the blocks have a score between 90 and 100 (they are in excellent condition) and 2.4 percent of the streets have a condition score of between 0 and 10 (they are in very poor condition).

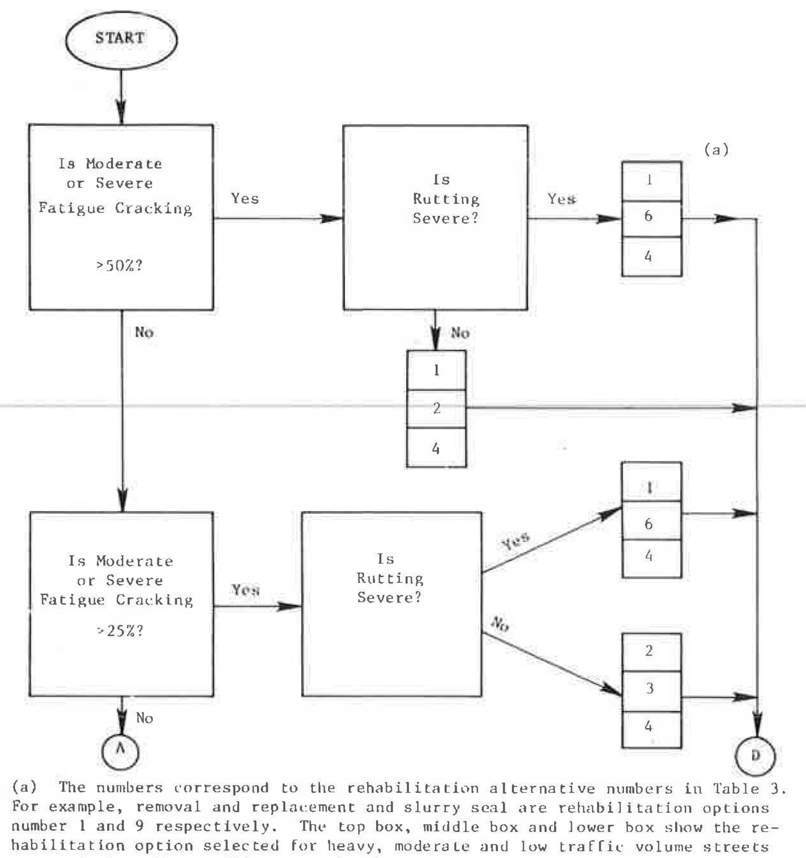
The condition score should be interpreted with care. The following generalities are reasonable considering how the value is calculated.

Condition Index	Street Condition
80	Better than average
50-80	Fair to good
50	Fair
30-50	Poor to fair
30	Very poor

One item to consider is what caused the street to have the low score--fatigue cracking, minor cracking, ride quality, or minor distresses. For example, a street may be rated 70 due to very poor ride quality; however, another street may be rated 75 due to 50 percent of its area being moderately fatigue cracked. On a local street the latter case is more important.

Another point to consider is a street that has a low score due to fatigue, longitudinal, and transverse cracking as compared with a street that has a similar score due to numerous miscellaneous distresses. The previous case is more important, and therefore, the engineer should compare the number of points deducted for various distresses when considering the absolute meaning of the pavement condition rating score values. The score is, more than any-

Figure 3. Rehabilitation techniques selection as function of existing pavement condition.



(a) The numbers correspond to the rehabilitation alternative numbers in Table 3. For example, removal and replacement and slurry seal are rehabilitation options number 1 and 9 respectively. The top box, middle box and lower box show the rehabilitation option selected for heavy, moderate and low traffic volume streets respectively. The traffic volume breakdown is:

Heavy traffic: Streets with ADT>10000 and streets with ADT>5000 if classification is Industrial or Principal Arterial

Moderate traffic: Streets with ADT from 5000 to 10000 and streets which are classified as Minor Arterials or Principal Collectors regardless of traffic.

Light traffic: Streets with ADT<5000 or streets which are classified as Minor Collectors or Locals regardless of traffic.

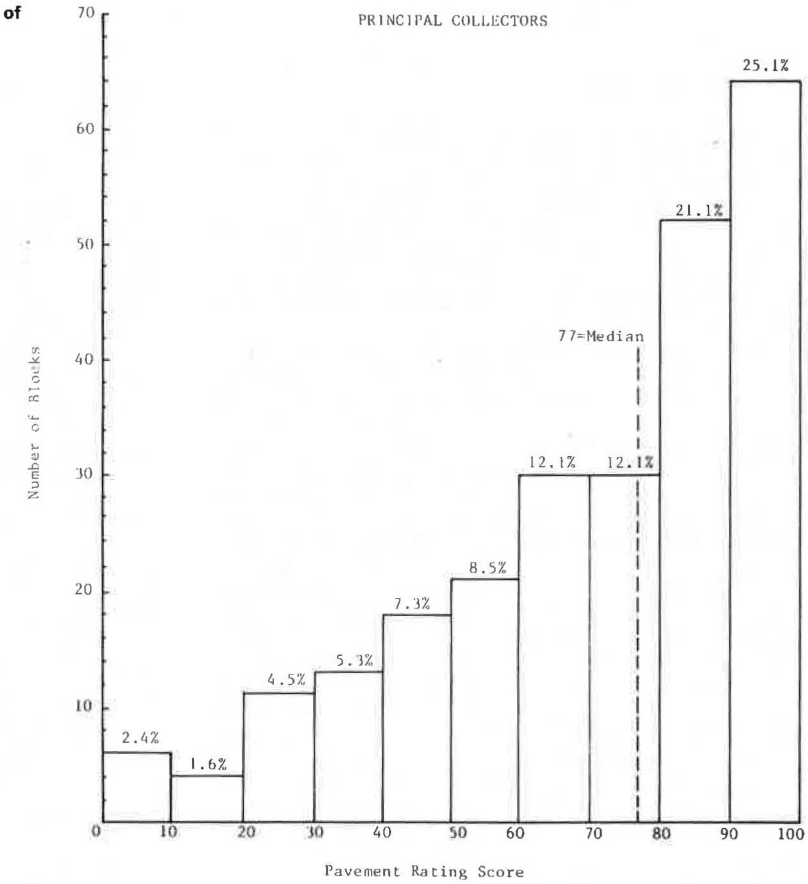


Figure 4. Frequency histogram showing current condition of principal collectors.

thing else, a relative indication of overall street condition, and was not used to select the rehabilitation alternative.

If a score of 50 is the breakpoint, then the following percentage of blocks in the individual Arvada systems were deficient in some respects and need some type of corrective action.

Classification	Percentage of Blocks with Scores <50	Median Score
Principal arterial	29.4	61
Minor arterial	39.8	57
Principal collector	21.1	77
Minor collector	22.8	77
Local	15.2	74

If arterial streets are lumped together and collector streets are lumped together, the data indicate that overall pavement damage is higher on the arterial streets than on the collector streets, and local streets have the smallest percentage of poor pavements. This is what might be expected because a substantial amount of pavement damage can be related to traffic and axle applications, and local streets have less heavy axle loadings than do collectors or arterials, respectively. Figure 4 shows an example of the relatively smooth condition distribution for principal collector streets. Streets that have ratings between 50 and 80 should be considered for corrective work as soon as possible to maintain their condition and protect the investment. Streets that have scores between 0 and 50 are all candidates for rehabilitation; however, the temptation to repair only these worst sections was avoided. The PMS procedure developed in Arvada determines the required rehabilitation alternative on every segment in the network, but the study team recognized that some way to establish the priority for these actions was needed.

REHABILITATION PRIORITIES AND PREVENTIVE MAINTENANCE

Generally the logical train of thought in selecting the streets that should receive rehabilitation first is to choose those that are currently in the worst condition (i.e., those that have the lowest scores). However, particular caution should be paid to the number of deduct points assigned for miscellaneous distresses, because these distresses are not as important as fatigue cracking, yet they may be causing a pavement to be rated very low. The deduct values given in Tables 1-5 were also modified to reduce the impact of ride quality on local streets, recog- nizing that they were excessive for that particular street classification where speeds are low.

Furthermore, in recognition of the need to select the most cost-effective projects, an equation was added to the computer program and it is used to produce the prioritized rehabilitation recommendation report (computer report 6). This equation is shown below:

$$P = (C/L)(\sqrt{ADT}/CI)(F) \qquad (2)$$

where

- P = prioritization value,
- C = cost of the specific rehabilitation alternative chosen as appropriate for a specific section,
- L = length of the pavement to be rehabilitated,
- ADT = average daily traffic,
- CI = pavement condition rating index score (as calculated with deduct values), and

F = 1.0 if street is not industrial or a bus route or
= 1.1 if street is an industrial classification or a bus route.

This equation was hypothesized. Until a data base of information could be developed, predictions of life-cycle pavement performance and costs could not be used with any confidence. The equation was developed empirically to contain the important factors of estimated repair cost, section size, traffic, current condition, and an indication of whether the street typically carries industrial or public transit vehicles.

An analysis of the prioritization equation indicates that, if the ratio of the rehabilitation cost to the length of pavement to be rehabilitated is high, then the first factor in the equation will be a larger number. If the amount of average daily traffic is high and the condition rating score is low, then the second term of the equation will be high. Finally, if the street is classified as industrial or has a bus route, an additional 10 percent will be added to the final priority value to increase the importance of these routes. Therefore, pavements that have highest values of P are pavements that, for most cases, will have the highest cost with respect to the length of pavement to be rehabilitated (indicating the need for a major improvement), have a high traffic level (thereby making it an important street), and be in a poor condition as indicated by the condition rating score. Inspection of the first year's network results proved that the equation had done an adequate job of establishing the priority order of the work load. As a more historical data base is gained this equation is a candidate for verification, improvement, or replacement.

IN-HOUSE WORK FOR CITY MAINTENANCE CREWS

Information was presented in the computer reports that can be used by the Arvada staff to estimate maintenance personnel and equipment needs. The information can be used as follows to plan each type of maintenance.

1. Concrete crew (sidewalks, crosspans, curbs, and gutters)--Computer reports 8 and 9 can be reviewed to determine the amount of curbs and gutters, crosspans, and sidewalks in poor condition. Work can be ordered by the probable types of repair. By using the cost per hour for the crew and the amount of work required, the personnel and equipment needs can be estimated.
2. Asphalt crew (ponding repair, shoved areas, localized fatigue, and cracking repair)--Computer reports 4A through 4G can be reviewed to determine the amount of maintenance required and to plan such maintenance.

Hours, crew sizes, equipment, and materials can be estimated based on these data.

FUTURE SYSTEM USE

In the future network data should be collected at the following frequency:

1. Arterial streets should be rated annually,
2. Collector streets should be rated every second year, and
3. Local streets should be rated every third year.

This frequency will save Arvada money and data collection time yet will allow Arvada engineers to become aware of street changes with enough time to plan future rehabilitations. Another reason for not rating the entire network annually is that the amount of money spent on pavement rehabilitation is such that, at present, only 10 percent of the estimated needs are being budgeted annually and there is already a long list of needs from this initial study.

Whenever a street condition is changed or improved through minor maintenance such as a seal coat or through major rehabilitation, the computer record will be updated to note this change rather than waiting for the survey crews to pick it up at a later rating. An update of the records as improvements are made to the network, probably at the end of each construction season, would be more appropriate. This will allow for a constantly updated status report of the network to be available as needed by the city council. It was also recommended that continued study be made of the rehabilitation selection criteria, deduct values assigned for specific distresses, and other reporting information from the computer programs.

City engineers are planning to add data on the year of last sealing, last overlay, or last reconstruction to the master data file. This information can also be used to assign importance and priority to the work. When retrieved from the computer data base this information will provide valuable insight into the performance of various maintenance and rehabilitation treatments. The master data file is flexible and can be expanded in the future to add other specific information on maintenance activities.

SUMMARY AND CONCLUSIONS

The Arvada PMS system outlined here was developed for $33,000 by the consultant; however, a great deal of cooperative effort was input by city of Arvada personnel who wanted an in-house self-sustaining system. The total hours spent were

Traffic division--91 hours for traffic and project administration,
Streets division--595 hours for basic field data collection and meetings,
Management information section--287 hours for programming and meetings, and
Engineering division--80 hours for street dimension data and meetings.

With fully loaded labor rates this represents a cost of approximately $14,000 for work by employees of the city of Arvada. In addition the system is a product of previous research and developments by the consultant.

Conclusions from the study are as follows.

1. A substantial amount of information can be gained through a simple visual inspection of a street network.
2. The network results were generally consistent with the expectations and thoughts of Arvada engineers concerning areas of need.
3. The prioritization equation produced a satisfactory first-cut ranking of projects for rehabilitation. It could be improved once some historical data are developed.
4. Long-term benefits are expected by the city because its personnel were involved in all phases of PMS development and implementation.
5. The network study demonstrated the need for future street rehabilitation and maintenance funds by providing information that quantified the overall network condition.
6. Close cooperation between the consultant and the agency is required to foster the production of a

working PMS that satisfies agency needs and expectations.

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Implementation of Idaho's Pavement Management System

M.A. KARAN, K. LONGENECKER, A. STANLEY, AND RALPH HAAS

The Idaho Department of Transportation (ITD) decided in 1979 to begin implementation of a comprehensive pavement management system, initially for their Interstate network and then for their state highway network. Two major phases have been involved in this implementation: (a) inventory of the Interstate network, documentation of the pavement performance management information system (PPMIS) program that was obtained from Utah, modifications and improvements to PPMIS, and implementation of PPMIS to the Interstate mileage and (b) further modification of the PPMIS program based on the findings of phase 1, expansion of the system to better suit ITD's needs, and verification of the new version of the PPMIS programs by using inventory data gathered in phase 1. The two phases of the project are described, including the original PPMIS program, modifications and extensions made, field inventory procedures used to collect data, implementation procedures employed, and assessment of the results. Phase 1 indicated that the PPMIS was a technically sound tool that could form the basis for Idaho's pavement management system. However, further improvements and modifications were necessary to fine-tune the system for Idaho conditions. These modifications and extensions were accomplished in phase 2. The Idaho version of PPMIS has been installed on ITD's computer facilities and is now operational. ITD personnel have used it effectively in making decisions regarding the management of the pavement network in the state.

Idaho's highway network includes about 612 miles of Interstate highways plus about 5,000 miles of paved or oiled state highways. Considering the size and population of the state, this represents a substantial investment and presents a major management challenge, especially when funds are scarce and the demand is increasing, as has been the case in the last decade or so.

Under the circumstances that currently exist the limited rehabilitation and maintenance dollars have to be spent in such a way as to ensure the maximum return to the state and to the taxpayers. The concept of pavement management provides an effective tool to achieve this objective. The Idaho Transportation Department (ITD) thought that the benefits of implementing a pavement management system in the state were worthwhile and thus began to plan actively toward their goal in 1977-1978.

The first step was a review of what was available. Utah's Pavement Performance Management Information System (PPMIS) was chosen because it offered a sound basis on which to develop and implement a system for Idaho. In addition to the features of PPMIS, because Idaho and Utah were similar in many respects, the regional and technical aspects of PPMIS were thought to be reasonably applicable to Idaho conditions.

The PPMIS program was acquired from Utah in 1978 and made operational on the ITD computer. Several difficulties arose, however, in applying the program. First, PPMIS was designed for Utah conditions and some of these are not directly transferable to Idaho without further calibration. Second, some of the models either needed improvement or modification to apply to Idaho. For these and several other reasons, the initial results of PPMIS were questionable from the standpoints of practicality and reasonableness.

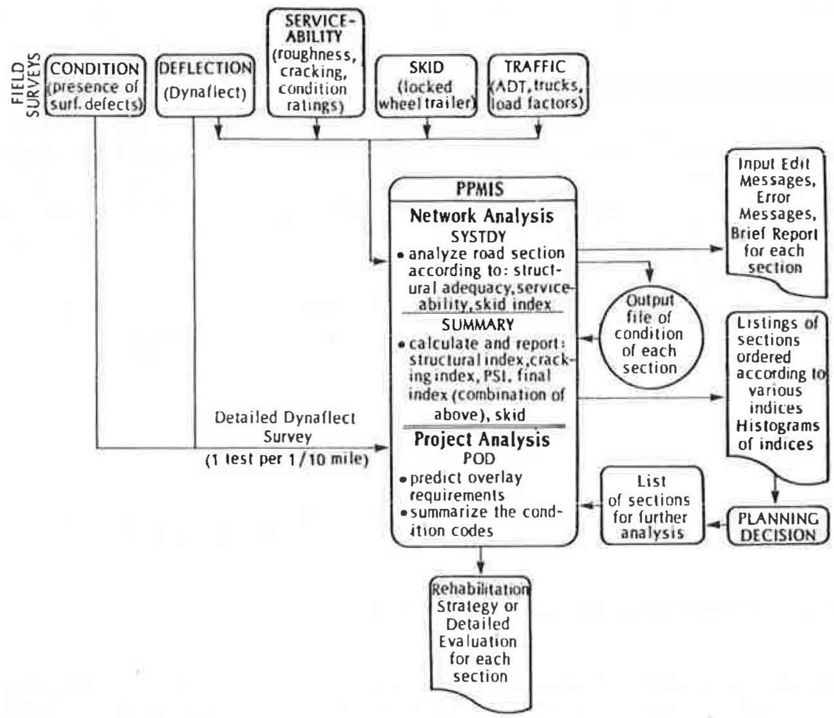
In 1979 ITD decided to accelerate its pavement management system development and implementation. The project was split into two phases because of fiscal planning considerations, and the first contract was awarded in August 1979 and completed in June 1980. The basic terms of reference for phase 1 were to conduct field inventory data collection on the Interstate mileage, document the PPMIS program, carry out certain modifications and improvements to PPMIS, and initiate the implementation of PPMIS to the Interstate mileage.

The second contract was initiated in October 1980 and completed in June 1981. In general, the terms of reference for phase 2 were to further modify the PPMIS program based on the findings of phase 1, expand the system to better suit ITD's needs, verify the resulting new version of the PPMIS programs by using inventory data gathered in phase 1, install the system on the ITD computer in Boise, and provide the required instructions.

The purpose of the paper is to describe the two phases of the overall project, specifically the following:

1. The original PPMIS program and procedures used for documentation and evaluation,
2. The modifications and extensions made to the original PPMIS program to suit ITD's needs and requirements,
3. The field inventory procedures used to collect data on the Interstate mileage,
4. The trial implementation procedures employed and assessment techniques used, and
5. The evaluation of the results and benefits derived from the system.

Figure 1. Overview of PPMIS program structure.



PHASE 1: DOCUMENTATION, MODIFICATION, AND APPLICATION OF INITIAL IDAHO PPMIS

The initial version of the Idaho PPMIS comprises three main computer programs, SYSTDY, SUMMARY, and POD. In terms of the general philosophy of pavement management these programs cover the two basic levels that should be included in a pavement management system--the network and project levels.

The general structure of PPMIS is presented in Figure 1. Programs SYSTDY and SUMMARY are grouped under network analysis (i.e., the network level of management) and program POD is under project analysis (i.e., the project level).

Analysis at the network level begins with program SYSTDY, which edits and processes the various types of information gathered in field surveys of the pavement network. SYSTDY transforms and organizes the field data into a highly summarized format for each pavement section in the network and also produces a brief printed report for each section.

The summarized data are subsequently used as input to program SUMMARY. This program transforms the summarized measurements into indices [i.e., structural adequacy, present serviceability (PSI), and distress indices]. These various indices are combined into a final, overall index value for each section. The sections analyzed are listed in ascending order of the various index parameters (including skid numbers) considered, and histograms are produced for each list.

The final report is a ranked list of sections in the network that can be used for programming improvements. In addition to the final index list, histograms are also produced of summaries for the various parameters processed in this program. Note that, although these final rankings can be used for priorities, an economic analysis of alternative improvement types and timings for the various sections might well result in a substantially different priority list. Consequently, the ranked list may be viewed more as the first step toward an eventual priority ordering based on economic optimality in that it represents a list of needs.

From the ranked list produced by program SUMMARY, a number of sections can be identified for more detailed project analysis. More extensive field testing can then be performed on these sections. This information is subsequently used by program POD in a more comprehensive analysis to provide a detailed evaluation and, if desired, a recommended rehabilitation strategy for each section. Each one of these three main programs (SYSTDY, SUMMARY, and POD) contains a series of subroutines (1). Sample outputs are subsequently provided in this paper.

Modifications to Initial Idaho PPMIS

The initial version of the Idaho PPMIS was tested by ITD personnel on portions of the Idaho Interstate Highway System and, as a result, the need for modifications was recognized first by them. Hence, the primary motivation for the modifications came from the need to adapt the initial PPMIS to conditions in Idaho.

During the subsequent documentation of this initial system in phase 1 a number of inconsistencies, errors, and shortcomings were found. These resulted in numerous modifications and improvements to the system. In some cases modifications were made to improve the operation of the program or to clarify the results presented. The objectives were to make the overall system as efficient as possible and to make the output as informative as possible.

The modifications made to the initial Idaho PPMIS can be classified into the following basic categories:

1. Corrections,
2. Technological adaptations,
3. Regional adaptations, and
4. Improvements.

Corrections include changes to the program structure because of illogical operation sequencing, extraneous or extra required steps in the logic, erroneous or inaccurate Fortran coding, and unreasonable parameter values. Technological adaptations include all changes to input format, variable definition, program structure, and mathematical models made necessary by the use of different equipment for the field inventory. Regional adaptations include the respecifications of limits, constraints, parameter values, and the methods of calculating certain parameters to reflect Idaho conditions and practices more accurately. Improvements are primarily in terms of the changes to print routines to produce reports that are easier to follow and less open to misinterpretation.

These modifications (1) are listed in Table 1 under the categories for each main program in PPMIS. The modifications conducted in phase 1 of the project resulted in the modified Idaho PPMIS that was then applied to the flexible pavements of the state's Interstate network.

Field Inventory Measurements

The modified Idaho PPMIS program requires the following four basic types of inventory information:

1. Structural capacity,
2. Surface condition,
3. Roughness, and
4. Skid resistance.

Structural capacity, which is based on deflection measurements taken on the surface of the pavement structure, is used in PPMIS to incorporate engineering input into the analysis. Surface condition information is used indirectly by the analyst or engineer to determine (a) where an accelerated loss of serviceability will most likely occur and (b) maintenance needs. Roughness information, which is the basis for determining the serviceability level of a pavement, is used in PPMIS to take into account the road user's response in addition to engineering measures such as structural capacity. Skid resistance information is used in PPMIS to identify pavement sections that have the potential for skid problems. The information is not analyzed in PPMIS. It is simply listed as supplementary information should criteria for minimum skid resistance be used.

In phase 1 the field measurements performed in collecting inventory information on the 612 miles of Interstate highway network included the following:

1. Section identification,
2. Deflection measurements,
3. Roughness measurements,
4. Surface condition survey, and
5. Skid measurements.

Section Identification

In the first task of the field work 612 miles of Interstate highways were broken into sections by ITD personnel. The existing milepost and coded segment (MACS) (2) system, past experience, contract lengths, traffic volumes, pavement type, age and thickness, and geometric characteristics were taken into consideration in the section identification process.

Sections were identified (by using the existing milepost system) separately in each direction of travel on four-lane divided Interstate highways. No directional split was considered on two-lane undivided portions of the Interstate system. In light of these considerations and through extensive field studies 376 sections were identified on the 612 miles of the Interstate highway system in the state.

Deflection Survey

Three Dynaflect deflection tests per mile, with a minimum of three tests per section, was the initial sampling frequency proposed, as opposed to one test per mile required by the original Utah PPMIS. The specific requirements of the modified Idaho PPMIS, size of the network, availability of equipment, personnel, time limitations, and statistical validity considerations were taken into account in designing this inventory scheme.

This sampling frequency was initially implemented for the Interstate highways in district 3. Statistical analyses were conducted after one week of testing to determine the significance of sampling at three tests per mile with a spacing of 0.1 mile between tests. The third measurement of the sampling frequency of three tests per mile did not make a significant contribution to the mean deflection of the section. Statistical tests conducted for different section lengths resulted in the same conclusion.

The short sections that have lengths of less than a mile were exceptions to this conclusion. Three tests were necessary for these sections as required by the original Utah PPMIS. Therefore, the third test was eliminated and the remainder of the Interstate highway mileage in the state was surveyed by taking two tests per mile with a minimum of three tests per section.

