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

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REFERENCES

1. A. Cheetham, M.A. Karan, R.C.G. Haas, and F. Meyer. Development and Implementation of a Pavement Management System for the Province of Alberta. Alberta Transportation, Edmonton, Jan. 1981.
2. Pavement Management Guide. Roads and Transportation Association of Canada, Ottawa, Ontario, 1977.
3. A. Cheetham and J.T. Christison. The Development of RCI Prediction Models for Primary Highways in the Province of Alberta. Alberta Transportation, Edmonton, April 1981.
4. A. Cheetham and M.A. Karan. A Pavement Quality Index (PQI) Model for Primary Highways in Alberta. Alberta Transportation, Edmonton, Jan. 1982.
5. J.B. Kerr and M.A. Karan. Pavement Management System Development and Implementation Project Interim Report for the Province of Alberta--Stage I. Alberta Transportation, Edmonton, March 1982.

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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

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

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

$$g = (P_i - P)/(P_i - P_t) \tag{2}$$

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:

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

A sample equation is given below for rutting area.

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

and

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

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

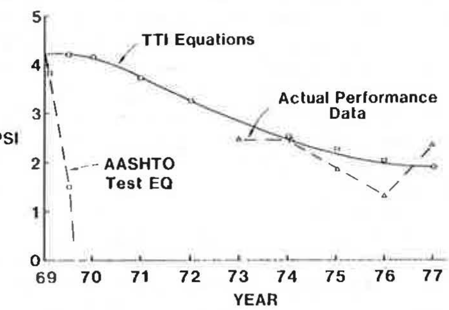


Figure 3. Plots of growth in area of various distress types of typical thin pavement section.

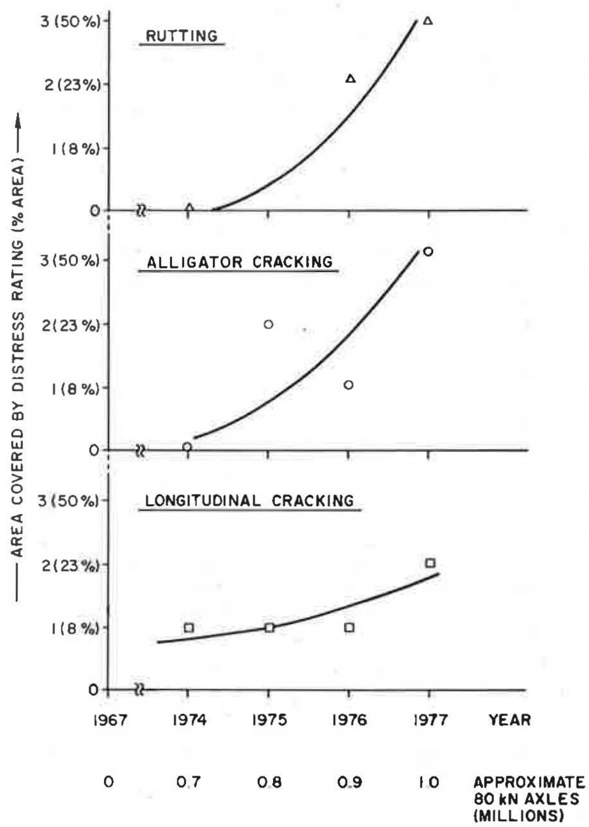


Table 2. Regression constants obtained for ρ and β equations by type of distress.

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).

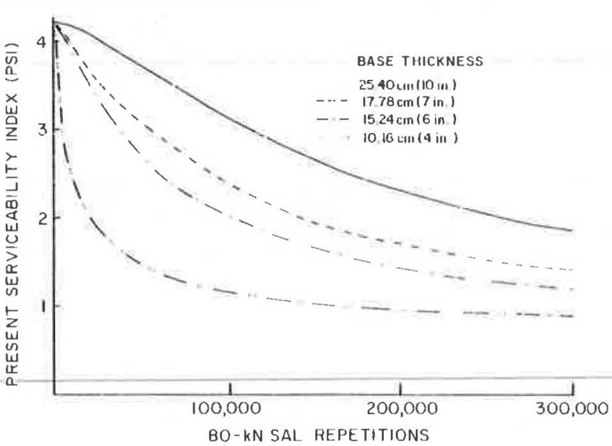
where

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 non-availability 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

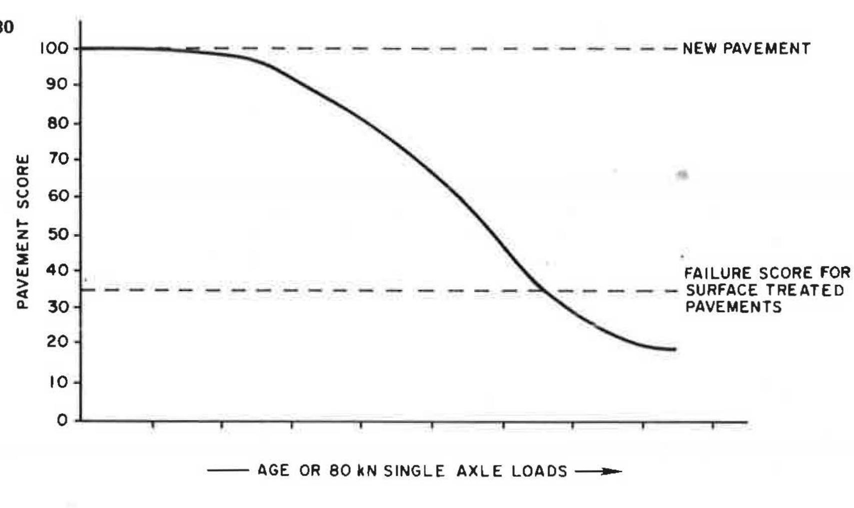
Several runs were made to test the validity of predicting pavement performance with these regression equations. Such a prediction using the PSI equation is shown in Figure 2. This is for Texas FM-556 in district 19, which is the section in the Texas Transportation Institute (TTI) flexible pavement data 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.

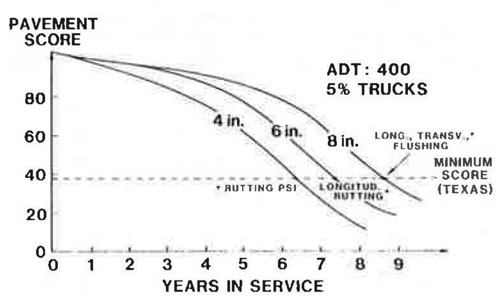
Base Thickness (in.)	Time to Failure for First Performance Period (years)	Predicted Distresses that Cause Major Reductions in Pavement Score
4	6.3	Rutting PSI
6	7.3	Longitudinal cracking Rutting
8	8.6	Longitudinal cracking Transverse cracking Flushing

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.