Two Dynaflect units were employed in the survey. Measurements of rut depth and air and pavement temperature were also taken during the Dynaflect survey. A Dynaflect correlation study was also conducted as a part of phase 1 deflection inventory program of Interstate highways in Idaho to determine

Table 1. Modifications to initial Idaho PPMIS.

Modification	Program SYSTDY	Program SUMMARY	Program POD
Corrections	Lane distribution factors, dynaflect conversion, order of calculations in structural analysis, adjustment of design period for environment, calculation of remaining years of structural adequacy, adjustment of roadmeter readings, calculation of total and average cracking plus patching, calculation of remaining years of acceptable serviceability, speed correction of skid indices	Calculation of cracking index	Dynaflect device conversion, calculation of annual traffic load, calculation of overlay thicknesses, testing for outliers
Technological adaptations	Serviceability data input, decimal conversion of serviceability input data, check for valid measurement device, conversion parameters for roadmeter data, conversion of roadmeter data, calculation of PSI	None	None
Regional adaptions	Minimum traffic growth rate limit, load equivalency factors	Calibration of cracking index model, calculation of final index, adjustments to final index	None
Improvements	Dynaflect readings and summary, serviceability summary, skidmeter summary	Skid index histogram	None

the consistency of the Dynaflect units employed in the survey. No significant difference was found between the Dynaflect units when the average deflections for sections (both flexible and concrete) were considered. Hence, no conversion was necessary for the raw data because PPMIS used average values for the sections. Details of the Dynaflect correlation study can be found elsewhere (3).

Roughness and Surface Condition Surveys

The automatic road analyzer (ARAN) unit (4-6) was employed to measure pavement roughness and to conduct a condition survey on the Interstate highway network in Idaho. The ARAN unit, which is housed in a van, measures roughness by use of an accelerometer mounted on the rear-axle housing of the vehicle. It can measure roughness (among many other pavement parameters) at 10-, 20-, 50-, and 100-m intervals at speeds up to 50 mph. In this project, however, roughness on the Interstate highway sections was measured on the traveling lane, at 50-m intervals and 30 mph operating speeds.

The condition survey that was conducted concurrently with the roughness survey was the main reason for using a relatively low speed. Roughness levels were measured and recorded automatically on a magnetic tape (in digital form) at 50-m intervals. The condition survey was conducted by a specially trained rater who was seated in the right front seat of the unit. The rater entered special codes (which were developed specifically for Idaho conditions and PPMIS program) into the system for the various distress parameters observed while traveling over the pavement by pressing the appropriate sequence of buttons on a hand-held keyboard.

A present serviceability index (PSI)-roughness correlation study was also conducted as a part of the phase 1 roughness survey program. A wide range of rural road sections (30 flexible and 12 rigid) in district 3 were selected for analysis and two, four-member panels of representative road users in the state were formed and trained in rating pavement serviceability.

The rating procedures used and extensive statistical analyses conducted, which are described in detail elsewhere (7), resulted in a reasonably good model for flexible pavements for predicting PSIs directly from ARAN roughness measurements. Although the model was also reasonable for rigid pavements, recommendations were made for further study in this area.

Skid Measurements

Skid measurements on the Interstate highway mileage were taken by ITD personnel by using their locked-wheel skid trailer. One test per mile with a minimum of three tests per section was taken on the traveling lane, at an operating speed of 40 mph. Test location, friction force, and air and pavement temperatures were recorded manually on special forms that were then used to determine and adjust the skid numbers to be used in PPMIS.

Application and Evaluation of Modified Idaho PPMIS

In order to test the reasonableness of the modified Idaho PPMIS the field inventory data collected on the Interstate system were used as input for a number of runs of the modified Idaho PPMIS program. The purpose of this application was to provide a basis for assessing the reasonableness of the modified Idaho PPMIS for conditions in Idaho.

A large-scale application was thought to be more meaningful in that it would enable the project team to test the system over a large sample size that covers a wide range of conditions that exist in Idaho. The pavement sections considered in this large-scale application were located in five out of the six ITD districts in the state.

Programs SYSTDY and SUMMARY were run for all the flexible pavement sections of the Interstate highways and the results were evaluated in the office. Next, extensive field inspections were performed with ITD officials for verification purposes. This was followed by meetings with the district staff who were most familiar with the pavement sections in their jurisdictions.

Similar to network analysis programs (i.e., SYSTDY and SUMMARY), project analysis program (POD) was run to analyze the detailed deflection and surface condition data collected on two sections in district 2. The results were then evaluated by the project team and ITD personnel who were most familiar with these projects. The results indicated that the PPMIS was a technically sound tool that could form the basis for Idaho's pavement management system; however, further improvements and modifications were necessary to fine-tune the system for Idaho conditions.

Phase 1 of the project then concluded with a number of recommendations for the calibration, improvement, extension, and further implementation of the system. These recommendations, which have been described in detail elsewhere (1), are summarized below.

Recommendations for network management are as follows:

1. Identify section (homogenity, length);
2. Determine network improvement needs;
3. Incorporate economic analysis;
4. Optimize network improvement sections, strategies, and timing; and
5. Conduct budget-level analysis and financial planning.

Recommendations for pavement behavior models are as follows:

1. Estimate structural condition,
2. Predict structural life,
3. Estimate present serviceability (PSI),
4. Estimate acceptable serviceability life,
5. Calculate cracking index, and
6. Conduct rigid pavement analysis.

Recommendations for program operation, structure, and logic are as follows:

1. Input structure,
2. Program structure and logic, and
3. Present output.

Recommendations for field inventory procedures are as follows:

1. Conduct deflection surveys,
2. Conduct roughness measurements,
3. Conduct condition surveys,
4. Conduct geometric surveys, and
5. Conduct skid surveys.

Recommendations for implementation and operation of the system are as follows:

1. Implement PPMIS for the state highway system,
2. Detail management responsibilities, and
3. Update inventory.

PHASE 2: FURTHER MODIFICATION AND EXTENSION OF MODIFIED IDAHO PPMIS

The main objective of phase 2 of the project was to implement phase 1 recommendations to make PPMIS more compatible with the conditions that exist in Idaho. Specifically, the objectives were as follows:

1. Implement phase 1 recommendations in terms of modifications to the existing PPMIS to calibrate and adjust models for Idaho conditions,
2. Expand PPMIS's capabilities to analyze rigid pavements and to include graphical presentations,
3. Correlate ITD's new Cox roadmeter with present serviceability rating (PSR) for both flexible and rigid pavements,
4. Apply new PPMIS programs to phase 1 field inventory data and assess reasonableness of new programs,
5. Install and make operational new PPMIS programs on ITD computing facility and provide reports and manuals, and
6. Conduct training course for ITD personnel in the operation of the system.

Developments for PPMIS Modification

The following developments, completed in phase 2 of the project, have either been implemented as modifications to PPMIS or will be implemented at a later date as determined by ITD.

Flexible Pavement Structural Condition Models

In phase 1 the models developed in Utah for evaluating the structural condition of flexible pavements were reviewed and found to be questionable for conditions in Idaho. Consequently, a more general model was developed by using deflections measured on the Idaho Interstate highway system in phase 1 and incorporating the spreadability and deflection basin area. The new models produced more reasonable results than did the Utah models.

Flexible Pavement Structural Life Models

Experience gained in phase 1 was used to develop a new environmental adjustment model for analyzing flexible pavements in Idaho. The models that were in PPMIS had been developed specifically for conditions in Utah and were found to be questionable for conditions in Idaho.

Structural Index Model

A new model was developed for calculating the structural index in program SUMMARY to make it compatible with the evaluation indices.

Serviceability Life Models

The original regression equations developed in Utah for predicting the remaining number of years of acceptable serviceability were assessed as to their applicability to conditions in Idaho and were found to be reasonable.

Flexible Pavement Cracking Index Model

A new cracking index model was developed to replace the original (modified for Idaho) model in PPMIS. This model was based on ITD's new crack rating system.

PSI-Cox Roadmeter Roughness Correlation

The correlation developed in phase 1 (7) was replaced with two new separate models for flexible and rigid pavements. Meyer and Karan (8) provide details of this correlation study.

Traffic Load Calculations

A new table of load equivalency factors was incorporated into the PPMIS program to facilitate updating as new information became available. After subsequent tests and several modifications to the calculation procedures the results were found to be questionable and, thus, the concept of using truck factors was applied as a temporary measure.

Input Structure

The input structure of PPMIS was changed significantly to accommodate certain modifications and extensions such as analysis of portland cement concrete (PCC) pavements. Users manuals (7,8) provide detailed descriptions of these changes.

Program Structure

Modifications in the structure of the PPMIS programs were necessary to facilitate the incorporation of several new analysis models such as those for PCC pavements, adjustments, and extensions. Some programs were rewritten, and the sequence of operations on some other programs was changed to increase efficiency.

Presentation of Output

Significant changes were made to PPMIS outputs to accommodate the modifications and extensions made to the analysis techniques (i.e., PCC pavements). One of the major changes is that PPMIS no longer requires all four of the major types of field data (i.e., deflection, roughness, surface condition, and skid) to produce a report. Figure 2, for example, shows SYSTDY output for a section where deflection and skid data are not present. Figure 3 is another example with all data types present. This feature of the program was extended to the SUMMARY program, as shown in Figures 4 and 5. The graphical presentation shown in Figures 4 and 5 was added to the program in addition to the tabular outputs that were available in the original PPMIS. Figure 6 shows sample tabular output of SUMMARY program.

As with the SYSTDY and SUMMARY programs, the output of POD was reconstructed to produce a more meaningful report. Tabular presentations were changed to facilitate ease of use. Summary tables were also added. A major addition to the POD output is the capability to report the tabulated information in a pictorial format. Figure 7 presents a tabular POD output and Figure 8 presents a graphical POD output.

Developments for Expansion of PPMIS Capabilities

The following developments were completed in phase 2 of the project and have either been incorporated into the PPMIS program or may be considered for implementation at a later data as determined by ITD.

Rigid Pavement Structural Analysis

A model based on Dynaflect deflection for evaluating the structural life and overlay requirements of rigid pavements was developed and calibrated for conditions in Idaho. The model covers only asphalt overlays of rigid pavements. A model for designing concrete overlays for rigid pavements was not included in PPMIS because of the funding limitations. The model selected for implementation in PPMIS was

Figure 2. Sample SYSTDY output for selected data types.

```
---------------------------------------------------------------------------------------------------------------------------------
* IDAHO PPMIS (2)  MACS: 01570B N 111.859  DIST: 6  COUNTY: BONNEVILLE      19  CITY:                          0 FAI-15     7/ 1/81   *
* IDAHO ROUTE:  15  BINGHAM CO LINE                MILEPOST: 111.859    IDAHO FALLS SUL             MILEPOST: 118.000  LENGTH:    6.17*
* MATERIAL: PLANT MIX SEAL COAT  BIT. SF. (PMSC)  MAINTENANCE SHED:          WIDTH: 12.00                                            *
* YEARLY INCREASE IN 18KIP LOADS:    4.0%      PRESENT 18KIP LOADS:        0.23569E+06      T. S. I. 2.5                            *
*--------------------------------------------------------------------------------------------------------------------------------*
*                                                                                                                                *
*                                                                                                                                *
*                                          NO  DYNAFLECT  READINGS  ARE  RECORDED                                                *
*                                                                                                                                *
*                                                                                                                                *
*--------------------------------------------------------------------------------------------------------------------------------*
*      ***CONDITION SUMMARY***          *                 ***SERVICEABILITY SUMMARY***                  *                        *
* AVERAGE CONDITIONS                    * DATE    9/26/79                                               *                        *
* SURFACE WEAR  5.0   POPOUTS      N/A  * TEST        SERV LIFE     100 +                               *                        *
* WEATHERING     5.0   UNIFORMITY  N/A  *  NO.   PSI    REMAIN    %      I                          71  *   NO  SKID  DATA  ARE  *
* RUT DEPTH      0.0                    *   1   3.9       9           80 +                          **  *                        *
*                                       *   2   4.1      11      0       I                          **  *                        *
* AVG. CRACKING AND PATCHING            *   3   4.2      14      F   60 +                           **  *                        *
*  ( PER 1000. SQ. FT.)                 *   4   4.2      14              I                          **  *                        *
* TRANSVERSE       (FT.)       4.       *   5   4.2      13      T   40 +                           **  *                        *
* LONGITUDINAL     (FT.)       3.       *   6   4.0      10      E       I                    14 14 **  *                        *
* MAP            (SQ.FT.)      0.       *   7   4.3      14      S   20 +                     ** ** **  *                        *
* ALLIGATOR      (SQ.FT.)     N/A       * MEAN 4.1     12.1      T       I                    ** ** **  *                        *
* PATCH-SKIN     (SQ.FT.)      0.       * S.D.  0.1      2.1     S    0 +--+--+--+--+--+--+--+--+--+--+--+  *                    *
* PATCH-DEEP     (SQ.FT.)      0.       *                           0  1  2  3  4  5  6  7  8  9 10 >10 *                        *
* AVG CONDITION OF TRANS,LONG CRACKS    *                                                               *                        *
* OPENING         5.0    ABRASION  N/A  *                         REMAINING STRUCTURAL LIFE (YEARS)     *                        *
* MULTIPLICITY    5.0                   *                         BASED ON PSI                          *                        *
*                                       *                                                               *                        *
---------------------------------------------------------------------------------------------------------------------------------
```

Figure 3. Sample SYSTDY output for rigid pavement section.

```
---------------------------------------------------------------------------------------------------------------------------------
* IDAHO-PPMIS (2)  MACS: 09070A E   6.146  DIST: 1  COUNTY: KOOTENAI         55  CITY:                          0 FAI-90     7/ 1/81   *
* IDAHO ROUTE:  90  POST FALLS ECL                 MILEPOST:   6.146    COEUR D'ALENE WUL           MILEPOST:   9.739  LENGTH:    3.59*
* MATERIAL: CONCRETE     (CONC)                      MAINTENANCE SHED:          WIDTH: 12.00                                         *
* YEARLY INCREASE IN 18KIP LOADS:    3.9%      PRESENT 18KIP LOADS:        0.43305E+06      T. S. I. 2.5                            *
*--------------------------------------------------------------------------------------------------------------------------------*
*                                              ***DYNAFLECT READINGS AND SUMMARY***                                              *
*  DATE    9/20/79  HR 11 MIN 51     TEMPERATURES: AIR  78.1 SURFACE   0.0 PAVEMENT    N/A                                       *
*  WHEEL PATH  OSWP         LANE EB-1     LAST REVISION 092079                                                                   *
*                                                                                                   100 +                        *
*  TEST        SNSR  SNSR  SNSR  SNSR  SNSR    SCI  BCI     CORNER  OVER  REMAINING SERVICE  %        I                        *
*   NO.          1     2     3     4     5                  SNSR 1  -LAY  LIFE (STRUCTURAL)       80 +                        *
*                                                                          18K LOADS YEARS    O        I                        *
*     1        0.33  0.28  0.25  0.20  0.15   0.05 0.05      N/A     N/A  3.002E+06     10    F   60 +           50              *
*     2        0.28  0.25  0.23  0.18  0.15   0.03 0.03      N/A     N/A  3.002E+06     10             I           **              *
*     3        0.39  0.35  0.31  0.26  0.21   0.04 0.05      N/A     N/A  1.832E+06      7    T   40 +           **              *
*     4        0.41  0.37  0.34  0.28  0.22   0.04 0.06      N/A     N/A  1.690E+06      6    E      I12 12 12 12 **              *
*     5        0.28  0.25  0.22  0.18  0.14   0.03 0.04      N/A     N/A  3.278E+06     10    S   20 +** ** ** ** **              *
*     6        0.32  0.28  0.24  0.19  0.14   0.04 0.05      N/A     N/A  3.278E+06     10    T      I** ** ** ** **              *
*     7        0.65  0.60  0.52  0.41  0.31   0.05 0.10      N/A     N/A  7.614E+05      3    S    0 +--+--+--+--+--+--+--+--+--+--+--+ *
*     8        0.85  0.80  0.71  0.58  0.45   0.05 0.13      N/A     N/A  2.778E+05      1          0  2  4  6  8 10 12 14 16 18 20 >20 *
*                                                                                                                                *
* MEAN         0.44  0.40  0.35  0.29  0.22   0.04 0.06      N/A     N/A  2.140E+06    7.1         REMAINING STRUCTURAL LIFE (YEARS) *
* STD DEV.     0.20  0.20  0.17  0.14  0.11   0.01 0.03      N/A     N/A  1.180E+06    3.6                                           *
* MEAN - SD    0.23  0.20  0.18  0.14  0.11   0.03 0.03      N/A     N/A  9.605E+05    3.6                                           *
* MEAN + SD    0.64  0.60  0.53  0.43  0.33   0.05 0.10      N/A     N/A  3.320E+06   10.7                                           *
* OUTLIERS ******************  0.58  0.45                                                                                        *
*                                                                                                                                *
*--------------------------------------------------------------------------------------------------------------------------------*
*      ***CONDITION SUMMARY***          *                 ***SERVICEABILITY SUMMARY***                  *  ***SKIDMETER SUMMARY*** *
* AVERAGE CONDITIONS                    * DATE    9/20/79                                               * DATA IS SPEED-COMPENSATED *
* SURFACE WEAR  5.0   POPOUTS      N/A  * TEST        SERV LIFE     100 +                               *   TEST NO.    SKID INDEX *
* WEATHERING     3.7   UNIFORMITY  N/A  *  NO.   PSI    REMAIN    %      I           75               *      1            38     *
* RUT DEPTH      0.0                    *   1   3.8       7           80 +           **               *      2            39     *
*                                       *   2   3.7       7      0       I           **               *      3            38     *
* AVG. CRACKING AND PATCHING            *   3   3.9       8      F   60 +            **               *      4            38     *
*  ( PER 1000. SQ. FT.)                 *   4   3.6       6              I           **               *    MEAN          38.3    *
* TRANSVERSE       (FT.)      33.       * MEAN 3.8      7.0      T   40 +        25 **               *    S.D.           0.5    *
* LONGITUDINAL     (FT.)       0.       * S.D.  0.1      0.8     E       I        ** **               *                          *
* MAP            (SQ.FT.)      0.       *                        S   20 +        ** **               *                          *
* ALLIGATOR      (SQ.FT.)     N/A       *                        T       I        ** **               *                          *
* PATCH-SKIN     (SQ.FT.)      0.       *                        S    0 +--+--+--+--+--+--+--+--+--+--+--+ *                      *
* PATCH-DEEP     (SQ.FT.)      0.       *                           0  2  4  6  8 10 12 14 16 18 20 >20 *                        *
* AVG CONDITION OF TRANS,LONG CRACKS    *                                                               *                        *
* OPENING         5.0    ABRASION  N/A  *                         REMAINING STRUCTURAL LIFE (YEARS)     *                        *
* MULTIPLICITY    3.0                   *                         BASED ON PSI                          *                        *
*                                       *                                                               *                        *
---------------------------------------------------------------------------------------------------------------------------------
```

Figure 4. Sample summary sheet showing distribution of road miles by all evaluation indices.

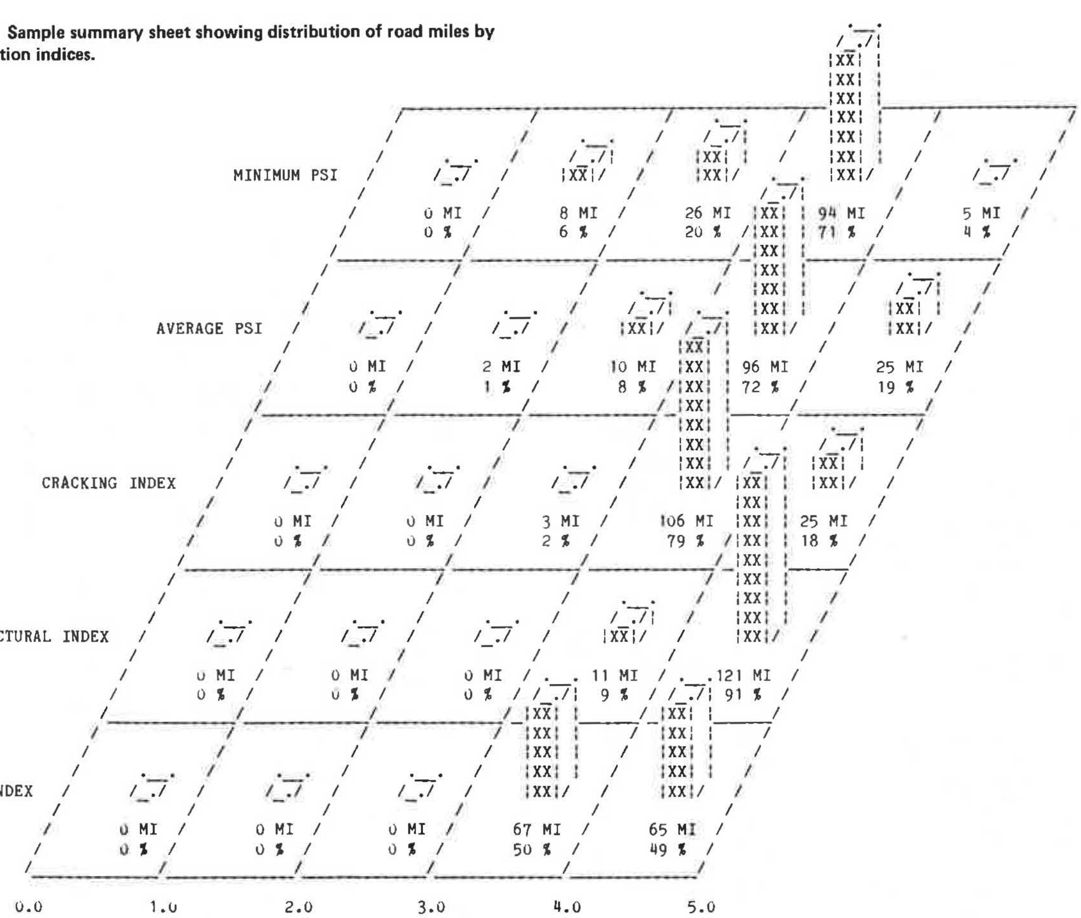


Figure 5. Sample summary output showing distribution of road miles by evaluation indices when structural index is excluded.

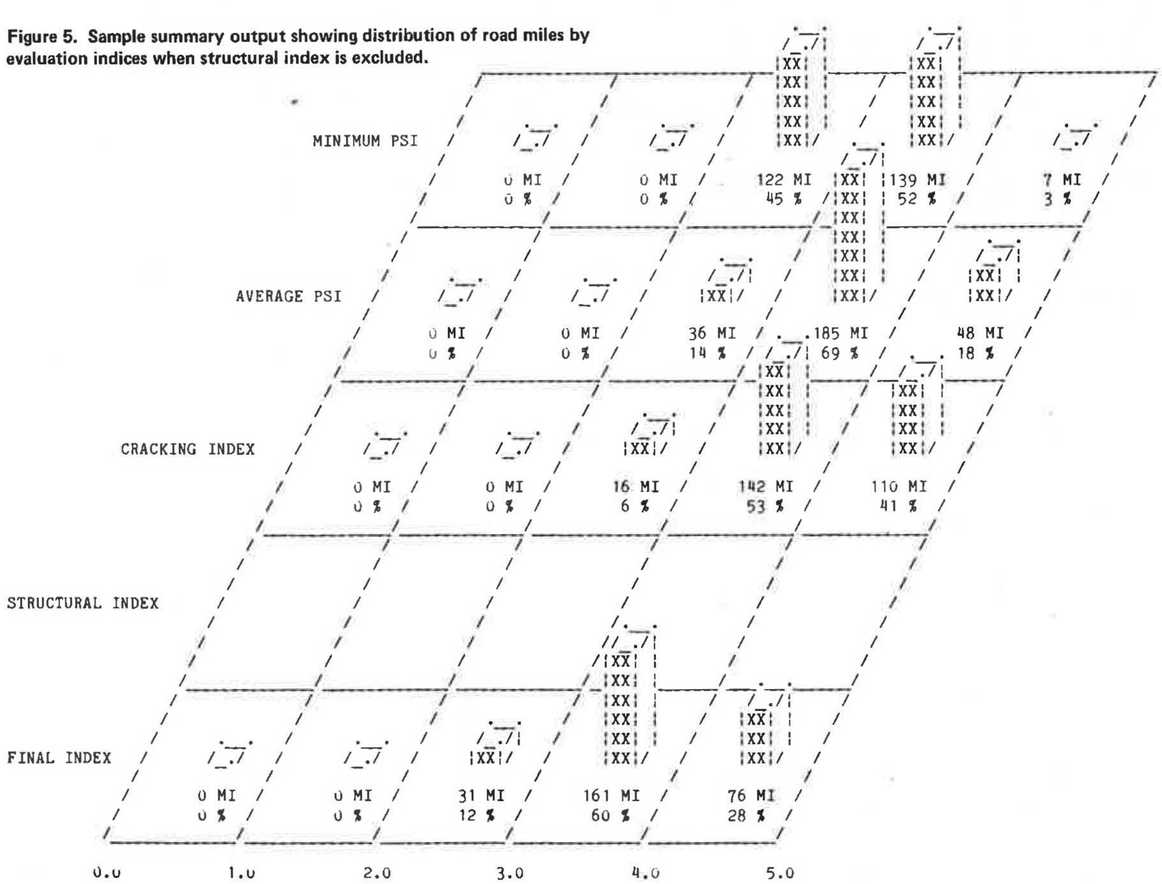


Figure 6. Sample output from program summary.

FINAL SUMMARY TABLE
OF
PAVEMENT EVALUATION
FOR
DISTRICT NO. 4

STRUCT=STRUCTURAL INDEX
CRACK=CRACKING INDEX
PSI=PSI INDEX
AVG FRIC=AVERAGE FRICTION INDEX

	CNTY	ROAD SEGMENT CODE	LENGTH	BEGINNING TERMINI	START	ENDING TERMINI	END	FINAL INDEX	RANKING OF STRUCTURE, FRICTION CRACKING & PSI: STRUCT	CRACK	PSI	AVG FRIC
1	53	08070D	3.66	SH 79 IC	168.36	J COOLEE CANAL	164.70	3.0	2.5	3.1	3.2	31.0
2	31	08070D	5.80	MINIDOKA CO LINE	216.04	SALT LAKE IC	221.84	3.0	4.5	2.5	2.3	34.0
3	31	08070D	5.80	SALT LAKE IC	221.84	MINIDOKA CO LINE	216.04	3.1	4.5	3.1	2.1	31.0
4	53	08070D	6.40	NORTHSIDE CANAL	194.14	MINIDOKA CO LINE	200.54	3.1	4.5	2.6	2.5	39.0
5	53	08070D	6.40	MINIDOKA CO LINE	200.54	NORTHSIDE CANAL	194.14	3.1	4.5	2.6	2.5	37.0
6	67	08070D	7.21	JEROME CO LINE	200.54	SH 27 IC	207.75	3.2	4.5	2.4	2.9	39.0
7	31	08070E	6.28	YALE RD IC	228.12	SALT LAKE IC	221.84	3.3	4.5	2.9	2.6	33.0
8	53	08070D	4.33	US 93 IC	172.69	SH 79 IC	168.36	3.3	1.5	4.0	4.2	39.0
9	67	08070D	7.21	SH 27 IC	207.75	JEROME CO LINE	200.54	3.4	4.5	2.9	2.9	34.0
10	53	08070D	6.11	VALLEY ROAD	188.03	NORTHSIDE CANAL	194.14	3.4	3.5	3.0	3.6	29.0
11	31	01560A	2.26	POWER COUNTY LINE	17.58	NEAR RAFT RIVER	15.32	3.5	2.0	4.1	4.1	55.0
12	53	08070D	3.95	US 93 IC	172.69	DRAIN DITCH	176.64	3.5	2.0	4.1	4.1	56.0
13	67	08070D	8.29	SH 27 IC	207.75	CASSIA CO LINE	216.04	3.5	3.5	2.9	3.8	43.0
14	53	08070D	4.33	SH 79 IC	168.36	US 93 IC	172.69	3.5	2.0	4.1	4.1	50.0
15	31	01560A	2.26	NEAR RAFT RIVER IC	15.32	POWER COUNTY LINE	17.58	3.5	2.0	4.1	4.2	55.0
16	53	08070D	6.11	NORTHSIDE CANAL	194.14	VALLEY RD	188.03	3.5	4.0	3.0	3.5	30.0
17	67	08070D	8.29	CASSIA CO LINE	216.04	SH 27 IC	207.75	3.6	4.5	3.0	3.4	42.0
18	53	08070D	5.06	DRAIN DITCH	176.64	SH 50 IC	181.70	3.6	3.5	3.6	3.8	49.0
19	31	08070E	6.27	SWEETZER ROAD	253.91	POWERLINE CROSSING	247.64	3.7	4.5	2.8	3.6	44.0
20	53	08070D	6.33	VALLEY RD	188.03	SH 50 IC	181.70	3.7	4.0	3.3	3.8	33.0
21	53	08070D	3.38	J COOLEE CANAL	164.70	GOODING CO LINE	161.32	3.8	4.5	3.0	3.7	35.0
22	31	01560A	8.00	SALT LAKE IC	0.00	MILE POST 8	8.00	3.8	4.5	3.4	3.5	42.0
23	47	08070D	5.29	GOODING CO LINE	161.32	WENDELL WCL	156.03	3.8	4.5	3.1	3.8	39.0
24	31	08070E	6.28	SALT LAKE IC	221.84	YALE RD IC	228.12	3.8	4.5	3.3	3.7	42.0
25	47	08070D	8.67	EAST BLISS IC	140.81	BEG DIVIDED HWY	149.48	3.9	4.0	4.1	3.6	27.0
26	31	08070E	6.70	POWERLINE CROSSING	234.82	YALE RD IC	228.12	3.9	4.5	3.2	3.8	42.0
27	31	08070E	7.11	SWEETZER RD	253.91	ONEIDA CO LINE	261.02	3.9	4.5	3.0	4.0	52.0
28	31	01560A	7.32	NEAR RAFT RIVER IC	15.32	MILE POST 8.000	8.00	3.9	4.5	3.4	3.8	39.0
29	31	08070E	7.11	ONEIDA COUNTY LINE	261.02	SWEETZER RD	253.91	3.9	4.5	3.3	3.9	50.0
30	53	08070D	6.33	SH 50 IC	181.70	VALLEY ROAD	188.03	3.9	4.0	3.7	4.0	31.0
31	53	08070D	3.95	DRAIN DITCH	176.64	US 93 IC	172.69	4.0	3.5	4.1	4.2	52.0
32	31	08070E	6.44	POWERLINE CROSSING	247.64	HALFWAY BTW POWERLINES	241.20	4.0	4.5	3.5	3.9	42.0
33	31	08070E	6.70	YALE RD IC	228.12	POWERLINE CROSSING	234.82	4.0	4.5	3.5	4.0	36.0
34	53	08070D	3.38	GOODING CO LINE	161.32	J COOLEE CANAL	164.70	4.0	4.5	3.7	3.9	40.0
35	31	01560A	7.32	MILE POST 8	8.00	NEAR RAFT RIVER IC	15.32	4.0	4.5	3.5	4.0	41.0
36	31	08070E	6.27	POWERLINE CROSSING	247.64	SWEITZER RD	253.91	4.1	4.5	3.3	4.2	44.0
37	53	08070D	3.66	J COOLEE CANAL	164.70	SH 79 IC	168.36	4.1	4.0	4.1	4.1	52.0

Figure 7. Sample page of POD output for rigid pavement section of I-90.

SECTION : I - 90

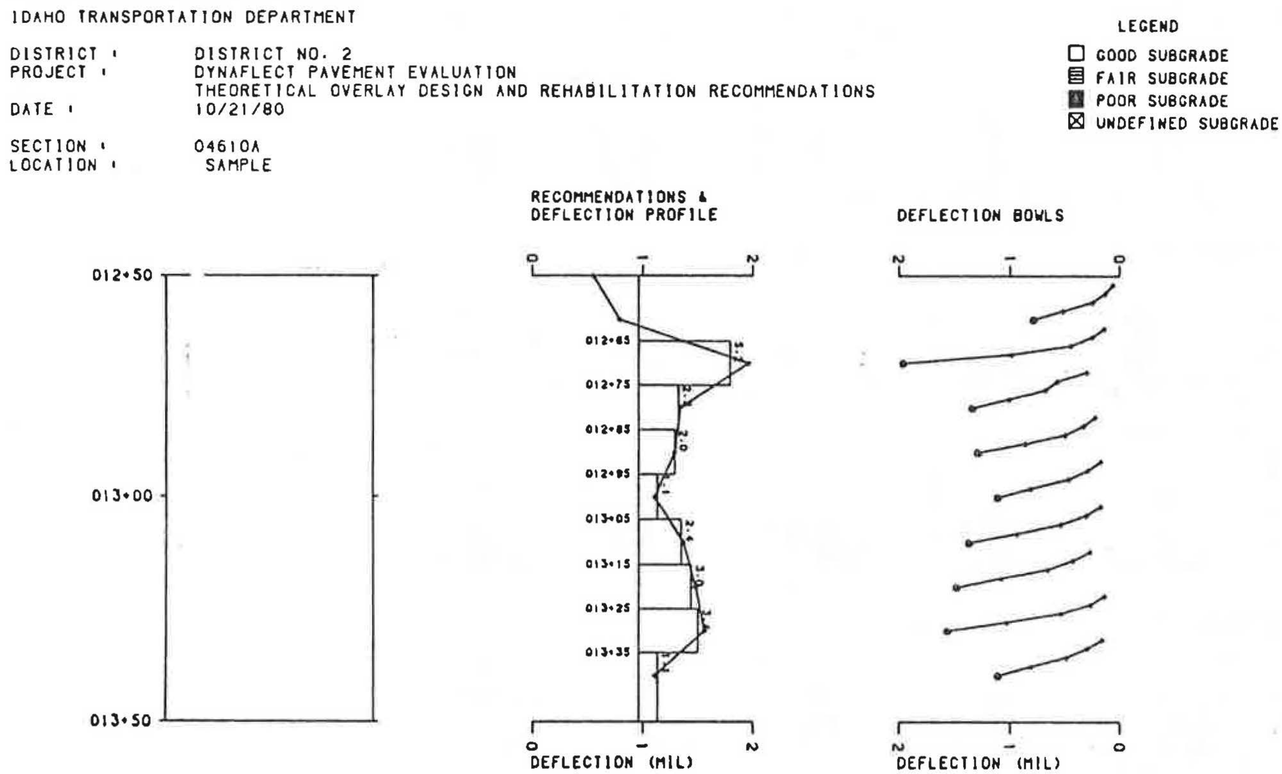
MILEPOST	DYNAFLECT SENSOR READINGS NO.1	NO.2	NO.3	NO.4	NO.5	WIC	SCI	BCI	SPD	YRS TO FAIL	OVERLAY THICKNESS	PAVEMENT SYSTEM CONDITION
66.60	0.71	0.64	0.54	0.41	0.32	1.01	0.07	0.09	73.8	8	6.93	
66.80	0.49	0.43	0.35	0.26	0.18	0.65	0.06	0.08	69.8	12	4.30	
67.00	0.35	0.30	0.22	0.15	0.08	0.52	0.05	0.07	62.9	21	0.00	
67.20	0.39	0.36	0.29	0.22	0.16	0.82	0.03	0.06	72.8	11	4.00*	
67.40	0.45	0.42	0.35	0.27	0.19	0.51	0.03	0.08	74.7	12	4.55	

* INDICATES THAT THE MINIMUM/MAXIMUM AC OVERLAY THICKNESS IS RECOMMENDED

SUMMARY FOR SEGMENT NO. 5 OF THIS SECTION

	NO.1	NO.2	NO.3	NO.4	NO.5	WIC	SCI	BCI	SPD	OVERLAY THICKNESS
MEAN VALUE	0.48	0.43	0.35	0.26	0.19	0.70	0.05	0.08	70.8	3.96
MINIMUM	0.35	0.30	0.22	0.15	0.08	0.51	0.03	0.06	62.9	0.00
MAXIMUM	0.71	0.64	0.54	0.41	0.32	1.01	0.07	0.09	74.7	6.93
STD. DEVIATION	0.14	0.13	0.12	0.10	0.09	0.21	0.02	0.01	4.8	2.50

Figure 8. Sample of POD graphical output.



based on the original work described by McCullough (9) and its subsequent expansion to include more recently developed methods of pavement materials characterization. Detailed description of one model employed in PPMIS can be found elsewhere (10).

Rigid Pavement Condition Rating System

The existing form of the condition (cracking) index model was found to be inadequate for rigid pavements because it was developed primarily for flexible pavements. A discriminant analysis procedure developed for the Texas State Department of Highways and Public Transportation (11) was adapted to jointed concrete pavements as described in detail elsewhere (10).

Partial Network Level Analysis

SYSTDY and SUMMARY programs were modified to operate with less than the standard number of inventory parameters (i.e., deflection, roughness, condition, and skid). This feature was thought to give more flexibility to ITD in the use of the system.

Expansion of PPMIS Program Features

The inclusion of structural analysis models for rigid pavements in PPMIS required considerable expansion of the SYSTDY and POD programs. Some other expansions were also necessary as a result of the changes in analysis techniques employed. Details of these expansions can be found elsewhere (8,10).

Application and Verification of Expanded and Modified Idaho PPMIS

Network Analysis Application

To test the reasonableness of the expanded and modified portions of the Idaho PPMIS the Interstate field inventory data collected during the phase 1 project in 1979 were used as input for a number of runs of the new version of PPMIS programs.

Of chief concern was the reasonableness of the newly developed rigid pavement analyses. The 1979 inventory included the 216.8 miles of rigid Interstate pavements. These pavements could not be considered in the phase 1 verification runs because rigid-pavement analyses had not yet been developed and incorporated into PPMIS. The new expanded and modified PPMIS made possible the determination of the need (priority) for improvement, as indicated by the final index, on a network of pavements that contain both flexible and rigid sections.

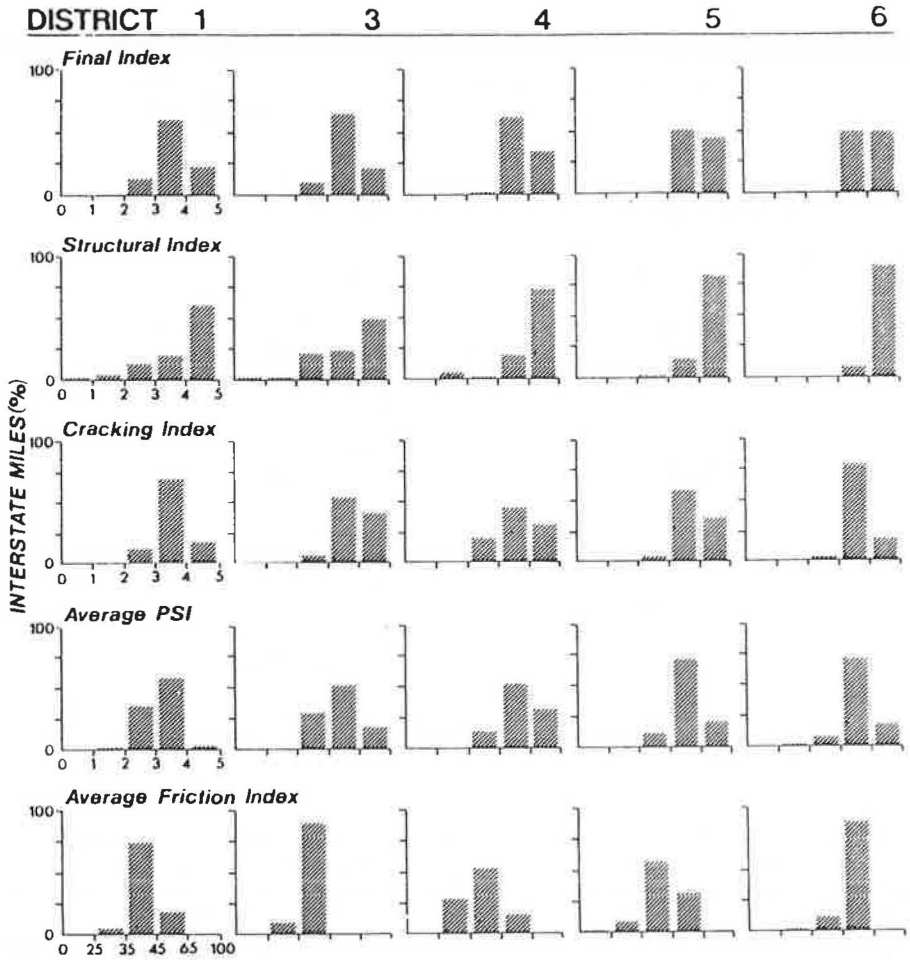
Figure 9 presents the summary results of the network analysis in terms of the distributions of final index, structural index, cracking index, average PSI, and average friction index for five districts in Idaho.

The comparison of final indices presented in Figure 9 indicates that districts 4, 5, and 6 are in much better condition than districts 1 and 3. This could perhaps be because district 1 and especially district 3 are the most highly populated districts in the state and, accordingly, would have the highest traffic volumes.

The district 6 Interstate mileage appears to be in the best structural condition; no mileage is in the structural index range of 0 to 2.9 and the district also has the highest percentage in the 4.0 to 5.0 range. The Interstate mileage in districts 1 and 3 is again in the poorest structural condition of all the districts in the state.

Figure 9 indicates that no section of Interstate mileage in the state has a cracking index between 0.0 and 1.0. District 5 appears to have the most Interstate mileage; the cracking index is between 4.0 and 5.0. The bulk of the Interstate mileage appears to be between 3.0 and 3.9.

Figure 9. Comparisons of Interstate mileage.



In terms of average PSI the Interstate mileage in district 1 was found to be in the poorest condition as compared with the other districts in the state. No Interstate mileage has an average PSI below 1.0 and only 2 miles appear to have an average PSI between 1.0 and 1.9. The bulk of the Interstate mileage has an average PSI between 3.0 and 3.9.

In terms of skid numbers, the district 6 Interstate mileage has only one mile below a skid number (SN) of 36 and is clearly the best in the state. Similarly, district 3 has by far the highest percentage between 36 and 45 and no mileage above a value of 46. This indicates that, although district 3 Interstate pavements are generally fair at the present time with respect to skid resistance, careful periodic monitoring is warranted to determine any changes.

These district comparisons can assist in determining initial budget allocations for improvement to the Interstate pavements among Idaho's districts. The initial allocations would be based on the estimated relative needs of each district for Interstate pavement improvements. A further refinement to this initial allocation could be effected by the inclusion of economic analysis and network optimization.

In addition to the network output shown in Figure 9, a more detailed output similar to the example given in Figure 3 was produced for each section on the Interstate system in the state. They were evaluated by ITD personnel and were found to be reasonable for ITD's purposes.

Project Analysis Application

The testing and application of the project analysis portion of the Idaho PPMIS (i.e., program POD) was done by using flexible pavement data received from each district in the state. A test run was also made using an example of a rigid pavement.

The detailed deflection testing on the projects was conducted by ITD personnel and equipment. The projects (one in each district) had been selected by districts and generally were real projects scheduled for improvement in an immediate future.

The results of the program POD were discussed in detail with the districts and were found to be reasonable. The formats in which the results were presented were also found to be reasonable and helpful to the district materials engineers.

The POD analysis results for one concrete section in district 1 were also discussed with the IDT personnel. The results were generally found quite reasonable; however, a more thorough analysis covering a number of concrete sections is necessary before a final conclusion can be drawn. On the other hand, the SYSTDY program produced reasonable results for the concrete sections in the state by using the same procedures as in the POD program. This tends to indicate that the POD program should produce reasonable results for concrete pavements as well as for flexible pavements.

INSTALLATION AND TRAINING COURSE

The modified and extended version of Idaho PPMIS program has been installed on ITD's computer facilities in Boise, Idaho, after the assessment of the results. Extensive test runs were made to debug the programs and to overcome the problems associated with installation of a software package on a different hardware. A 2-day training course was held to train the ITD personnel in the use of the system. Detailed engineering and software documentations along with user manuals were provided to the potential users in ITD to facilitate ease of use of the system.

The Idaho version of the new PPMIS program is now operational and is being used effectively by ITD personnel for the management of the state's pavement network.

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Rigid Pavement Network Rehabilitation Scheduling

MANUEL GUTIERREZ de VELASCO AND B.F. McCULLOUGH

The development and application of a scheme, in the form of a computer program, to order the priority and schedule a set of rigid pavements for rehabilitation within a specified time frame and budget constraints are presented. The scheme makes use of a distress index to order the priority of a group of pavement sections and to decide when a pavement has reached its terminal condition. The distress index is calculated by combining into a single number the various distress manifestations that occur in a pavement section. The initial pavement condition is determined from field distress condition surveys and the future condition is determined by means of distress prediction models. The immediate application of the computer program is to generate lists of candidate pavements for rehabilitation; however, the use of the program can be extended to analyze the effect of several different budget policies on the condition of the pavement network.

The need for better management tools to allocate money, staff, equipment, and materials in an efficient manner has become evident with the continuous increment of requirements to maintain and rehabilitate a pavement network.

A relatively small amount of research effort has been placed on restoration as compared with the provision of new facilities because most previous capital investments have been centered on construction of new roads. This trend is reversing, however, and the prime effort is shifting toward the maintenance and rehabilitation of existing pavements.

During the last decade, pavement management systems (PMS) have been applied successfully to improve the management and technology of pavements (1-3). Among the PMS studies, the methods for planning maintenance and rehabilitation in a pavement network have become relevant in recent years. The desired result from a network application is a work program for each year during an analysis period. However, different degrees of complexity can be achieved and, for an agency without PMS experience, starting with a simplified version and progressing in a staged manner has been suggested (4). The concepts, steps involved in the development, and application of a pavement rehabilitation priority-ordering and scheduling scheme at the network level are presented.

PMS

PMS involves the application of systems engineering to assist decision makers in defining optimum strategies for maintaining pavements in a serviceable condition over a given period of time. The development of PMS is a cyclic procedure that works toward an ideal system. For example, improvements are achieved by continuous upgrading of the network by using the models and the algorithms in the program to predict the rehabilitation needs. An ideal system should be capable of predicting (a) the precise future condition of each project in a given network, (b) the proper timing and type of maintenance required, (c) the date to overlay, (d) the costs, and (e) the consumption of resources.

Management decisions involving pavements can be considered at two different levels: the network and the project. A network consists of a series of projects under the jurisdiction of an agency. A project is a pavement unit that has been defined by the agency for construction and record-keeping purposes. The developments in this paper are confined to those at the network level.

At the network level the management system provides information to help decision makers in the development of agencywide programs of new construction, maintenance, or rehabilitation that will make optimum use of available resources (3). The results of the analysis should provide a program for construction, maintenance, and rehabilitation of pavements within available resources. Several schemes for maintenance and rehabilitation management have been presented in the literature or are currently in use by state agencies (4-11). Each one is different, which is a reflection of the needs of a particular agency.

SYSTEM OUTPUT FUNCTION

Among the important developments required in PMS is an output function that involves the various parameters that affect decision making in pavements, such as riding quality, skid resistance, distress, traffic, and costs. In general, riding quality has been the most important factor considered, primarily because of the influence of the AASHO Road Test, where the concept of present serviceability index (PSI) was developed. Although the PSI equation includes patching and cracking, roughness is the dominating term. From experience with rigid pavements in Texas, distress was found to be a more useful output function for ordering the priority and scheduling a set of pavements for rehabilitation. Thus, sources of information for developing productive algorithms must be used.

Continuously Reinforced Concrete Pavement System Output Function

In a large number of the cases observed the pavement serviceability history does not appear to change with time or traffic; however, the distress condition does change. Figures 1 and 2 indicate how serviceability and distress vary with traffic for Texas pavements. Each point represents a surveyed project (1 to 10 miles) of continuously reinforced concrete pavement (CRCP) in Texas (12,13). The serviceability index was derived from roughness data obtained by using profilometer measurements. The traffic figures were provided by the Texas State Department

Figure 1. Serviceability index versus traffic applications (both directions) for Texas CRCP sections surveyed in 1974 and 1978.

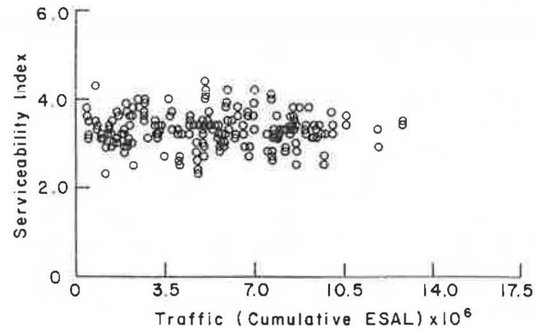
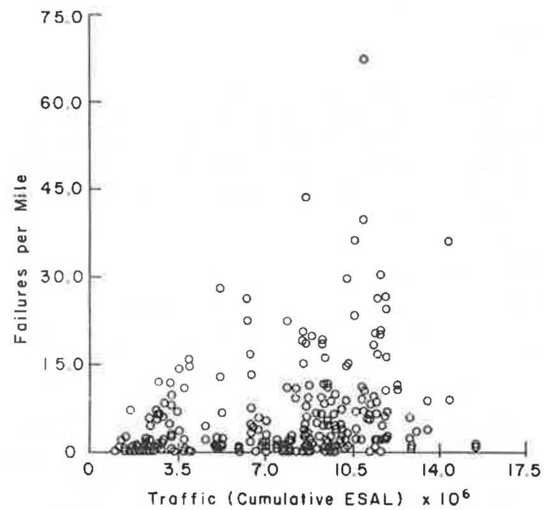


Figure 2. Number of failures per mile (punchouts and patches) versus traffic applications (both directions) for Texas CRCP sections surveyed in 1974 and 1978.



of Highways and Public Transportation (TSDHPT). The number of failures (punchouts and patches) per mile was obtained from the records of the CRCP condition surveys performed in Texas in 1974 and 1978 (described later in the paper). The figures show that the serviceability index is independent of the traffic; i.e., the serviceability index value does not vary. Unfortunately, only a few of these projects have been monitored over time. For these cases history shows that the pavements remain near the initial PSI. One likely reason for having a constant serviceability is the continuous repair of the highway performed by the district's staff. Hence, although from a structural or economics standpoint the section is approaching its terminal point, the riding quality remains unchanged. Thus, the use of distress measures may be a more realistic way to evaluate a pavement's terminal condition. This observation is contrary to the basic hypothesis of the AASHO guides for rigid pavement (14). However, these concepts, which are based on results of the AASHO Road Test (15), do not consider that deteriorating pavement sections receive maintenance.

Therefore, distress manifestations (in this case failures per mile) appear to be better indicators of the deterioration of CRCP than the serviceability index, as seen by the variability. In other words, with CRCP distress generally appears to be a more significant factor in the decision-making process than the serviceability index.

An additional advantage of using distress is that it relates directly to maintenance requirements and measures indirectly other pavement functional indicators such as serviceability. Among the disadvantages of using distress manifestations is the lack of cost equations, because past research has made extensive use of the PSI concept in developing these equations.

Sources of Information for Predictive Algorithms

Field data were collected for CRCP and asphalt concrete (AC) overlaid pavements, and literature information used for jointed pavements.

CRCP

Condition surveys (i.e., field measurements of distress to the pavement condition) have been carried out by the Center for Transportation Research (CTR). The rural districts in Texas were surveyed in 1974, 1978, 1980, and 1982. The urban districts were surveyed in 1976 and 1982. The following manifestations were measured: transverse cracking, localized cracking, spalling, pumping, punchouts, and patches. Detailed information on the condition survey procedures used is given elsewhere (16).

Jointed Pavements

Although jointed pavements [jointed concrete pavement (JCP) and jointed reinforced concrete pavement (JRCP)] are used in the state, this type of pavement has not been monitored on a regular, scheduled basis. Therefore, other sources of information were used in this study. Data used by Carey and Irick (17) to develop the serviceability-performance concept were used to develop some of the distress models (16). Other models have also been adopted from the literature (15,18,19).

Rigid Pavements Overlaid with AC Pavement

The monitoring of overlaid rigid pavements is a recent task; therefore, the existing information does not present extensive time histories of distress occurrence. A project in Walker County on I-45 represents one of the oldest, better monitored asphalt concrete overlays of rigid pavements in the state (i.e., it has a history of approximately 14 years).

DEVELOPMENT OF COMPUTER PROGRAM

Although a PMS is not necessarily a computer program, the amount of calculations necessary render the development of computer programs essential to transform the concepts into a working reality. The key issue of any PMS is to move past the conceptual stage and develop an actual working system.

Program PRP01

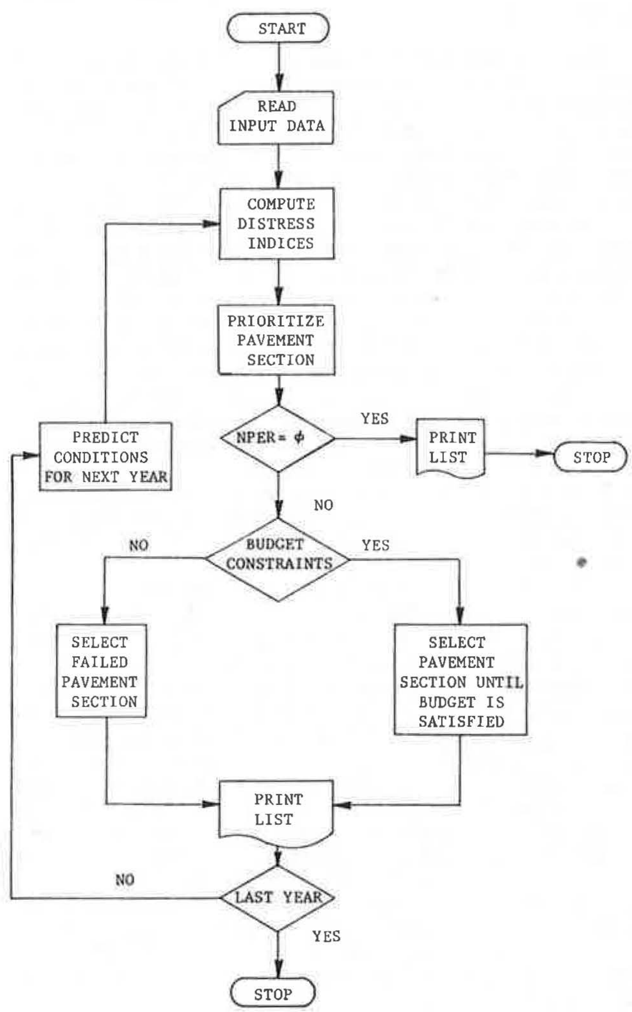
Program PRP01 was developed to schedule rehabilitation of rigid pavements (JCP, JRCP, and CRCP) within a certain design period. The input data are condition survey information on a set of rigid pavements for the same year. The solution is obtained by using distress models (i.e., distress indices and distress prediction equations). All of the distress models were integrated as subroutines in the program to facilitate future modifications.

The program output has several alternatives:

1. A priority-ordered list of pavement sections according to their distress condition at the time of the condition survey;
2. A multiperiod rehabilitation schedule of the pavement sections without consideration of budget constraints; the selection of candidates for each year is made on the basis of the magnitude of the distress index; and
3. A multiperiod rehabilitation schedule of the pavement sections that account for budget restrictions; the selection for each year depends on the magnitude of the distress index and budget availability.

Figure 3 is a simplified flowchart of the computer program. Information on the distress condition of each project is required as input. The program starts by calculating the distress index for each section. Then the priority order of the sections is determined according to the magnitude of their distress indices. The design period is checked at this stage: If the design period is set equal to zero the program prints the priority list and stops. If the design period is larger than zero the program continues. Budget restrictions are checked next by using two different criteria: If no budget constraints are imposed by the user, the rule for selecting the candidates for rehabilitation is that all pavements that reach terminal condition are included in the list for that year. If budget constraints are present the selection of candidates is made on the basis of availability of funds. The

Figure 3. Simplified flowchart of computer program PRP01 developed to order the priority of and schedule rehabilitation.



rehabilitation cost of each project is calculated in order of project priority and this cost is accumulated until the budget constraint is reached. A list of candidate projects is printed for each year of the design period. The program checks to see if the design period has been covered, in which case it exits; otherwise, conditions are predicted for the next year and the program returns to the step in which the distress indices are calculated.

Models in Program PRP01

Distress Index

Several approximate methods for developing a distress index were studied--subjective parameters, regression analysis, factor analysis, and discriminate analysis (16, 20-23). The following conclusions have been obtained from the study of these methods.

1. The equations that have subjective parameters rely heavily on engineering judgment and experience and, therefore, are useful when sufficient information is not available.
2. Factor analysis is difficult to interpret and the assumption used in this approach (i.e., the resulting equations measure structural performance or deterioration of a pavement section) is not supported.
3. Regression analysis and discriminant analysis are feasible techniques for developing distress and decision criteria indices; the selection of one or the other is dependent on the dependent variable selected.

Discriminant analysis was chosen to develop rigid pavement distress indices because it conforms to available data. It is a statistical technique used to classify data into groups; its objective is to construct a boundary (i.e., a discriminant equation) such that the elements of each group can be separated. Once the equation is defined any new element can be assigned to one of the predetermined groups. This technique was applied to develop an equation to discriminate CRCPs that have an acceptable level of distress from pavements that require overlay.

Distress condition surveys of CRCP in Texas were performed in 1974 and 1978. Several distress manifestations were recorded--punchouts and patches per mile, percentage of minor spalling, percentage of severe spalling, and percentage of pumping. Some of the pavements surveyed during 1974 were overlaid before the survey in 1978. These data are used to determine the reasons for the decision to overlay. Data on several variables from two groups (overlaid and nonoverlaid pavements) that describe their difference are used.

The jointed pavement data used in the analysis are those used by Carey and Irick (17) to develop the serviceability-performance concept. The justification for the use of this information is based on the findings of Hutchinson (24) and Weaver (25). Hutchinson found that subjective estimation procedures, typified by Road Test panel ratings, were inappropriate for the task because they tended to measure pavement distortion and deterioration rather than riding quality, which is the essence of serviceability. Weaver reinforces this point in his results to develop a serviceability index for New York. He found that inclusion of experts in the rating panels or inappropriate definition of objectives biases the results of serviceability studies. Therefore, the acceptability or unacceptability of pavement sections in the Road Test was assumed to be influenced by the pavement condition.

Although the outcome of the discriminant analysis is a decision criteria index, its relative magnitude can be used as a distress index. Further details of the application of this technique are presented elsewhere (13,16).

The following discriminant equations were obtained by using the statistical package for the social sciences (SPSS) (20). The discriminant score can be interpreted as follows: If it is positive for a given pavement section then the section is in good condition; if the score is negative (i.e., smaller than zero) the section is considered to have failed. The larger the magnitude of the discriminant score, the better the condition of the pavement. The equation obtained from continuous pavements was of the form

$$Z_c = 1.0 - 0.065FF - 0.015MS - 0.009SS \qquad (1)$$

where

Z_c = distress index or discriminant score for continuous pavements,
FF = failures (punchouts and patches) per mile,
MS = percentage of minor spalling, and
SS = percentage of severe spalling.

The equation classified correctly 88 percent of the 224 cases used in the analysis. Of course, the prediction capability of the discriminant equation will need to be checked in the future. The equation obtained for jointed pavements, after algebraic manipulation so that it resembles Equation 1, was

$$Z_j = 1.0 - 0.005C - 0.006S - 0.021P - 0.003F \qquad (2)$$

where

Z_j = distress index or discriminant score for jointed pavements,
C = cracks (number per mile),
S = spalling (%),
P = patches (number per mile), and
F = faulting in wheelpath (number per mile).

The equation classifies correctly 92 percent of 49 cases.

Distress Prediction Equations

The initial pavement condition is determined from field distress condition surveys and input into the program. The future condition is calculated internally by means of distress prediction equations. Field data were used to obtain models for CRCP and AC overlaid rigid pavements through regression analysis. The models for jointed pavements have been adopted from the literature (15,18,19). The models derived assume that at some point in time information on the distress of a pavement was collected and used to forecast the future condition of the pavement.

The models developed predict failures (punchouts and patches), minor spalling, and severe spalling for CRCPs and cracking, spalling, and faulting for jointed pavements. Further information on the distress prediction equations and their development is presented elsewhere (16).

The information used for the development of the equations did not come from an experimental design but from data collected primarily with the purpose of evaluating pavement conditions. Future improvements of distress prediction equations should include experimental design techniques. Guidelines exist in the literature (26,27) for that purpose.

Overlay Thickness

The field information collected about overlaid rigid pavements was used to derive an equation for calculating an approximate AC overlay thickness and, from it, to estimate the overlay cost of a pavement section. The thickness design can be explained by using the data in Figure 4; the figure shows the average percentage of distress history for different overlay thicknesses as monitored in the field. The program selects a percentage from the original distress of a pavement section, which corresponds to the failure condition, and an overlay life. An approximate thickness is estimated from two inputs.

Outputs from Program PRP01

Figures 5-7 are partial outputs of sample runs made with the priority-ordering and scheduling program PRP01 and are used to explain the contents of the lists produced by the program for the various available options.

Figure 5 is the type of output generated when the option selected is priority ordering of projects using the condition survey information directly. This option does not involve any type of distress prediction. The program calculates the distress index for each of the sections and sorts them all according to the relative magnitude of the indices, the worst condition first. The output contains four columns. The first column is section identification. The second is the distress index (note that the numbers increase progressively as the condition of the sections improves). The third column is the cumulative equivalent single axle loads (ESALs) that were input by the user. The last column is the ranking of each section as obtained from the distress indices, with the poorest pavement listed first.

Figures 6 and 7 are the type of output obtained for both the second and third options of the program (i.e., multiperiod rehabilitation scheduling with and without budget constraints). Any of these options produces a list of projects that require overlay for each year of the design period (similar to Figure 6) and a summary for the design period (similar to Figure 7). Figure 6 represents a specific year of the year-by-year output, year 1, in six columns. The first three columns and the sixth column are identical to those in Figure 5. The fourth column contains the project lengths in miles, and the fifth column contains the estimated cost of each overlay. At the bottom of the printout the total length and the total cost to overlay the candidate projects for the specific year are printed.

For the second option (scheduling of pavement sections without budget constraints) the distress

Figure 4. Plot of average percentage of distress time history for different overlay thicknesses as monitored in Walker County, Texas.

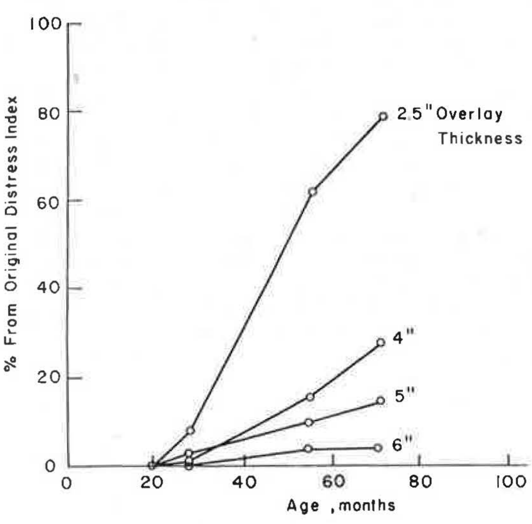


Figure 5. Sample output from program PRP01 by using priority-ordering option.

PRIORITY LIST OF TX CRCP FOR REHABILITATION
INPUT DATA FROM 1980 CONDITION SURVEY

LIST OF PRIORITIZED SECTIONS AT TIME OF CS

SECTION ID	DISTRESS INDEX	CUMULATIVE ESAL (MILLIONS)	RANK
1908EB	-4.476	5.480	1
1908WB	-2.215	5.480	2
1007WB	-2.124	5.830	3
1906EB	-2.041	5.310	4
1014EB	-1.694	5.160	5
1006WB	-1.688	5.940	6
904 SB	-1.652	7.220	7
1001EB	-1.490	5.700	8
1007EB	-1.384	5.830	9
1004WB	-1.194	5.680	10
1303WB	-1.073	2.020	11
2009WB	-1.062	3.160	12
1902EB	-1.057	5.770	13
1906WB	-1.007	5.310	14
1001WB	-.941	5.700	15
2009EB	-.909	3.160	16
1303EB	-.864	2.020	17
1014WB	-.850	5.160	18
1008EB	-.760	4.440	19
1010EB	-.739	5.130	20
1305SB	-.683	1.490	21
1306WB	-.653	1.820	22
1004EB	-.513	5.680	23
907 NB	-.475	5.830	24
1902WB	-.465	5.770	25
1010WB	-.445	5.130	26
1313WB	-.391	1.910	27
1011WB	-.334	4.630	28
108 SB	-.290	3.250	29
1003EB	-.276	5.950	30
1307WB	-.245	1.510	31
1909EB	-.242	6.420	32
1904WB	-.238	6.160	33
1002EB	-.200	5.860	34
1904EB	-.199	6.160	35
1706SB	-.198	5.490	36

Figure 6. Sample output page from computer program PRP01 by using scheduling option.

PRIORITY LIST OF TX CRCP FOR REHABILITATION
INPUT DATA FROM 1980 CONDITION SURVEY

LIST OF PAVEMENT SECTIONS REQUIRING OVERLAY
YEARS AFTER CONDITION SURVEY= 1

SECTION ID	DISTRESS INDEX	CUMULATIVE ESAL (MILLIONS)	SECTION LENGTH (MILES)	OVERLAY COST (DLLS)	RANK
1908EB	-4.[illegible]37	5.918	9.80	4686232.	1
1908WB	-3.[illegible]55	5.918	10.00	3629770.	2
1906EB	-2.784	5.629	7.00	2454482.	3
1007WB	-2.617	6.180	5.00	1714050.	4
904 SB	-2.322	7.653	1.80	591402.	5
1014EB	-2.218	5.676	8.20	2651398.	6
1006WB	-2.123	6.296	5.20	1656546.	7
1001EB	-1.749	6.099	4.00	1195507.	8
1007EB	-1.617	6.180	4.80	1399609.	9
	-.094		55.80	19978996.	

Figure 7. Sample output summary from computer program PRP01 by using scheduling option.

PRIORITY LIST OF TX CRCP FOR REHABILITATION
INPUT DATA FROM 1980 CONDITION SURVEY

SUMMARY TABLE

YEAR	AVG. DI	LENGTH (MILES)	BUDGET (DLLS)
1	-.094	55.80	19978996.
2	.032	64.30	18852102.
3	.134	65.10	17574961.
4	.204	75.90	19301323.
5	.307	75.30	17445063.
6	.452	93.40	19739679.
7	.556	99.90	19022178.
8	.699	108.40	19017960.
9	.833	114.70	18582421.
10	.996	0	0
	.415	756.50	169514682.

indices for years other than the first one in the design period are close to zero and of equal value. Therefore, further ranking of the sections can be made in terms of cumulative ESAL.

Figure 7 presents the summary of the year-by-year analysis, which includes the average distress index calculated for the network, the total length of projects recommended for rehabilitation, and the yearly budget. An overall summary is printed in the lower part of the table.

PROGRAM APPLICATIONS

The obvious application of the computer program PRP01 is to generate lists of candidate pavements for rehabilitation. The use of the program can be extended to analyze the impact of several different budgeting policies on the condition of the pavement network. The purpose of this section is to present the effects of different budget policies by using information from the 1980 East Texas CRCP condition survey. The data used for the analysis came from 139 sections, representing 7 districts, with a total length of 756.5 miles and age that ranges from 9 to 18 years.

Analysis Approach

Several computer runs were performed for a 10-year analysis period using several budget levels--$5, $10, $15, $20, and $30 million per year. An additional computer run was developed without considering budget restrictions. The output of the runs was plotted to observe the effect of the various yearly budgets on the distress condition of the pavement network.

The numbers used in the analysis are not definitive because the cost of overlay used was approximate. An accurate figure should include costs such as the costs of handling traffic, materials, equipment, and labor.

Effect of Yearly Budget

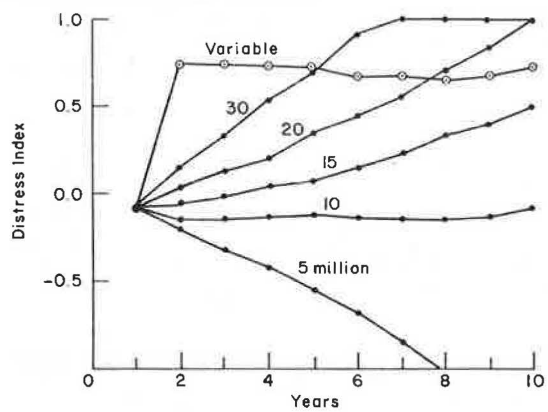
Table 1 gives summary information for each budget level considered in the analysis. The second column

Table 1. Summary information for several different budget levels from computer program PRP01 by using Texas CRCP information.

Budget Level ($000,000s/year)	Length Repaired (miles)	Budget Used ($000,000s)	Avg Overlay Cost per Mile ($000s/mile)	Avg Distress Index
Variable	532.2	119.957	225.40	0.628
5	70.8	35.052	495.09	-0.670
10	261.0	91.934	352.24	-0.128
15	506.3	137.974	272.51	0.154
20	756.5	169.515	224.08	0.415
30	756.5	157.850	208.66	0.648

Note: Ten-year analysis period.

Figure 8. Average distress index for network through time for various yearly budgets using Texas CRCP information.



contains the total number of miles repaired for the design period considered, and the third column contains the total budget used in the design period, in millions of U.S. dollars. The fourth column contains the average overlay cost per mile for each budget level, without considering the time value of money. The average overlay cost per mile was obtained by dividing the total budget by the number of miles repaired. Column five gives the average distress index for each budget level. The poor condition of the network, exemplified by negative average distress index values for the low budget levels, is obvious, along with the improved condition for higher budgets.

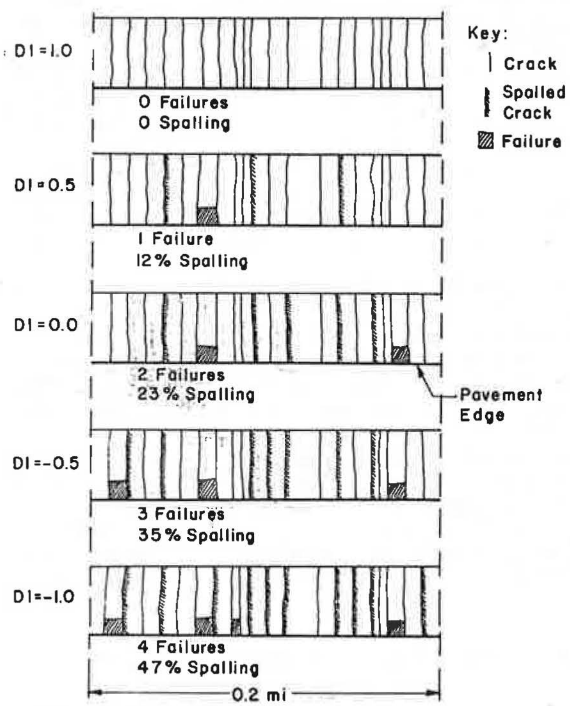
Figure 8 presents information on the average distress index predicted each year within the design period for the network and for the various budget levels. It is apparent from this figure that if a low budget is used (i.e., $5 million/year), the network will continue to deteriorate. The rate of deterioration can be reduced or even reversed, however, if higher budgets are adopted. Also, note the yearly budget (i.e., $10 million) for which the present condition of the network is maintained. This budget level may not be a feasible alternative because of the network's low initial distress condition. The use of a variable budget involves investing an extensive amount of money the first year, about $84 million for the problem in question, to improve the condition of the network, and a yearly budget of about $4 million (lower than the $10 million required if the network is not restored to a better condition) for the rest of the design period. The additional cost incurred by postponing the overlay of a pavement section is given in Table 2.

To help the reader visualize the meaning of the distress index, Figure 9 was produced. A 0.2-mile section is depicted in the figure with several dif-

Table 2. Additional cost incurred by postponing overlay of pavement section, developed from Texas CRCP information.

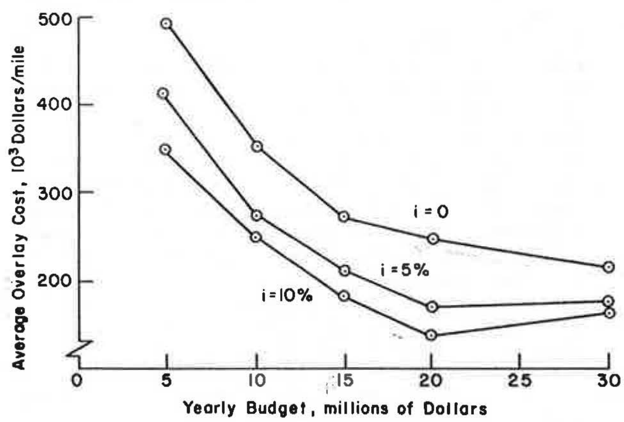
Year of Overlay	Network Average: Cost per Mile ($000s/mile)	Network Average: Increase (%)	Severely Deteriorated Section: Cost per Mile ($000s/mile)	Severely Deteriorated Section: Increase (%)	Slightly Deteriorated Section: Cost per Mile ($000s/mile)	Slightly Deteriorated Section: Increase (%)
1	247.87		478.16		180.37	
2	265.30	7.03	545.20	14.02	182.22	1.03
3	284.65	14.84	624.59	30.62	184.07	2.05
4	306.22	23.54	718.06	50.17	185.93	3.08
5	330.37	33.28	828.06	73.18	187.96	4.21

Figure 9. Sample distress condition of 0.2-mile CRCP section with different values of distress index.



Note : Not All Cracks Shown

Figure 10. Average overlay cost per mile versus different yearly budgets for various interest rates using Texas CRCP information.



ferent stages of distress. Notice the different stages of deterioration that correspond to various magnitudes of the distress index. If a low budget is used, the deterioration of the pavement follows stages similar to the ones presented in Figure 9.

CONCLUSIONS AND RECOMMENDATIONS

The computer program presented was used to analyze the impact on the future distress history of a pavement network of several different budgeting policies. The program was shown to be a useful tool for selecting an adequate budgeting policy. From the analysis conducted the following additional conclusions were drawn; although they may seem obvious, the program corroborates and provides means to estimate them (see Figure 10).

1. A minimum budget is required to maintain the condition of a pavement network. This minimum is variable, depending on the original condition of the network.

2. If the network is allowed to deteriorate, the amount of money required to upgrade its condition will increase with time; that is, more money will be needed to upgrade the network as time goes by.

3. In addition to the availability of funds and personal preferences, an economic analysis is an important factor in the selection of a budget. User costs are not included in the analysis, however, so detailed consideration should be paid to the initial and the predicted distress condition of the network.

The program estimates, in terms of both dollars and distress predictions, should be verified further to corroborate and improve them. As with any PMS, continuous upgrading is required to achieve optimum management of funds. If the rehabilitation scheduling procedure is to include flexible pavements, similar distress indices need to be developed so as to have a common yard stick to measure both types of pavements (i.e., rigid and flexible).

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Description and Evaluation of Alaska's Pavement Rating Procedure

ROBERT L. McHATTIE AND BILLY G. CONNOR

Pavement condition rating methods used on Alaska's roadways since 1978 are described and examined. The methods are intended to provide the specific performance data necessary to optimize construction and maintenance planning and the allocation of available funds. Rating elements include simplified measurements of ride roughness, fatigue (alligator) cracking, patching, and rut depth. These features are reported individually and are also combined with traffic data to indicate more general levels of roadway serviceability. Field evidence shows that a high degree of variability exists in the measurement of cracking, patching, and rutting. Coefficients of variation above 20 percent were estimated for each type of rating element from experimentally repeated measurements. On a given road section estimates of fatigue cracking made by 15 crews differed by up to twice the calculated average. Rut depth measurements were typified by calculated standard deviations of about half the mean value. Report findings suggest that great care be exercised on future pavement performance inventories. Standardization techniques are suggested that should improve manual rating methods. Mechanized or electronic data acquisition techniques must be developed to eliminate human error.

The Alaska Department of Transportation and Public Facilities (ADOTPF) initiated use of newly developed pavement rating procedures during its 1978 highway inventory. The purpose of this study is to evaluate the statistical validity of individual measurements that comprise it. The current Alaskan rating attempts to quantify surface fatigue cracking, patching, and wheelpath rutting as an aid to planning design, construction, and maintenance. The amount of error associated with measurements of pavement distress is examined, and improvements are suggested that can be incorporated into future inventory work.

The research data base used consisted of data and experience accumulated from two complete inventories of the Alaskan paved highway system conducted during 1979-1981. The study also examines results of repetitive sampling conducted specifically for this project on five typical pavement sections located near Fairbanks, Alaska.

DEVELOPING ALASKAN PAVEMENT RATING PHILOSOPHY

During the winter of 1977-1978 the planning division of ADOTPF decided to revise its existing highway inventory procedure to fill the need for accurate, quantitative data for programming highway maintenance and construction funds. The department's research section was commissioned to produce a practical inventory that would stand the scrutiny of statistical evaluation.

As a first step, the literature was researched to see how other states and foreign transportation agencies had negotiated the same ground. A method for rating pavements was first developed for use in the AASHO Road Test of the late 1950s to early 1960s. Pavements are classified numerically based on the subjective observations of engineering specialists and normal highway users (1). The rating scale was arbitrarily set between 0 and 5, where 0 is extremely poor and 5 is perfect. The key distress manifestations selected are surface deterioration, ride roughness, rutting, cracking, and maintenance patching. This rating technique produces a number termed present serviceability rating (PSR) for classifying a given section of road. Figure 1 (2) indicates the number of individual raters necessary, statistically, to estimate the true value of PSR by using the completely subjective AASHO method. This figure indicates that for one or two raters the error associated with estimation of PSR is greater than 1. The error can range ±1 from the true value; therefore, the full range of possible estimation is two, which represents one-third of the total 0 to 5 scale.

The AASHO researchers then took the next logical step of converting the rating from a subjective to an objective method by deriving a regression equation that closely matches PSR panel scores. Independent variables for the regression equation consisted of standardized measurements of fatigue cracking area, maintenance patch area, wheelpath rut depth, and longitudinal surface variation (roughness). The road surface condition values calculated by the regression equation are termed present serviceability index (PSI).

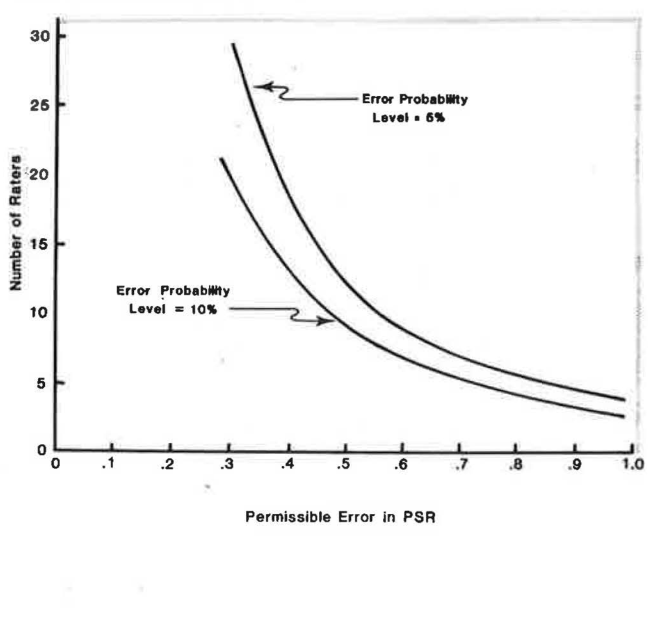
$$PSI = 5.03 - 1.91 \log (1 + SV) - 1.38RD^2 - 0.01 (C + P) \quad (1)$$

where

SV = mean slope variance in the two wheelpaths as measured absolutely by a longitudinal profilometer (in./mile x 10^6),
RD = mean rut depth (in.),
C + P = cracking + patching (ft^2/1,000 ft^2 total surface).

Most pavement rating methods developed subsequent to the AASHO study, including Alaska's, are related in some degree to the original AASHO form and were intended to provide key performance feedback to the overall pavement management process. Generation of Alaska's rating scheme was expedited by a summary and critique of highway agency pavement management practices. A federally sponsored workshop was held in Tumwater, Washington, in November 1977 to examine the existing state of the art in the field of pavement management systems (PMS). United States and

Figure 1. Estimating PSR.



Canadian representatives were invited providing they were actively implementing and, therefore, experienced in a PMS program. At the time ADOTPF was attempting to devise a rating method for asphalt concrete pavements, the Tumwater conference report was by far the most comprehensive source of information concerning rating schemes available (3). The Tumwater report not only discussed various field methods but also compared them critically. Rating system elements were suggested that provided the best input to the overall PMS.

Assuming that PMS would be the ultimately intended use of pavement inventory data, the following consensus emerged from the Tumwater conference:

1. Ride roughness should be rated objectively.
2. Structural capacity should be rated, but whether to rate structural capacity on the basis of deflection tests or surface distress measurements was not clearly decided.
3. Pavement distress should be rated. This includes measurement of rut depth, cracking, and patching.
4. Rut depth measurements were considered along with skid testing to provide an indication of road safety. Rut measurements should, therefore, be included in any highway rating scheme.
5. The use of a single classification number such as PSI was said to provide a valid measure of pavement condition.
6. Little standardization of terminology and measurement technique exists among the available systems of pavement rating when these systems are examined in detail.

Each of the preceding points was considered seriously before development of the Alaskan rating system. Table 1 (3) indicates the salient features of the road rating methods used by the U.S. states and Canadian provinces represented at the Tumwater conference.

The objectives and basic rating elements listed below were chosen by ADOTPF from background research and a definition of departmental needs. They guided the development of Alaska's inventory rating by providing use targets. Only the most commonly recognized pavement condition indicators were selected for consideration as elements in Alaska's rating procedure.

ADOTPF decided that a pavement condition (rating) must

1. Provide information for planning and ordering the priority of rehabilitative design and maintenance of existing pavements,
2. Provide information on the relative condition of total highway mileage within various jurisdictions for budgetary apportionment purposes, and
3. Provide design feedback information.

Table 1. Pavement monitoring features and evaluation.

Agency	Surface Condition	Roughness of Ride	Skid Resistance	Structural Capacity	Rating System	Primary Decision Criteria
Arizona	Crack survey	Mays ridemeter on annual basis	Mu meter-500 ft at each mile post	Dynaflect-3 locations/mile	Pavement management information system	Compares major maintenance alternatives
California	Pavement condition survey based on alphanumeric rating	Ride score not part of pavement distress	Measured periodically		Alphanumeric rating combines severity and extent of defects	Defects compared to repair strategies and costs
Florida	Structural defects of cracking, rutting, and patching	Mays ridemeter correlated with CHLOE profilometer			Combined ride rating and defect rating	Adjusted pavement rating evaluated for priority programming
Kentucky	Used as feedback for design deficiencies	Roughness index correlated to PSI. Use ride-quality meter or GM profilometer		Road rater for specific design evaluation	Correlation of several factors for design input	Input used to develop overlay design
New York		Vehicle response profiler is heart of system			Pavement serviceability system, based on correlation with known serviceability levels	Aimed at identifying budget needs, failed pavements, effectiveness of expenditures
Pennsylvania		Mays ridemeter used to develop serviceability	ASTM skid trailer	Road rater		
Texas	Distress survey based on vehicle-mounted camera-visual distress rating	Mays ridemeter correlated with Surface Dynamics profilometer	Skid trailer	Dynaflect for critical locations	Relative design, ratio of allowable 18K axle loads to those predicted for next 20 years	All highways must carry their traffic safely and comfortably
Utah	Pavement distress based on 11 observed parameters	PCA roadmeter on 1-mile increments	Mu meter, 0.5-mile sections tested every 2 miles	Dynaflect for predicting remaining life	Present serviceability index	Overall priority ranking for preventative rehabilitation
Washington[a]	Pavement condition survey every 2 years covering entire network	PCA roadmeter on all sections	ASTM skid trailer for high accident locations-considered separately	Limited use of Benkelman beam	Combined structural rating and ride score	Tabulate rehabilitation strategies and costs based on pavement condition
Ontario	Pavement condition rating, 1 to 2 year cycle	Subjective riding comfort index		Dynaflect, random sample locations in need of rehabilitation	Subjective, pavement condition rating	Required overlay prediction based on expected performance
Saskatchewan	Annual surface condition rating	PCA roadmeter at intervals of 1 month to 1 year		Benkelman beam data used for overlay design	Condition rating system used to order priority of projects for overlay or scaling	Preventative maintenance is primary goal

[a]Photo logging of entire system.

Literature review plus common sense pointed to the need for a rating method that would characterize the road condition adequately and allow a high degree of reproducibility at a minimum cost. The data must provide true reproducible characterization of pavement condition that changes from year to year in a rational manner (i.e., pavements should not appear to heal anomalously with time unless maintenance has actually been done). The rating technique, therefore, had to be as simple as possible and include the largest practical sampling of each road section.

The following were chosen as rating parameters by Alaskan researchers:

1. Fatigue cracking (alligator cracking),
2. Major patching (at least full lane width),
3. Wheelpath rut depth, and
4. Ride roughness as measured by the Mays ridemeter.

Fatigue cracking was selected as a rating parameter because it is an excellent indicator of structural condition and load-life potential. Design-life vehicle load capacity is said to be reached when significant alligator cracking becomes apparent. Fatigue cracking is also often associated with unacceptable rutting, rough rides for the vehicle, and desintegration of the pavement surface.

Major patching needed to repair a host of problems, including fatigue cracking, embankment settlement, and rutting, gives a general picture of the maintenance effort required on a given road section. Patching is also a principal source of surface roughness and usually becomes cracked and potholed with time.

Wheelpath rutting is generally considered important in terms of driver safety and travel costs. A consensus of available literature indicated that rutting deeper than approximately 0.5 in. is a safety hazard that can cause hydroplaning on wet road surfaces at high vehicle speeds. Rutting also has an effect on vehicle steering and reduces the mechanical life of chassis components. Deep rutting usually accompanies advanced alligator cracking and signifies that pavement structural soil layers (base or subbase) have been loaded beyond capacity. This condition is aggravated through use of materials subject to extensive moisture-related softening (thaw weakening).

Ride roughness is measured because it is the characteristic of pavement that is of primary concern to the driving public. The combination of differential settlement and leveling patches is common to all parts of Alaska and is the major cause of roughness felt by the driving public. Ride roughness is measured objectively on a continuous basis by using available technology such as the Mays ridemeter.

Some recognized surface distress features were disregarded in order to simplify the rating process. These include raveling, longitudinal cracks, thermal cracking, shoving and bleeding, potholes, and deflection. Statewide skid measurements in 1975 indicated that the materials used in Alaskan road-building provided consistently high skid numbers. Reasons for this include a high degree of aggregate hardness and limited potential for asphalt bleeding because of Alaska's relatively cool air temperatures.

Testing of deflection statewide will ultimately become part of the normal inventory process. This process began in 1982 and will require approximately five years per statewide cycle. The falling weight deflectometer is currently being used to collect inventory data.

RATING AND SCORING PROCEDURES USED SINCE 1978

A discussion of pavement rating methods used by ADOTPF since 1978 and an explanation of how field data are manipulated for purposes of scoring and reporting follow. Figure 2 illustrates the manner in which raw field data are transformed into a useful pavement inventory report.

Development of Field Methods

The rating process is done as two separate operations, each requiring the use of a two-person crew. Phase 1 consists of measurement of ride quality. The Mays ridemeter trailer is currently being used because of the relatively low cost of gathering data and its reasonably good repeatability. The trailer-mounted meter provides a standardized vehicle, suspension, and tire type. In phase 2 the surface distress features are measured. These include alligator cracking, full-lane patching, and rut depths.

The Mays ridemeter can provide a continuous sampling of highway roughness at 50 mph automatically. Studies of the repeatability of this test have been made by others and are beyond the scope of this report. The objective nature of ridemeter measurements suggests that they be considered a relatively reliable element of the current pavement inventory.

Methods for measuring alligator cracking and major patching were evaluated initially on seven sections of roadway near Fairbanks. Each section was divided into 0.1-mile subsections that were rated independently. Full-width patching was characterized on the basis of total length (density); fatigue cracking was typified by both density and severity. A type-1 or type-2 classification was adopted for cracking of lesser or greater severity. Alligator cracking was defined as cracking that is visible while driving 7 to 10 mph. It is measured as the total percentage of the road section length that exhibits cracking, regardless of wheelpath location. Histograms were constructed from field data (Figure 3) to show the frequency distribution of fatigue cracking for the subsections within each mile. The distribution of cracking is strongly polymodal (showing no single mean value) and bounded to both the 0 and 100 percent occurrence level. Distributions of fatigue cracking are obviously non-Gaussion in character. Based on these data Table 2 gives the probability of a random selection of a 0.1-mile sample that will predict the true mean condition of each section of roadway. The probability is obviously small in all cases. In view of these data, a 1-, 2-, 3-mile or more length of paved road could not be rated accurately for fatigue cracking based on measurements in a randomly selected subsection several hundred feet long. The normal assumption of a 10 to 20 percent sampling density is of no value in this case. Fatigue cracking, therefore, must be measured by continuous observation through each mile of roadway. All data collected subsequent to the initial trial have supported this decision.

Full-width patching was observed to have a distribution of occurrence similar to that of fatigue cracking and it was also decided that this feature could be properly characterized only by continuous observation.

The frequency of rut depth measurement was also examined briefly before development of the rating method through multiple readings taken on each of eight 1-mile-long pavement sections near Fairbanks. Rut depth averages ranged from 0.185 to 0.244 in. The standard deviations of the sample ranged between 16 and 35 percent of the sample means and the plotted frequency distributions of rut depth measurements appeared reasonably indicative of normal

(Gaussian) behavior. Based on these trials the assumption was made that rut depth measurement could be evaluated by normal statistical techniques. Sampling frequency was addressed through the statistical method used for estimating a true mean value from a small sampling. An estimation of true population average is given by

$$\mu_o = \bar{X} \pm ST\sqrt{N} \quad (2)$$

where

μ_o = true population average (i.e., true average rut depth),
X = average rut depth as determined from sample,
S = standard deviation of sample,
N = number of measurements constituting the sample, and
t = Student's t-value for a given confidence level and N.

Figure 2. Elements of Alaskan pavement inventory.

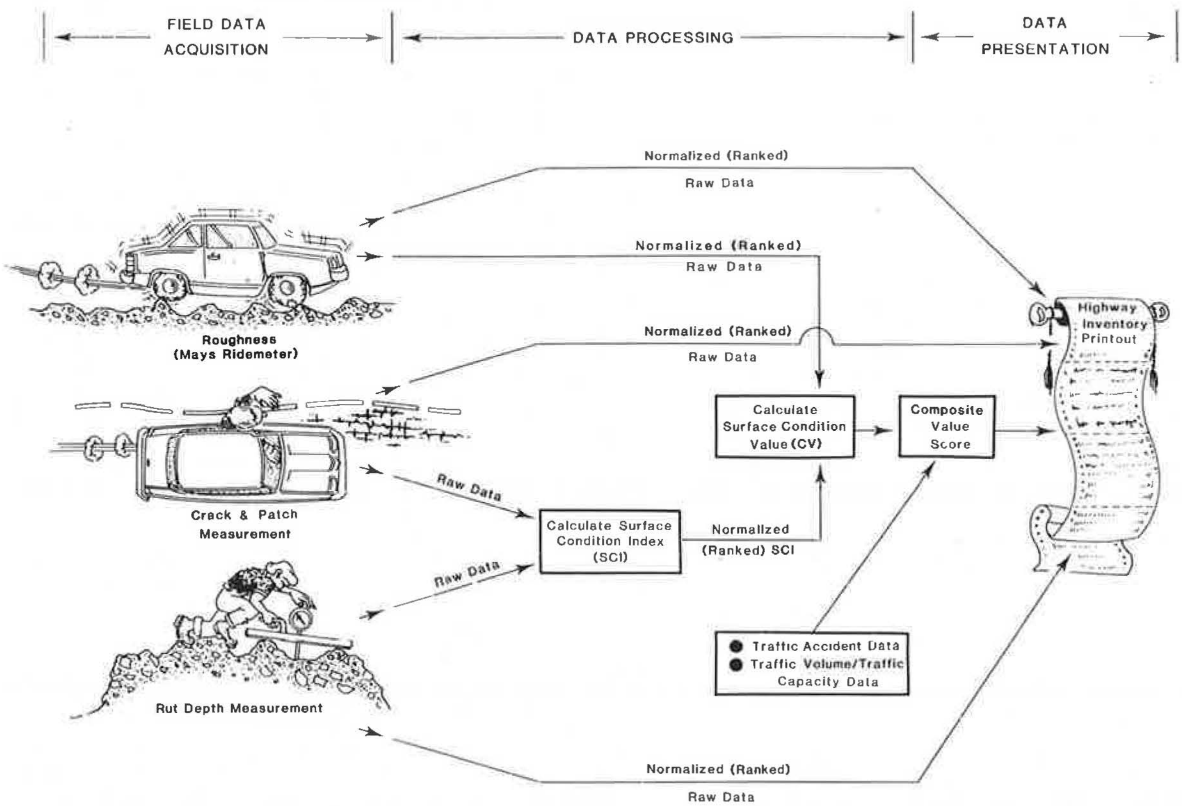
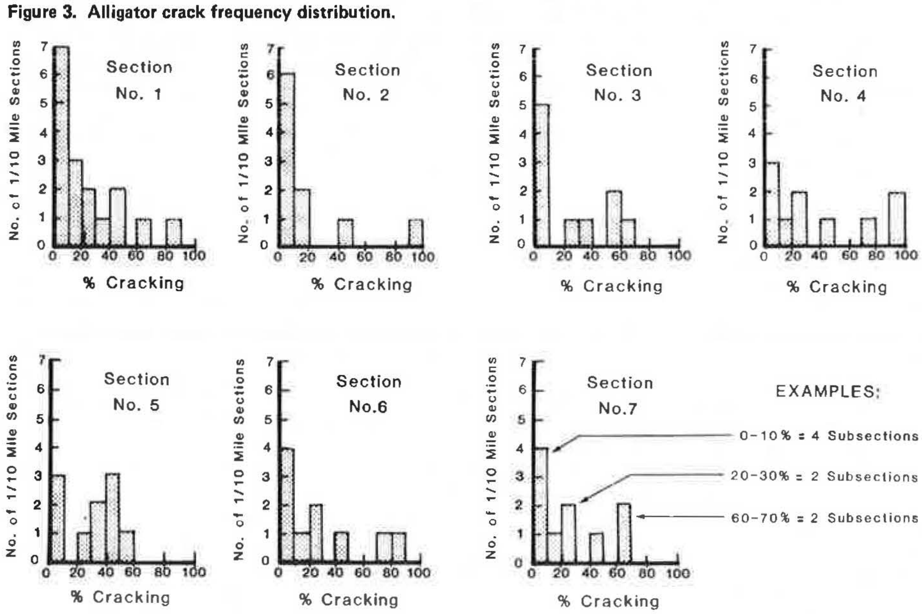


Figure 3. Alligator crack frequency distribution.



Note: Figure shows variation in amount of cracking in subsection measurements within seven sections. Except for section 1, all were 1 mile long and included 10-1/10-mile subsections. Section 1 was 1.7 miles long and included 17-1/10-mile subsections.

This equation is an expression of the central limit theorem, which describes the distribution of sample means about a true mean. In modified form the equation can be expressed as follows:

$$(\mu_o - \overline{X})/S = T\sqrt{N} \quad (3)$$

The error in estimating true rut depth average (i.e., $\mu_o - \overline{X}$) is small in relation to the sample standard deviation (at a given level of confidence) when the term $T/\sqrt{N}$ is minimized. Figure 4 is a plot of N versus $T/\sqrt{N}$ used to select sampling frequency for the initial inventory runs in 1978. Flattening of the curve beginning between N = 4 and N = 7 suggested that a sampling of at least four locations would be necessary to ensure that the error in estimating true mean rut depth would be less than 2 standard deviations of the sample. Figure 4 indicates that the error of estimating true mean rut depth is about 1.6 x S for N = 4. Because S of the trial road sections averaged approximately 0.05 in., errors in estimating rut depth during inventory work would be expected to be no larger than ±1.6 x 0.05 (i.e., 0.08 in.). This accuracy was considered good enough for beginning the pavement inventory process. Fewer than 4 readings per mile were required in the 1978 rating method if rutting was generally observed to be less than 0.25 in.

Summary of Required Measurement Frequencies

On the basis of limited field trials it was decided that pavement distress, except for rut depth measurement, should be characterized by continuous observation of the entire road. Three field seasons of field data collection have reinforced the idea of using 100 percent sampling.

The frequency of measurements necessary to determine average rut depth adequately was calculated from a preliminary statistical assessment. Measurement of ruts was known to be a disproportionately time-consuming job when compared with other distress observations. The hope was that the experience in accumulation would show that no more than four sets of readings would be required per mile of road.

Table 2. Proability of sampling true mean performance.

Section	Percentage of Total Area Cracked	Range Acceptable From	To	Probability of Selecting 0.1-Mile Section with Correct Percentage of Crack
1	23	13	33	0.29
2	20	10	30	0.20
3	24	14	34	0.10
4	39	29	49	0.10
5	31	21	41	0.20
6	28	18	38	0.30
7	27	17	37	0.20

Road Condition Scoring Using Alaska's Pavement Rating

Alaska uses its pavement inventory data to construct a mile-by-mile summary report listing individual condition scores (percentage of cracking, rut depth, percentage patching, and ride roughness) and also a combined condition value (CV) score. The CV is analogous to the AASHO PSI and provides a single numerical descriptor of a given road section.

The CV is calculated from the inventory data in the following way:

$$\text{CV} = [\text{Mays ridemeter score (ranked)} + \text{Surface condition index (ranked)}] \div 2.0 \quad (4)$$

Ranked data indicate that the data have been transformed mathematically into a percent-worse-than score before calculation of CV. This provides a normalizing of raw scores on a 0 to 100 (worst to best) scale.

$$\text{Percentage worse than} = [(1/2E + L)/N] \times 100 \quad (5)$$

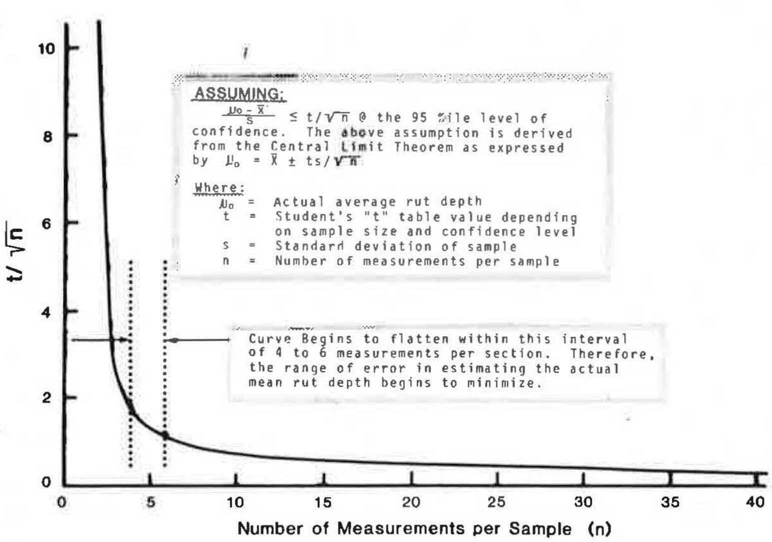
where

E = number of statewide sections rated the same,
L = number of statewide sections rated worse, and
N = total number of statewide sections rated.

The Mays ridemeter score shown in Equation 4 is derived directly through the percentile ranking equation from raw Mays ridemeter data. Surface condition index (SCI) is calculated by means of Equation 6 and then transformed to a percentage-worse-than ranking through the ranking formula.

$$\text{SCI} = 1.38R^2 + 0.01(A+P) \quad (6)$$

Figure 4. Plot of relative error factor ($t/\sqrt{n}$) versus number in sample.



where

R = average rut depth (in.),
A = percentage of road section that is alligator cracked, and
P = percentage of road section that is covered by full-width patching.

In addition to reporting a summary of the previously discussed information, the pavement inventory report also includes, for multiple mile sections, the ranked scores of a volume/capacity ratio and the section's accident rating value. Finally, the condition value plus capacity and accident scores are combined in the form of a geometric mean to produce a composite value calculated as

$$\text{Composite value score} = [\text{Condition value} \times \text{Capacity (ranked score)} \times \text{Accidents (ranked score)}]^{1/3} \quad (7)$$

This composite score, like CV, is used mostly for generalized administrative planning and programming purposes. By combining the three parameters in this manner the lowest value may affect the calculation significantly. Thus, if any of the three values has a very low score, it caused that mile to be flagged. Figure 5 is a sample page from the 1979 inventory summary.

REVIEW OF ALASKA'S PAVEMENT RATING METHODS BASED ON RECENT FIELD STUDIES

After the Alaskan pavement rating method had been in use for 2 years a more detailed evaluation of its constituent measurements was thought necessary. A field study was begun in 1980 to investigate the repeatability of cracking and patching measurements made by different rating crews. Frequency of measurements necessary to estimate a true mean rut depth was also reviewed.

Method of Study and Data Acquisition

Five roadway sections were selected near Fairbanks that reflect the average range of road surface conditions commonly encountered. Each of the sections was rated by 15 different two-member crews using the current standard Alaskan procedure. Members were drawn mostly from the middle-level professional and technical ranks of road design, maintenance, right-of-way, and materials sections, but only four raters had previous pavement rating experience. Raters with previous experience were drawn from the department's research and development section.

Each crew of raters was given the same introduction to pavement rating and directed from one pavement section to another by the instructor. Ratings by each crew required a full day and the sequence of pavement sections remained constant throughout the duration of the study. Considered important was that the sequence of sections not change because this assured that the sun angle relative to the viewer remained consistent for each crew for each section. Sun illumination was known through accumulated field experience to greatly affect pavement crack visibility. To maximize the observational abilities of each rating crew all ratings were performed from a light truck or van. A nearly vertical windshield combined with a relatively high seating position allowed the most advantageous view of pavement surface of any standard type of vehicle. Each section was inspected at under 10 mph in order to identify and measure cracking. Rut depths were measured in each of the four wheelpaths every 0.2 mile. Distances were measured with an electronic odometer capable of 1-ft resolution.

Analysis of Field Data

An indication of measurement variabilities between crews is given through the coefficient of variation (C_v) associated with each distress type:

$$C_v = (\text{SD/Mean value}) \times 100 \quad (8)$$

where SD is the standard deviation.

In general, a small C_v of approximately 5 to 10 percent indicates that a good estimate of a true mean value is possible from relatively few individual measurements. C_v values associated with measurement of all pavement distress indicators were considered very high. This tends to contradict the initial hypothesis that, because of the simplicity of the rating method, reproducibility of ratings among crews could be taken for granted. The following estimates of C_v were calculated from project data.

Item	Avg. C_v (%)
Type 1 alligatoring	43
Rut depth, calculated average	25
Rut depth, calculated standard deviation	40

The significance of the foregoing listing should not be understated as the uniformity of C_v from section to section indicated.

Type 2 (severe) alligator cracking and full-width patching are not listed because their infrequent occurrence within the test sections did not provide an adequate sampling to allow a good evaluation of differences among rating crews. Based on these limited observations, however, the variability in measuring patching length is somewhat lower than for alligator cracking that has C_v of perhaps 10 to 20 percent. A clear distinction between type 1 and type 2 alligator cracking was not easily made by the rating crews. The tendency, except in the most obviously severe cases, was to place all cracking into the type 1 category. Most crews apparently selected a lower severity classification whenever the question of degree of damage arose. This problem can probably be remedied to some extent during the instruction process by specifically advising that pavements be rated critically.

The large amount of variability observed in the collected data is given in Table 3. In view of the similarity in training and background among these experimental raters and previous inventory crews, these variabilities could be expected on pavement sections throughout the state.

Table 3 summarizes the variation in data for the sections tested. The variation in all the pavement distress measurements is large when considering the range in cracked length. When considering the variation as a percentage of section length, the variation is less. The maximum variation between the mean and maximum values is 7 percent. It can be argued that the alligator cracking expressed as a percentage of section length need only be determined to be within 10 percent of the true percentage for inventory purposes. If it is assumed that the mean is the true value, then all five sections meet this criterion. More detailed measurements may be necessary for design processes.

The overall effect of variations in crew measurements on determinations of rut depth is magnified because ADOTPF usually reports maximum rut depth in terms of average plus two standard deviations. For example, the mean, mean + 1 standard deviation, and mean + 2 standard deviations are given for the fol-

Figure 5. Sample pavement inventory.

LOCATION			CONDITION ELEMENTS					PERFORMANCE VALUES			
TERMINI	SECTION LENGTH	CDS MILE	ADT	RIDE (in/mi)	CRACKING (%/mi)	PATCHING (%/mi)	RUTTING (in/1000)	CONDITION VALUE	SERVICE VALUE	ACCIDENT VALUE	COMPOSITE VALUE
FAP 35 Parks Highway		312									
(State Route 170000)		313		24	0	0	72 ✲	85			
		314		63	5	0	83 ✲	52			
JCT Old Nenana Hwy. Ester JCT	20	315		35	0	0	62 ✲	82			
SECTION AVERAGES			806	29	1	1	87	79	82	74	78
		316		132	41	15	137	16			
		317		154	16	29	119	15			
		318		129	4	14	188	26			
		319		166	27	21	169	13			
JCT FAS 649 GEIST ROAD	5	320		88	0	36	29	29			
SECTION AVERAGES			2362	134	18	23	128	20	1	60	11
JCT FAU AIRPORT SPUR	1	321		61	0	1	65	67			
FAP 35 PARKS HIGHWAY (STATE ROUTE 170000) SECTION AVERAGES			4090	61	0	1	65	67	60	72	66
		322		56	0	1	116	63			
		323		90	0	3	220	39			
		324		112	0	0	138	49			
JCT STEESE AND RICHARDSON HWYS	4	325		60	0	0	340	45			
SECTION AVERAGES			17919	80	0	1	204	49		42.6	

Table 3. Observed variation in pavement distress measurements.

Pavement	Section No.	Range	Average	Range as Percentage of Section Length	Avg as Percentage of Section Length
Type 1, alligator cracking	1	29–187 ft	140 ft	1–4	3
	2	35–1,434 ft	820 ft	0–14	8
	3	69–700 ft	300 ft	1–13	6
	4	54–218 ft	120 ft	1–4	2
	5	190–505 ft	340 ft	4–10	6
Type 2, alligator cracking	1	None detected	0 ft	0	0
	2	4–19 ft	0 ft	0	0
	3	0–13 ft	0 ft	0	0
	4	None detected	0 ft	0	0
	5	None detected	0 ft	0	0
Full-width patching	1	350–382 ft	360 ft	7–7.5	
	2	439–1,042 ft	820 ft	8–20	
	3	91 197 ft	100 ft	?–4	
	4	None detected	0 ft	0	
	5	None detected	0 ft	0	
Rut depth, average inner wheelpath	1	0.016–0.059 in.	0.040 in.		
	2	0.110–0.393 in.	0.210 in.		
	3	0.114–0.289 in.	0.180 in.		
	4	0.100–0.257 in.	0.170 in.		
	5	0.134–0.271 in.	0.210 in.		
Rut depth, SD inner wheelpath	1	0.005–0.055 in.	0.020 in.		
	2	0.051–0.601 in.	0.160 in.		
	3	0.040–0.198 in.	0.080 in.		
	4	0.023–0.263 in.	0.080 in.		
	5	0.060–0.241 in.	0.110 in.		
Rut depth, average outer wheelpath	1	0.050–0.167 in.	0.090 in.		
	2	0.116–0.410 in.	0.240 in.		
	3	0.089–0.248 in.	0.150 in.		
	4	0.062–0.272 in.	0.180 in.		
	5	0.172–0.445 in.	0.270 in.		
Rut depth, SD outer wheelpath	1	0.022–0.105 in.	0.055 in.		
	2	0.069–0.596 in.	0.240 in.		
	3	0.040–0.322 in.	0.090 in.		
	4	0.032–0.184 in.	0.080 in.		
	5	0.098–0.257 in.	0.160 in.		

lowing sections by using Table 3 outer wheelpath data.

Section	Mean (in.)	Mean + SD (in.)	Mean + 2 SD (in.)
1	0.090	0.145	0.200
2	0.240	0.480	0.720
3	0.150	0.240	0.330
4	0.180	0.260	0.340

The foregoing examples demonstrate a wide range of uncertainty as to the measured depth of rutting even though calculated mean values are low.

Discussion of Measurements of Alligator Cracking

In several of the following figures the variations in measurements have been normalized. This normalization step is used so that various road sections can be compared directly even though each has a different mean rut depth or length of alligator cracking. Normalization of scoring, (e.g., percent of alligator cracking and average rut depth) is accomplished as follows:

$$\text{Normalized percentage of alligator cracking} = (A-B)/C \qquad (9)$$

where

A = percentage of alligator cracking as measured by an individual crew on a specific road section,
B = average percentage of alligator cracking calculated from the measurements of all crews on the above section, and
C = standard deviation value calculated from the measurements of all crews on the above section.

Figure 6 shows how normalized scores of individual crews rank in relation to calculated average values on all five pavement sections. This plot indicates the ability of certain crews (e.g., 7 and 8) to see more damage than others. Conversely, crew 14 saw much less cracking in all five pavement sections than the calculated average. Figure 6 includes the instructor's subjective assessment of each crew in terms of (a) communication between crew members [rated low (L), moderate (M), and high (H)] and (b) initial impression of rating ability (rated fair, good, and expert). Note that crews 2 and 10, rated expert by the instructor, had at least a full season's rating experience and were included for purposes of comparison with the other crews. Although Figure 6 indicates that some crews could apparently see more pavement damage than others, this difference was not accounted for in obvious attitudes or abilities. Note, however, that crew 8, which saw much more pavement damage than crew 14, also rated higher in the instructor's opinion. Best results are obtained when conversation concerning the rating process is encouraged between crew members, especially during the first few days of inventory.

The data in Table 4 attempt to delineate reasons for differences among crew ratings. The samples have been broken down into a stratified format and cross indexed in terms of crew communications and weather and pavement surface condition at the time of rating. The numbers given in Table 4 as $\bar{X}$ (characteristic sample average) and SD (characteristic sample standard deviation) have been normalized, as

Figure 6. Range of variation in each crew's measurement of type 1 alligator cracking.

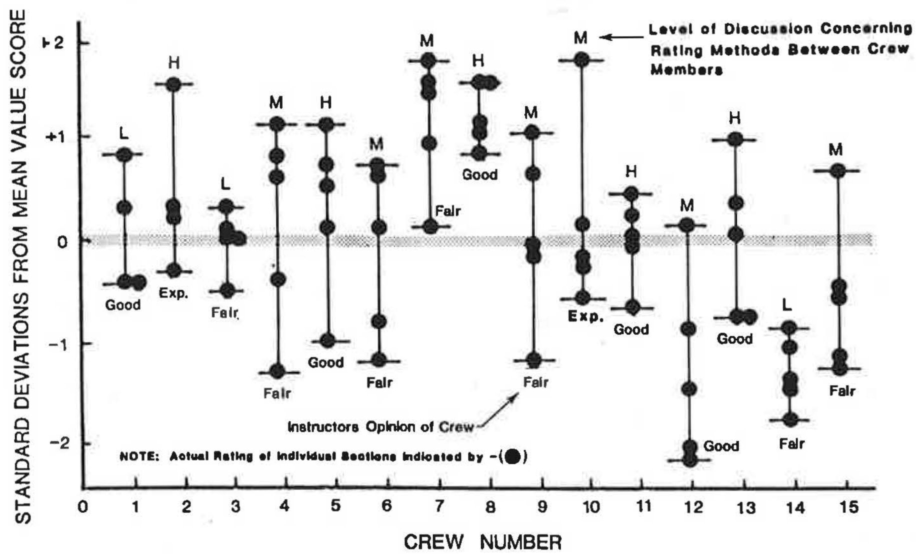


Table 4. Analysis of type 1 alligator cracking.

Discussion of Rating Methods	Statistic	Sunny	Cloudy	Cloudy, Slightly Wet Road Surface	Rain, Very Wet Road Surface	Weighted Avg of Rows
Active	$\bar{X}$	-0.1	0.5	0.8	0.2	0.4
	SD	0.5	0.7	0.8	0.9	0.7
	N	7	5	8	4	
Moderate	$\bar{X}$	0	0.3	-0.3	-1.7	-0.1
	SD	0.8	1.1	0.6	0.5	0.9
	N	7	19	5	4	
Little	$\bar{X}$	0.1	-0.7			
	SD	0.6	0.8	No samples	No samples	0.5
	N	4	10			0.7
Weighted avg of columns	$\bar{X}$	0	0	0.4	0.8	
	SD	0.6	1.0	0.7	0.7	

previously described, thus allowing all five pavement sections to be considered in the same analysis. The combination of a slightly wet (SW) pavement surface and a highly communicative crew resulted in more visible cracking and a characteristic average of +0.8 SD above the overall sample average. Also, in examining the weighted (for sample number) averages of both rows and columns, good crew communication and a slightly wet road surface are individually associated with increased damage observation.

Surface Wetness

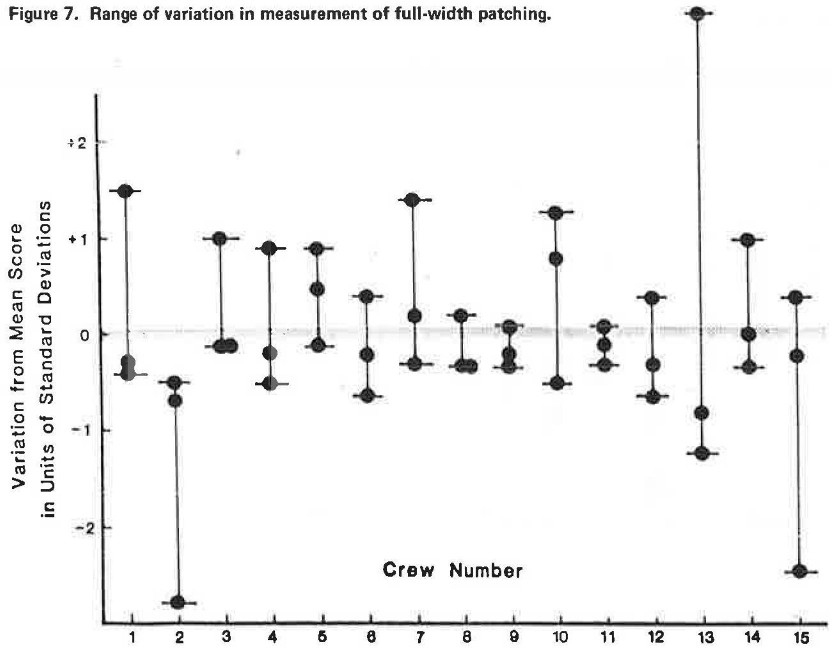
The effect of a slightly wet surface in optimizing the visibility of alligator cracking is fairly obvious to even the casual observer and can often cause hairline alligator cracking to stand out in vivid detail. On the other hand a very wet road surface, such as obtained during or shortly after a rainstorm, camouflages all but severe cracking. Observations of cracking should be discontinued during rainstorms or other periods when the pavement surface is covered by free water. Table 4 generally associates the least observed cracking with a very wet (VW) surface condition. The ideal, slightly wet surface condition is created when the road surface is dry except in and around individual cracks. In this case, water stored in the cracks during rainfall will keep the adjacent pavement wet longer than in areas of no cracking.

The previous discussion leads to the conclusion that pavement ratings could be done best shortly after a rainstorm; however, a dry road condition represents the more normally encountered situation. Because of the need for a standard rating procedure crack measurements should be made only on dry pavement.

Sun Angle

Illumination effects due to variations in vertical and horizontal sun angle are known to affect crack visibility strongly. Experience indicates that optimal lighting conditions are provided by a more or less head-on sun incidence. Frontal light tends to shade and, therefore, darken the visible side of crack segments that are perpendicular to the observer and most easily viewed. This has the net effect of maximizing apparent tone and texture differences between cracked and uncracked pavement. The travel direction chosen for the experimental ratings produced over-the-shoulder lighting on four of the five pavement sections, which is usually considered a worst-case viewing condition. Each test section was examined at approximately the same time of day by each crew to ensure a consistent sun angle.

Figure 7. Range of variation in measurement of full-width patching.



Discussion of Full-Width Patching Measurements

The occurrence of full-width patching within the test sections was somewhat limited. Data from section 3 indicate that differences in patching measurements among crews may be about half those expected from observations of cracking. The distribution of normalized scores indicated in Figure 7 represents only the three test sections that actually contained patching. The variation among crews is markedly less pronounced than for alligator cracking.

Patching appears to be more easily measured than alligator cracking even though both are evaluated in a similar way. In most cases patching, at least new patching, is actually seen quite easily. Observation conditions that provide the best view of alligator cracking also tend to make patched areas stand out. Again, very wet surfaced roads resulted in the most variable measurements among crews, and cracks are most easily seen on a slightly wet pavement. Regardless of the better viewing condition afforded a slightly wet surface, the dry road condition is most commonly encountered in field work and is, therefore, suggested as the standard for inventory purposes.

Discussion of Rut Depth Measurement

The approach taken initially to determine a sample number (as indicated in Figure 4) was a rough attempt to limit the possibility of gross errors. Sufficient field data have since been collected to allow a more valid estimation of rut depth. The problem of rut depth measurement can be addressed by normal statistical methods. The principal questions asked are

1. How frequently must rut depth measurements be taken? and
2. Must rut depth measurements be taken in both inner and outer wheelpaths?

Sampling Frequency

The frequency of sampling must be high enough to ensure (to some specified confidence level) that a calculated mean rut depth is reasonably close to the actual mean rut depth. Actual or population average in this case is that value that would be measured from an infinitely large sampling. Sampling tables available in references such as the Chemical Rubber Company statistical handbook (4) indicate minimum sample numbers necessary to attain specific levels of confidence against either a type-1 or type-2 error being committed. A type-1 error occurs if statistical calculations indicate that the sample mean is not representative of the population mean, when in fact it is. Conversely, a type-2 error occurs when statistics indicate that the sample mean is representative of a population mean when it is not.

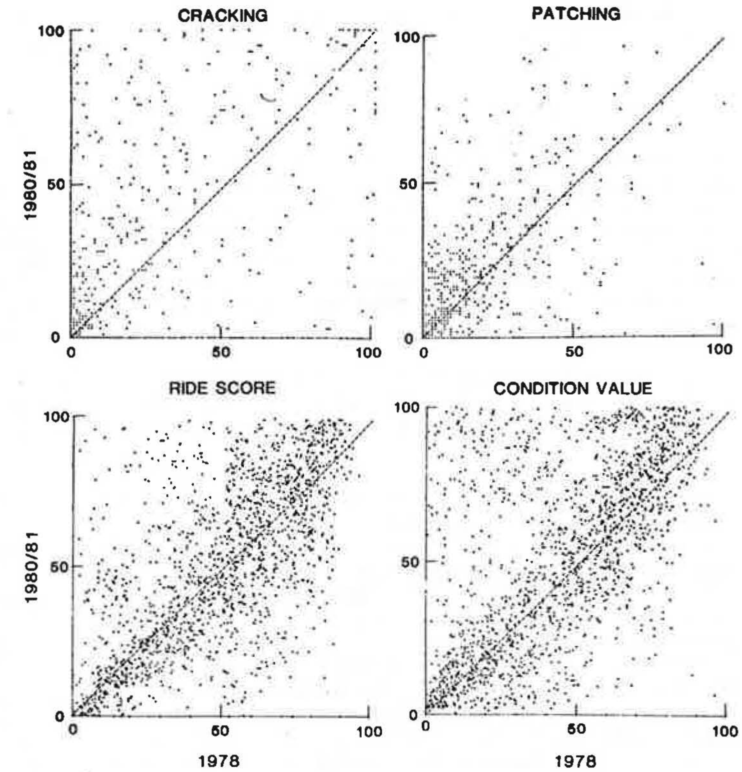
It was assumed that, for predicting the actual rut depth average from sample data, an error of no more than ±0.05 in. would be allowable. In most sampling situations, little concern is expressed over type-2 errors. This philosophy leads to 50 percent level of type-2 error control (i.e., no control and a significantly reduced sample size).

Determination of sample size is dependent on expected standard deviation; therefore, it is important to consider the magnitude of values that might commonly be encountered. Rut measurements made on the five Fairbanks test sections indicated a range of standard deviations from about 0.02 to more than 0.35 in. associated with average rut depths between 0.02 and 0.40 in. Indications of variability in rut measurement derived from the Fairbanks test section data suggest that minimum sampling be based on a standard deviation perhaps as high as 0.30 to 0.35 in. This magnitude of deviation plus 90 to 95 percent confidence level against error results in a minimum sample size in excess of 100. Rut depth measurement, therefore, begins to appear impossible except through an automatic rut-measuring device capable of high-density sampling.

Alternatives

Several sources of rut measurement data were used to construct functional relationships among average rut depth, calculated standard deviation, and required number of sampling points. This report substantiates previous contentions (5) that true mean rut depth can be accurately characterized only through a

Figure 8. Comparison of 1978 with 1980/81 pavement inventory data.



Note: All numbers have been normalized to a 0-100 (worst-best) scoring system

large sampling. Problem rut depths on the order of 0.4 to 0.5 in. or larger would require an assumed standard deviation of at least 0.3 in. Reasonable error confidence levels indicate a sampling obviously greater than 100/section. Furthermore, the inability to predict whether inner or outer wheelpath represents the worst-case condition would require doubling of the sampling effort. In dealing with this question the choices are

1. Assume rutting to not be a problem and cease measurement,
2. Perform a few random measurements per mile at locations that appear from general observation to represent worst-case conditions, or
3. Purchase or build an automatic device for rut measurement as described by Jurick (5).

Deeply rutted sections are usually associated with severe alligator cracking on most Alaskan road sections; therefore the measurement of both is unnecessary. Rutting within the state is rarely as deep as 0.5 in., which is considered critical in most literature sources. Alternative 1 appears to be a reasonable course of action at present. Alternative 2 provides numbers and the numbers can, of course, be included in subsequent discussions of pavement condition. The numbers generated from alternative 2 have no basic statistical validity, however, and might be thought of as inventory garbage. Alternative 3 is preferred if departmental policy requires an accurate determination of rut depth. A 1981 cost estimate for the purchase of an automatic rut-measuring device was $150,000 to $200,000.

COMPARATIVE LOOK AT PREVIOUS INVENTORY DATA

This section looks at actual pavement inventory data in view of the findings of this paper. Because of the rather gross variability evident in the experimental measurement of cracking and patching, a direct mile-by-mile comparison between two previous inventories is made.

Figure 8 shows the apparent variation in pavement distress between 1978 and 1980/81. As shown, these data have been normalized to provide a total scoring range of 0 to 100 (worst-best). Note that data at coordinates (0 percent, 0 percent) and (100 percent, 100 percent) are often clustered in the graphs for cracking and patching, which accounts for the appearance of fewer than expected individual points on these plots. The same number of data points was recorded for all of the graphs.

A line of x = y has been included in each plot and differentiates pavement sections that apparently or actually improved with time (points above the line) from those that became worse (points below the line). Examination of plotted data indicates

1. A very high degree of overall scatter and
2. An unusually large number of data points above the line of x = y (i.e., performance improvement with time).

Taken together, these findings demonstrate a marked degree of randomness inherent in the rating process. The implication of point number two is especially significant in view of the common-sense assumption that pavement condition deteriorates with time. This assumed generality could, of course, be altered by reconstruction, overlay, or careful patching, and no attempt was made to remove specific points that represent reconditioned pavement sections from the plots. This should, however, account for only a small percentage of total rated mileage. A significant degree of randomness is suggested because even sections that scored better than average in 1978 showed a very high rate of apparent improvement with time. The likelihood that initially good pavements (scoring 50 to 100) will be improved substantially within a period of 3 years through maintenance is slight.

SUMMARY AND CONCLUSIONS

The development and evaluation of Alaska's inventory rating procedure for flexible pavements has been described. Development of the system was based on the generally accepted principles of pavement rating practice as outlined in recent literature. The Alaskan rating method attempts to measure basic elements of road quality from two important viewpoints:

1. The highway user--ride roughness and
2. The highway engineer--fatigue (alligator) cracking, major (full lane width) patching, and wheelpath rutting.

These rating features are reported on a mile-by-mile summary both individually and in terms of a composite serviceability score. A concerted effort was made during the development of the rating method to keep all distress measurements as simple as possible but still provide adequate information for pavement management needs.

The rating method was evaluated through a special field study and experience accumulated during the 3 years since its implementation. Findings indicate a large variation in the abilities of different rating crews to characterize the extent and severity of patching and cracking. The range of variation in crack and patch measurements obtained by 15 crews on 5 selected pavement sections was found to be as much as twice the mean measured value. These differences are apparently associated with the level of interest in the task expressed by each crew and weather factors that control visibility of pavement surface features. Examination of previous inventory ratings confirmed the data scatter indicated by the experimental pavement sections.

The variation in rut depth measurements was large enough to require very high sampling frequencies. A mechanized form of rut-measuring device, capable of more than 100 measurements per section in both inner and outer wheelpaths, is suggested. Marked differences between average depths of inner and outer wheelpaths require data from both locations in order to define the worst-case condition.

Conclusions

The assumption that Alaska's pavement rating methods are simple enough to ensure a high degree of reproducibility is not demonstrated by the available data. A great deal of variation is apparent in the field measurement of cracking, patching, and rutting. This is indicated through examination of experimental data as well as from data collected from previous inventory work. The use of machine measurements is suggested wherever possible in all phases of the rating process.

The visual rating of pavements is a difficult process that requires careful and rigorous standardized technique. Pavement rating instructions must be formalized to include guidelines for training rating crews and ensuring acceptable performance. Specifications are necessary for standardization of viewing height, acceptable lighting conditions, and vehicle speed.

Recommendations

The ability to quantify pavement performance is a requirement of almost any approach to pavement management. Alaska's pavement rating method should therefore be viewed as a tool to be improved rather than discarded.

Recommendations for improvement include

1. Phase out visual measurements of pavement distress as reliable machine methods become available;
2. Except for very rough classification purposes, discontinue rut measurements until sampling rates of more than 100/mile can be achieved; and
3. Continue existing approach but with greatly increased and improved crew training and a strict standardization of observation technique.

An ideal form of instruction would include the use of standard road sections. On these sections the crew would attempt to match the ratings assigned by experienced personnel. A five-day tuning period is suggested for new rating crews. Ratings performed during this first week would not be included in the inventory summary before verification by repeated observation.

Observation conditions for the inventory measurement of cracking and patching should be standardized:

1. Vehicle speed of 6 mph or less,
2. Rating of only completely dry road surfaces,
3. Use of optimal sun incidence whenever possible for best illumination--a horizontal sun angle of ±70 degrees from head-on or a vertical sun angle of more than 10 and less than 60 degrees from the horizontal (this point should be emphasized even if it requires that the direction of travel, i.e., direction of the rater's view, be changed),
4. Standardized viewing height of 5.5 ft ±0.5 ft, and
5. Use of utility van-type vehicle that has a nearly vertical windshield.

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Windshield Surveys of Highway Condition: A Feasible Input to Pavement Management

DAVID T. HARTGEN AND JOHN J. SHUFON

The procedure used by the New York State Department of Transportation to collect highway condition data by using an in-motion windshield survey is described. The windshield survey is performed by road rating teams from the department's regional offices. Cost is about $50,000 for the 16,000-mile system or about $3.12/mile. Rating is done with carefully developed photograph scales in which photographs show not specific distress signals but rather general impressions of roads at various condition levels. Periodic training ensures consistency in assessing highway condition, and this decentralized approach permits a rapid data collection effort at a low cost to the agency. Also presented are the many uses of these data in the state's pavement management activity, both as a network-level condition-assessment process and as a screening process to identify sections of highway that require further engineering analysis. The conclusion is that windshield surveys conducted in accordance with the outlined rating methods can provide pavement managers with a current and reliable assessment of network-level highway condition and point to possible problem sections that require more detailed analysis. Low costs, speed of delivery of data, and avoidance of expensive measuring devices are also significant advantages of the method.

The purpose of pavement management is to protect the capital investment in the highway system and to ensure maximum serviceability to the motoring public at reasonable cost. Pavement management involves planning, design, construction, maintenance, and periodic evaluation of pavement structures. The pavement management process involves comparison of investment alternatives at both the network and project levels, coordination of the various activities of the highway agency, and the efficient use of existing information and methodology.

The New York State Department of Transportation (NYSDOT) is taking a number of steps to improve its pavement management practices. A pavement management task force is reviewing department practices and procedures in pavement evaluation as well as the information needed for sound management. Methods are also being developed to predict network-level condition and funding needs for alternative rehabilitation strategies. Considerable work has also been undertaken to improve and streamline various condition rating procedures. This paper reviews progress in this last subject; another paper (1) describes the condition prediction model. The purpose of this paper is to describe New York's current windshield condition survey, the various uses of the data, and the improvements that were made to the scoring procedures for the 1981 and 1982 effort.

OVERVIEW OF CONDITION SURVEY METHODS

Reliable, current highway condition data are vital to sound pavement management (2-7). These data are used to establish priorities for capital construction and maintenance, to decide on treatments for roads in need of attention, and to project pavement performance over time (4).

The amount and type of data collected for pavement management depend primarily on the intended uses of data in the management process. Pavement condition is often assessed by analyzing data on surface condition, structural adequacy, roughness, and skid resistance. Clearly, the collection and processing of these data for each highway link on an annual basis would be ideal. This is not possible on large (16,000 miles) highway systems like New York State's without a large expenditure. Lack of available funds and staff for full detailed surveys, along with the relatively slow rate of change for many of these items, suggests that collection of full data on all sections is neither efficient nor necessary.

Figure 1. Overview of New York State highway condition data.

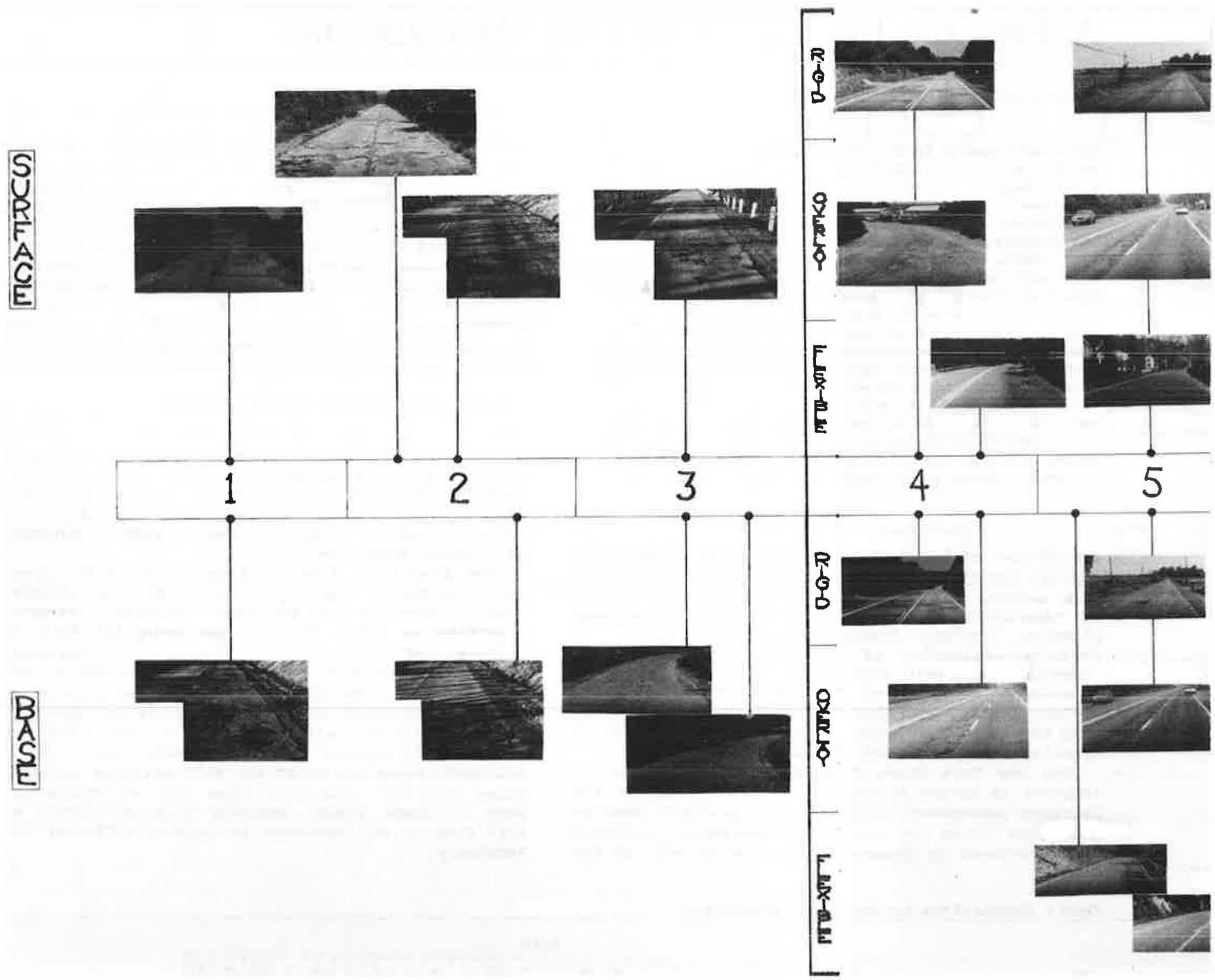
YEAR

Item	59	60	61	62	63	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80	81	82
Total System - Surface Condition	X	X	X		X		X	X	X	X		X		X		X		X		X			X	X
• Highway Condition Survey - Base Condition	X	X	X		X		X	X	X	X		X		X		X		X		X			X	X
Partial System - PRI																								
• PRI Survey																	X	X	X	X	X	X	X	
Samples																								
• HPMS (1,800 sections*) - PSR Rating																						X		X
• Continuous Counters (59 sections) - Hy Condition Scores																							X	X
• Albany Co. Deterioration Study (121 sections)																								
- PRI																						X	X	X
- BPR Roughometer																						X	X	X
- Crack/Patch indices																						X	X	X
- Highway Condition Scores																								X

* State System Only

Focus of this paper

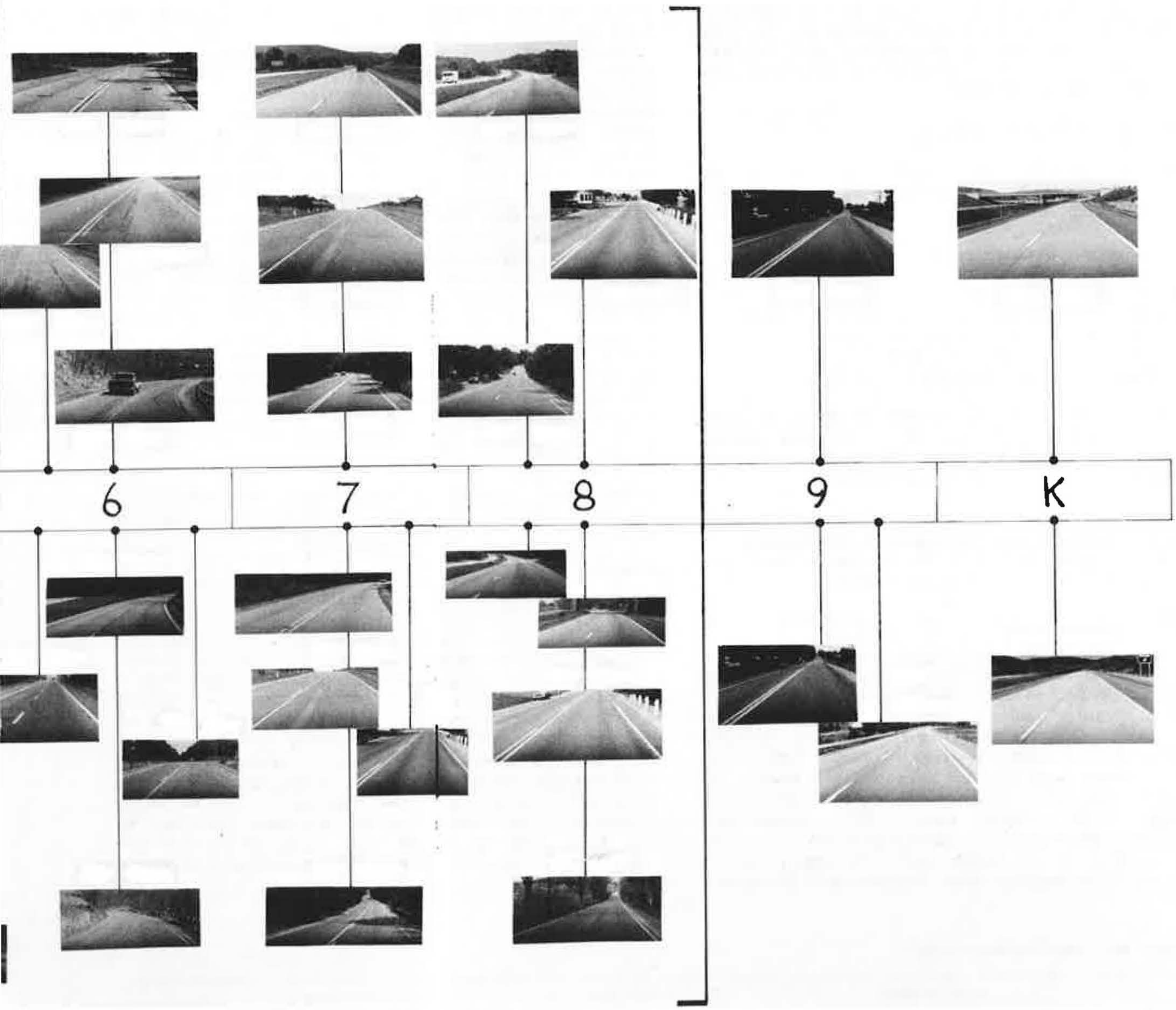
Figure 2. Photographic scales of pavement condition.



Note: K is used to represent 10.

Sampling procedures are one alternative to the more expensive mass-inventory method of data collection (4,5,8,9). Detailed data that require intricate measurement could be collected on the sample sections on a regular basis. A small number of samples carefully monitored over time could provide the analyst with all the information necessary for developing performance curves, determining service lives of various rehabilitation actions, and evaluating current construction practices. Expansion of the sample data to the entire highway system could also provide network-level estimates of condition and cost information. The federal government has recently adopted this approach in its Highway Performance Monitoring Sytem (10), which will track a sample of highway sections over time. Sampling strategies, however, are not without problems. Extreme care must be applied to the design of the sampling process to balance the cost of collecting the data and the information gained from the survey (5). Every sample survey contains sampling errors, thus results are not known with certainty. And, although sampling strategies can provide valuable input to the pavement management process, they cannot (because they are sample-based) provide the information necessary for project selection and priority ordering.

Another method used in the collection of condition data is known as a partial survey. A partial survey occurs where a preliminary visual examination of the highway system is made and is used to identify highway segments that require additional, more detailed information (8). This approach combines the best elements of the complete inventory (a census of all sections) but does not collect unnecessary detail for sections in good condition. Detailed data, comparable to those collected in sample approaches, may then be collected on selected sections to determine the exact nature of the prob-



lems. The partial survey is therefore best viewed as a filtering mechanism that provides overall monitoring capability and serves as an arrow to point to potential problems.

The methods used by NYSDOT involve both partial surveys and samples. Rideability data were collected on most of the state system (that portion that has posted speeds of 30 mph or more) annually (11) until 1981. These data were obtained by mechanical measurement of rideability at posted speed and converted to a 0 to 5 present rideability index. Pavement condition is also assessed annually on the entire state system by using a visual scoring process. These items are summarized by system (and section) and made available to the department's regional offices to assist in preparing the next year's work program. These data are being supplemented by detailed condition, characteristics, and work history data at three sample panels of highway sections. The history of these surveys is shown in Figure 1.

Highway condition surveys have been undertaken periodically in New York State for more than 20 years. In the early 1960s one team of main office engineers conducted this assessment; results were then reported in the New York State Highway Sufficiency Ratings publication, which was used extensively in highway planning and programming. In the mid-1960s, the field scoring function was decentralized to the department's regional offices. Problems with the sufficiency survey (staff turnover, inadequate training for consistency among regions, slow data processing, and lack of useful summaries) reduced the usefulness of the information. A major effort was undertaken in 1981 and 1982 to solve these problems. Procedures were implemented to enable a rapid and accurate windshield assessment of the highway network. These procedures involved the use of visual and verbal scales developed to ensure consistency among the regional field scoring teams. Teams were trained intensively in the use of these scales. Data processing procedures were also revised, and the results were furnished to the regions 2 to 3 weeks after receipt. Uses of the data include project selection, network summaries for the governor's message on transportation and capital plant renewal (12), and projection of condition under various repair strategies.

DEVELOPMENT OF CONDITION SCALES

The goal of the windshield condition survey is to provide preliminary data indicative of overall condition. The data must be

1. Consistent between regions or highway types,
2. Rapidly collectable,
3. Repeatable over time,
4. Reasonably accurate but not overly precise,
5. Easily understandable by lay persons,
6. Inexpensive to collect, and
7. Consistent with existing procedures.

The procedure developed by NYSDOT to achieve these objectives involves the use of visual and verbal scales designed to standardize the scoring process. Two separate scales have been developed:

1. Surface condition: A 1 to 10 scale that represents the condition of the pavement surface and
2. Base condition: A 1 to 10 scale that reflects problems with the underlying base.

Visual Scales

In early 1981 NYSDOT developed a set of visual scales to be used by regional staff in conducting the highway condition survey. These scales were developed through a modification of a psychological perception measurement technique known as Q-sort. Basically, this method involves a small number of experts (judges) who sort or rate a large number of photographs that show highways in various stages of condition. Eight pavement experts from various offices within the department ranked each of 50 photographs on a 1 to 10 scale once for the pavement surface and once for the base (rupture and displacement) condition. Those photographs that have the least variance in scores among the judges were selected for the photographic scales and assembled to form the actual visual scales for surface and base.

Slight refinements were made to the visual scales before the 1982 field scoring effort. Additional photographs were added to the midrange of 4 to 8, a critical area where investment decisions are often made, and the scales were stratified by pavement type. The refined scales were reproduced and included in the field scoring manual used by the regional survey teams. These scales are shown in Figure 2. Note that these photographs represent actual scale values. They are not used to portray various examples of distress signals. Details of the Q-sort procedure, including statistical validation, are described in detail in another paper (13). The use of photographs for describing the various distress signals and defining frequency and severity measures is a well-known and extensively used procedure (4,8,14); the use of photographs as scale points in evaluation of pavements has not been tried, at least to our knowledge.

Verbal Scales

Before 1981 verbal scales formed the basis of NYSDOT's highway condition surveys. These scales were also revised in 1981 and 1982 for use with the visual scales. The pavement management task force determined the types of distress most common to New York State pavements. The task force defined the distress signals for rigid (portland cement concrete), flexible (asphalt concrete), and composite (asphalt surface course overlaid on concrete slabs) pavements. Once the types of distress were determined, verbal scales were reviewed for the surface and base (rupture and displacement) for each type of pavement according to the frequency and severity of distress. Photographs of each distress signal for both surface and base (not the scale photographs determined earlier) and the frequency and severity criteria to be used when scoring were included in the field scoring manual (15) supplied to the regional survey teams. The verbal scales are given in Tables 1 and 2.

Table 1. Verbal rating scales for pavement surface.

Score	General Condition	Frequency of Distress	Rigid Pavements: Distress	Rigid Pavements: Severity	Flexible Pavements: Distress	Flexible Pavements: Severity	Overlay Pavements: Distress	Overlay Pavements: Severity
10[a]	Excellent		No distress, recently constructed or reconstructed		No distress, recently constructed or reconstructed		No distress, recently overlaid	
9	Excellent		No distress, joints functioning properly		No distress, recently resurfaced		No distress, hairline reflection cracking may exist	
8	Good	Infrequent	Joint spalling, cracking, and scaling	Very slight	Raveling, cracking, and wheel track wear	Very slight	Reflection cracking	Very slight
7	Good	Infrequent to occasional	Joint spalling, cracking, and scaling	Slight	Raveling, cracking, and wheel track wear	Slight	Reflection cracking, multiple cracking at reflection cracks	Slight
6	Fair	Infrequent to occasional	Joint spalling, cracking, scaling and patching may exist	Moderate	Raveling, cracking, rutting, and patching may exist	Moderate	Multiple cracking, raveling along cracks	Slight to moderate
5	Poor	Occasional to frequent	Joint spalling, cracking, scaling, and patching may exist	Moderate to severe	Raveling, cracking, rutting, and patching may exist	Moderate to severe	Multiple cracking, raveling along cracks	Moderate to severe
4	Poor	Occasional to frequent	Joint spalling, cracking, scaling, and patching may exist	Severe	Raveling, cracking, rutting, and patching may exist	Severe	Surface delamination	Severe
3	Poor	Frequent	Joint spalling, cracking, scaling, and patching may exist	Severe	Raveling, cracking, rutting, and patching may exist	Severe	Surface delamination	Severe
2	Poor		Extremely deteriorated, motorist discomfort, and travel difficult		Extremely deteriorated, motorist discomfort, and travel difficult		Extremely deteriorated, motorist discomfort, and travel difficult	
1	Poor		Impassable at posted speed		Impassable at posted speed		Impassable at posted speed	

[a]Coded K in Figure 2.

Table 2. Verbal rating scales for base.

Score	General Condition	Frequency of Distress	Rigid Pavements Distress	Rigid Pavements Severity	Flexible Pavements Distress	Flexible Pavements Severity	Overlay Pavements Distress	Overlay Pavements Severity
10[a]	Excellent		No distress caused by underlying roadbed movement, recently constructed or reconstructed		No distress caused by underlying roadbed movement, recently constructed or reconstructed		No distress caused by movement or deterioration of the underlying portland cement concrete slab	
9	Excellent		No distress caused by underlying roadbed movement		No distress caused by underlying roadbed movement			
8	Good	Infrequent	Slab displacement, pumping with resultant fines	Very slight	Longitudinal cracking in wheelpaths	Very slight	Non-joint-related reflection cracking	Very slight
7	Good	Infrequent to occasional	Pumping, faulting, and base-related cracking (longitudinal, diagonal, corner)	Slight	Wheelpath rutting, multiple wheelpath cracks	Slight	Non-joint-related reflection cracking, surface distortion	Slight
6	Fair	Infrequent to Occasional	Pumping, faulting, and base-related cracking	Moderate	Wheelpath rutting, alligator cracking	Moderate	Non-joint-related reflection cracking, surface distortion (faulting)	Moderate
5	Poor	Occasional to frequent	Pumping, faulting, and base-related cracking	Moderate to severe	Wheelpath rutting, alligator cracking	Moderate	Non-joint-related reflection cracking, surface distortion (faulting)	Moderate to severe
4	Poor	Occasional to frequent	Pumping, faulting, and base-related cracking	Severe	Wheelpath rutting, alligator cracking, and pieces of asphalt displaced	Severe	Non-joint-related reflection cracking, surface distortion (faulting)	Severe
3	Poor	Frequent	Faulting, cracking	Severe	Wheelpath rutting, alligator cracking, and pieces of asphalt displaced	Severe	Slab exposed and deteriorated	Severe
2	Poor		Extremely deteriorated, rupture, and displacement frequent, and motorist discomfort		Extremely deteriorated, rupture and displacement frequent, and motorist discomfort		Slab exposed and extremely deteriorated, motorist discomfort	
1	Poor		Impassable at posted speed		Impassable at posted speed		Impassable at posted speed	

[a]Coded K in Figure 2.

A note of explanation is in order concerning evaluation of the base. Obviously, one cannot see the base or any material underlying the surface when scoring a section of highway; however, certain problems manifested in the pavement surface are caused by inadequate road bed support (2,14,16,17). The base scale addresses this type of structural problem and is helpful in estimating different costs for rehabilitation (see Uses of the Survey Data).

Validation of Scales

Even though considerable effort was expended to ensure consistency in condition assessment, the question always arises as to whether the scales are being used properly in the field.

To ensure this the following procedures were followed. To ensure internal validity (replicability of the scales themselves), the scales were redeveloped by the same judges three months after initial development. Test-retest correlations showed excellent ($r > 0.9$) reproducibility (13). To ensure historical comparisons, differences among the improved scales developed in 1981 and those in use earlier were quantified and found to be negligible (13). To ensure external validity (replicability of field scores by using the scales), a small portion of the highway system (750 miles) was surveyed twice in 1982 by different raters. Table 3 gives the results: of 1,130 sections double-scored, 96 percent were scored within ±1 scale unit by both teams. The overall difference was -0.11 units (±1 percent) for surface and -0.45 (±4 percent) for base. These tolerances are satisfactory and demonstrate that visual rating systems can have high consistency if properly designed.

DATA COLLECTION AND PROCESSING

NYSDOT's highway condition survey process is shown in Figure 3. The cycle begins in early spring when refinements and modifications, if necessary, are made to procedures. A training seminar is held in the main office. Each team receives a manual of instructions for conducting the survey, complete with photographs and verbal scales (18). A series of films that simulate travel over highways in various conditions is shown at the training seminar. Each film is scored and then discussed, and the individual scorers are then instructed on how to improve their ability to make judgments by using the visual scoring materials. Additional films of highways are then shown and convergence of scoring is reasonable. In 1983 a field test over a fixed route was included. The training lasts 1.5 days and is viewed as essential to a decentralized windshield condition scoring procedure.

The survey itself is conducted by 11 two-person crews, one from each of NYSDOT's regional offices. Crew members vary in background. Some are technicians, engineers, and analysts; others are not technically trained but perform administrative or clerical functions. The quality or quantity of the work produced by these teams has been consistent if all teams receive the same intensive training.

The crew drives each highway section, usually at the posted speed, and uses the photographs for scoring. Field scoring sheets are given to each crew, arranged by route within county for easy driving sequence. The score sheets show detailed characteristics and condition data for each section; sections are short pieces of road, about 0.75 mile in length, usually with homogeneous characteristics, and built or repaired under one contract. As each section is traversed the surface and base scores are determined and placed on the score sheet and other corrections are made to field data.

Experience has shown that the team need not stop or even slow down on most sections, so the work proceeds rapidly. Scoring is done on all sections, not just those shown a given speed or outside of cities. Typically, about 125 to 200 miles a day can be surveyed, depending on weather, number of sections, and variation in condition. In 1981 and 1982 the 11 teams completed the scoring of the 16,000-mile touring route system in 10 calendar weeks, interspersed with other duties. In 1982 the scoring cost about $50,000 or $3.12/mile. Data processing and analysis were completed for another $25,000.

After each county is rated in the field, field sheets are sent to each county's resident engineer, who completes the scoring process by providing a maintenance index rating for each score section. The maintenance index is also a 1 to 10 scale, indicative of the amount of maintenance being performed on each section. Once this activity is completed, the field scoring sheets are sent back to the regional offices where they are assembled, packaged, and sent to the main office for final processing.

The main office processing procedure begins with a manual edit. Each score sheet is checked for accuracy and completeness; at the same time, current traffic volume and design hour volume are added to each record. The data are then passed through a series of programmatic contingency checks designed to find errors missed in the manual edit. The data are then summarized and transmitted to the region for review and analytic use.

Although the time alloted for the 1982 field scoring effort was two months (mid-May to mid-July) most of the regional administrators allow the scoring crews about a one month window in existing work loads to perform the survey. All data are scheduled to be returned to the regional offices by mid-August. Since main office and regional administrators are particularly interested in a timely product, this rapid turnaround is important to the success of the project.

USES OF THE DATA

The data from NYSDOT's windshield condition survey are used extensively in the pavement management process. The freshness and easy accessibility of the data appeal to both the administrator and technician. Results of the 1981 and 1982 surveys have

Table 3. Comparison of ratings for sections rated by two teams.

		Surface		Base	
Region	No. of Samples	Mean Differences in Rating	Percentage Within ±1 Unit	Mean Differences in Rating	Percentage Within ±1 Unit
1	113	0.23	95.6	-0.20	95.6
2	155	-0.08	97.4	-0.50	91.6
3	87	-0.09	100.0	-0.26	98.9
4	130	0.05	98.5	-0.06	97.7
5	60	-0.02	95.0	-0.55	90.0
6	83	0.12	95.2	-0.31	92.8
7	153	0.65	98.7	0.80	92.2
8	135	-0.21	94.1	0.22	88.9
9	81	-0.35	98.8	-1.07	67.0
10	109	-0.44	91.7	-0.51	89.0
11	24	0.21	100.0	-0.42	100.0
Total	1,130	-0.11	96.6	-0.39	91.2

Figure 3. Annual cycle of highway condition assessment.

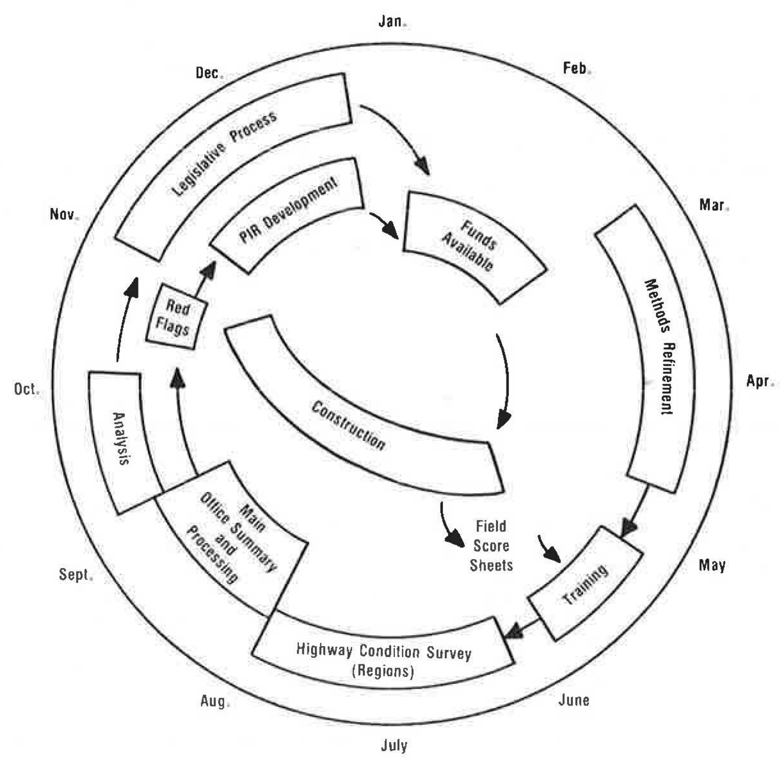


Figure 4. Sample red flag listing.

RED FLAG LISTING

INPUT FILE: 1982 SUFFICIENCY

RC ET GY	ROUTE	CO TRC YDS	BEGIN MILEPT	END MILEPT	PVMT WDTH	SHLD WDTH	1980 PRI	AADT/YR	FEDERAL AID SYSTEM	SURFACE SCORE <6	RUPT SCORE <6	MAINT INDEX <6	STRUCT SCORE <60	V/C RATIO >1.0	SUFF SCORE <60	SECTION LENGTH
18	196	011	00.45	00.87	28	00		2600 81	FAUS	5	5		56			.42
18	196	011	00.87	01.16	20	05		2600 81	FAUS	5	5		56			.29
18	196	011	01.28	02.43	20	02		2600 81	SECONDARY	5	5	5	50			1.15
18	196	011	02.43	02.50	24	00		2600 81	SECONDARY	5	5	5	50			.07
18	196	011	02.50	03.36	20	05		2600 81	SECONDARY	5	5	5	50			.86
18	196	011	03.36	05.93	20	04		2600 81	SECONDARY	5	5	5	50			2.57
18	196	011	05.93	05.97	22	10		2600 81	SECONDARY	5	5		59			.04
18	196	011	05.97	06.02	48	04		2600 81	SECONDARY	5	5		59			.05
18	197	021	00.23	00.31	24	00		5500 81	PRIMARY URB	3	3		42			.08
18	197	021	00.94	01.01	18	03		2000 81	FAUS			5	57			.07
18	197	021	06.12	06.38	20	10		3000 79	SECONDARY			5	57			.26
18	254	021	00.00	00.53	24	03		11500 79	PRIMARY URB	5	5	5	50			.53
18	254	021	00.53	00.67	24	00		11500 79	PRIMARY URB	5	5		56			.14
18	313	011	00.00	00.85	22	08		1400 77	SECONDARY	5	5		53			.85
18	313	011	00.85	01.60	22	08		1400 77	SECONDARY	5	5		53			.75
18	313	011	01.60	04.52	22	08		1400 77	SECONDARY	5	5		53			2.92
18	338	011	01.80	02.88	20	03		900 81	SECONDARY	5	5	5	50			1.08
18	338	011	02.88	03.17	20	03		600 81	SECONDARY	5	5	5	50			.29
18	338	011	03.17	06.93	20	03		600 81	SECONDARY	4	4	4	40			3.76
18	372	011	00.00	00.24	36	00		1800 81	PRIMARY RUR			5	57			.24
18	372	011	00.30	00.34	26	00		1800 81	PRIMARY RUR	4	4		49			.04
18	372	011	00.34	00.54	20	05		1800 81	PRIMARY RUR	4	4		49			.20

		SURFACE SCORE <6	RUPT SCORE <6	MAINT INDEX <6	STRUCT SCORE <60	V/C RATIO >1.0	SUFF SCORE <60
	TOTAL RED FLAG MILES IN COUNTY ARE:	60.27	62.37	83.38	95.15	4.80	.00
TOT CTY MILES: 243.11	PERCENT OF MILES IN COUNTY FLAGGED:	24.79%	25.66%	34.30%	39.14%	1.97%	.00%
	TOTAL RED FLAG MILES IN REGION ARE:	384.35	380.20	592.28	597.03	76.29	10.26
TOT REG MILES: 2035.48	PERCENT OF MILES IN REGION FLAGGED:	18.88%	18.68%	29.10%	29.33%	3.75%	.50%

THIS TOTAL EXCLUDES OVERLAP MILEAGE; OVERLAP RED FLAG MILES SHOWN ON LOWEST NUMBER ROUTE ONLY.

been used by the department in determining network-level highway needs and in preparing legislative requests.

Red Flag Analysis

The red flag computer summary is provided to the regions immediately after main office processing of the survey data is completed. The red flag programming gives the location of highway sections that fall below a selected level of condition. Thus, it serves as an early alert on problem sections that may need attention.

The select criteria are determined by the user and can be changed readily to accommodate a variety of requests. (The highway condition system is stored on-line and can be quickly assessed through remote computer terminals. Each year's file is about 20,000 records of 150 characters each.)• The red flag also computes the total mileage deficient in each county and region and the percentage of the total mileage that fails the desired criteria. Figure 4 shows typical red flag output.

Specialized Computer Summaries

Many specialized computer summaries are prepared in response to requests from numerous main office and regional administrators and technicians. The department's general purpose computer programs allow for quick on-line access and versatile output capabilities. For example, condition values can be summarized by miles of facilities in various states of condition, by pavement type within county, or by any other variable contained on the file.

Deterioration Rates

A knowledge of deterioration rates is vital in selecting optimal treatment strategies for rehabilitating pavements and for projecting network deterioration over time. NYSDOT reviewed data from the highway condition file to obtain preliminary estimates of deterioration. This analysis was conducted by arranging highways according to condition score versus the number of years that have elapsed since the last contract work was performed on the pavement (the year of last contract work is included on the condition file). The results of this analysis (19) yielded preliminary network-level estimates of deterioration by type of pavement. A research endeavor is currently being conducted (20) to provide the data necessary for deterioration analysis. This study involves long-term monitoring of pavements in the Albany area to determine the effects of various rehabilitation strategies on pavement serviceability.

Highway Condition Projection Model

The Highway Condition Projection Model (HCPM) is a long-range forecasting tool used to predict the long-term impact on highway condition of alternative general rehabilitation strategies. The model operates by projecting the condition of each highway section and the costs necessary to repair it under a given rehabilitation strategy specified by the analyst. A recent paper (1) describes the model.

Highway Sufficiency Ratings Publication

NYSDOT publishes the Highway Sufficiency Rating Book, a complete list of all sections and their condition. The publication displays comprehensive location, physical, operational, and rating measure data for each link on the state touring route system. It contains highway condition scores, including surface and base condition, maintenance index, and computed values for capacity, volume/capacity rates, and sufficiency rating. The publication

Figure 5. Sample from sufficiency rating publication.

IDENTIFICATION INFORMATION								PHYSICAL CHARACTERISTICS									OPERATIONAL CHARACTERISTICS							RATING MEASURES			
Route Number	County Name	Region & County	County Order	Control Segment	End Milepoint	Reference Marker	State Highway Number	Section Length	Lanes	Roadways	Median	Pavement Width	Surface Base Subbase	Shoulder Width	Access	Area Type	F.A. System	Funct. Class	Traffic Count Yr	AADT Volume	Design Hour Volume	Adjusted Rated Capacity	Volume/Capacity Ratio	Rt. Sr Pvmt Ride Maint	Structural Rating	PRI	Sufficiency Rating
9	ALBANY	11	08	1	00.00	ALBANY CO CITY OF ALBANY ROUTE 20 OVERLAP CONTINUES																					
9	ALBANY	11	08	1	00.04		66-1	00.04	4	2	4	56	077	00	2	5	2	B	80	23900	1480	2610	0.5	668	66		83
9	ALBANY	11	08	1	00.09		66-1	00.05	4	2	4	56	077	00	2	5	2	B	80	23900	1480	2610	0.5	668	66		83
9	ALBANY	11	08	1	00.09	END ROUTE 20 OVERLAP																					
9	ALBANY	11	08	1	00.21		66-16	00.12	2	2	4	56	075	00	2	5	2	B	78	17700	1100	930	1.1	668	66		76
9	ALBANY	11	08	1	00.38		66-16	00.17	2	2	4	48	075	00	2	5	2	B	78	17700	1100	930	1.1	778	73		79
9	ALBANY	11	08	1	00.49		66-16	00.11	4	2	4	44	075	00	2	5	2	B	78	7600	470	2530	0.1	778	73		87
9	ALBANY	11	08	1	00.63		66-16	00.14	6	2	4	80	065	00	2	5	2	B	78	7600	470	4100	0.1	888	80		90
9	ALBANY	11	08	1	00.79		66-16	00.16	UNDER CONSTRUCTION								2	B									
9	ALBANY	11	08	1	00.91	ROUTE 787I IS OVER NO CONN																					
9	ALBANY	11	08	1	01.10		68-16	00.31	UNDER CONSTRUCTION								2	B									
9	ALBANY	11	08	1	01.18		68-16	00.08	UNDER CONSTRUCTION								2	B									
9	ALBANY	11	08	1	01.26		CTYST	00.07	6	2	2	80	073	00		5	0	F	78	10600	660	4100	0.1	889	83		92
9	ALBANY	11	08	1	01.26	ROUTE 32 JUNCTION																					
9	ALBANY	11	08	1	01.31		CTYST	00.05	4	1		54	073	00		5	0	F	78	7900	490	1310	0.3	889	83		92
9	ALBANY	11	08	1	01.89		CTYST	00.58	4	1		54	482	00		5	0	F	78	7900	490	1310	0.3	559	62		81
9	ALBANY	11	08	1	01.89	ROUTE 9W JUNCTION LEFT																					
9	ALBANY	11	08	1	02.00		CTYST	00.11	4	1		54	487	00		5	0	F	78	9100	560	1380	0.4	889	83		92
9	ALBANY	11	08	1	02.24		CTYST	00.24	2	1		36	465	00		5	0	E	79	15100	930	840	1.1	889	83		87
9	ALBANY	11	08	1	02.34		67-18	00.10	3	2	2	66	454	00		5	0	E	79	15100	930	880	1.0	889	83		88
9	ALBANY	11	08	1	02.38		67-18	00.04	6	2	4	74	477	00		5	0	C	79	15100	930	4100	0.2	889	83		92
9	ALBANY	11	08	1	02.46		67-18	00.08	6	2	4	66	077	00		5	0	O	79	15100	930	3760	0.2	889	83		92
9	ALBANY	11	08	1	02.62		67-18	00.16	6	2	4	74	077	00		5	0	O	79	15100	930	3890	0.2	889	83		92
9	ALBANY	11	08	1	02.70	RR IS UNDER																					
9	ALBANY	11	08	1	02.86	ROUTE 90I IS UNDER																					
9	ALBANY	11	08	1	02.96		67-17	00.34	4	2	4	64	077	00		5	0	C	80	3900	240	2480	0.1	889	83		92
9	ALBANY	11	08	1	03.37		67-17	00.41	4	2	4	52	075	10		5	0	C	80	3900	240	2910	0.0	889	83		92
9	ALBANY	11	08	1	03.37	ROUTE 377 IS OVER WITH CONN																					
9	ALBANY	11	08	1	03.51		67-18	00.14	2	2	4	46	075	00		5	0	C	80	3900	240	930	0.2	889	83		92
9	ALBANY	11	08	1	03.72		67-18	00.21	4	2	4	60	075	00		5	0	E	79	15900	980	2480	0.4	889	83		92
9	ALBANY	11	08	1	03.82		67-18	00.10	2	2	5	30	075	08		5	0	E	79	15900	980	970	1.0	889	83		91
9	ALBANY	11	08	1	03.82	CITY OF ALBANY																					
9	ALBANY	11	08	2	00.00	TOWN OF COLONIE																					
9	ALBANY	11	08	2	00.55	9 1108 2005	22	00.55	2	1		30	462	04		4	0	E	76	14400	1050	930	1.1	988	83	3.48	86
9	ALBANY	11	08	2	01.60	9 1108 2013	22	01.05	2	1		30	462	04		4	0	E	78	15800	980	930	1.0	677	67	3.73	81
9	ALBANY	11	08	2	01.60	ROUTE 378 JUNCTION																					
9	ALBANY	11	08	2	03.04	9 1108 2031	22	01.44	2	2	5	24	472	04		4	0	E	78	21100	1310	930	1.4	998	87	4.00	79
9	ALBANY	11	08	2	03.69	9 1108 2037	119	00.65	2	2	5	24	482	04		4	0	E	80	19400	1200	770	1.5	888	80	3.29	72
9	ALBANY	11	08	2	03.91	9 1108 2040	119	00.22	4	2	5	48	475	00		4	0	E	80	19400	1200	1830	0.9	778	73	2.86	87
9	ALBANY	11	08	2	03.91	ROUTE 155 JUNCTION																					
9	ALBANY	11	08	2	04.38	9 1108 2044	119	00.47	4	2	5	48	475	00		4	0	E	78	32100	2340	1830	1.2	779	78	3.11	77
9	ALBANY	11	08	2	04.49	9 1108 2045	119	00.11	4	2	5	48	475	00		4	0	E	78	32100	2340	1830	1.2	777	70	3.38	74
9	ALBANY	11	08	2	04.70	9 1108 2048	119	00.21	4	2	4	48	075	00		4	0	E	78	32100	2340	1830	1.2	667	63		71
9	ALBANY	11	08	2	04.70	ROUTE 7 OVER																					
9	ALBANY	11	08	2	05.08	9 1108 2051	119	00.38	4	2	5	48	075	00		4	0	E	78	21700	1580	1830	0.8	667	63	3.71	82
9	ALBANY	11	08	2	05.65	9 1108 2057	119	00.57	4	2	5	48	475	00		4	0	E	78	21700	1580	1830	0.8	667	63	3.56	82
9	ALBANY	11	08	2	05.65	ROUTE 9R JUNCTION																					
9	ALBANY	11	08	2	06.06	9 1108 2061	5555	00.41	4	2	5	48	475	00		4	0	E	79	21900	1790	1830	0.9	777	70	4.27	85
9	ALBANY	11	08	2	06.59	9 1108 2065	5555	00.53	4	2	5	48	445	10		4	0	E	79	21900	1790	2030	0.8	776	67	3.85	84

presents link data contiguously by route within county and is used as a handy reference for highway information both within and outside the department. A typical page from the sufficiency ratings is shown in Figure 5.

CONCLUSIONS

A pavement management system is a valuable process used to optimize investments in the highway infrastructure. Current, reliable, and easily accessible condition data are essential to pavement management activity. Procedures being developed by NYSDOT to improve its existing windshield condition survey procedure are described in this paper. The survey, conducted annually by scoring teams from the department's 11 regional offices, involves the use of visual scales based on photographs. Intricate measurements are not required; therefore, data can be collected, processed, and summarized in a short period of time at relatively low cost to the agency. In 1982 the entire process from data collection through summarization took about 5 months and cost approximately $75,000. (This cost was not a new cost to the agency because a condition survey has historically been conducted. The effort actually cost less than in previous years because of improvements in scoring and processing procedures.)

Decentralization of data collection is the key to rapidity in data turnaround time. A strong training and validation effort ensures accuracy and consistency. The results of the survey can be used with confidence to determine network-level condition, to estimate network deterioration rates, to estimate overall costs to rehabilitate the system, and to forecast long-range implications of various rehabilitation strategies.

The most important use of the data is to red flag candidate highway sections that require more detailed engineering evaluation. The windshield survey data are used by this agency as a screening process to identify highway sections that are candidates for rehabilitation and require further analysis. Detailed engineering analysis can then be conducted in the regional offices, generally by design personnel. Decisions on the best treatment strategy for the candidate section can then be developed. Because it can serve as both a screening device for project selection and as input to network needs, the windshield survey is an important part of New York State's pavement management process.

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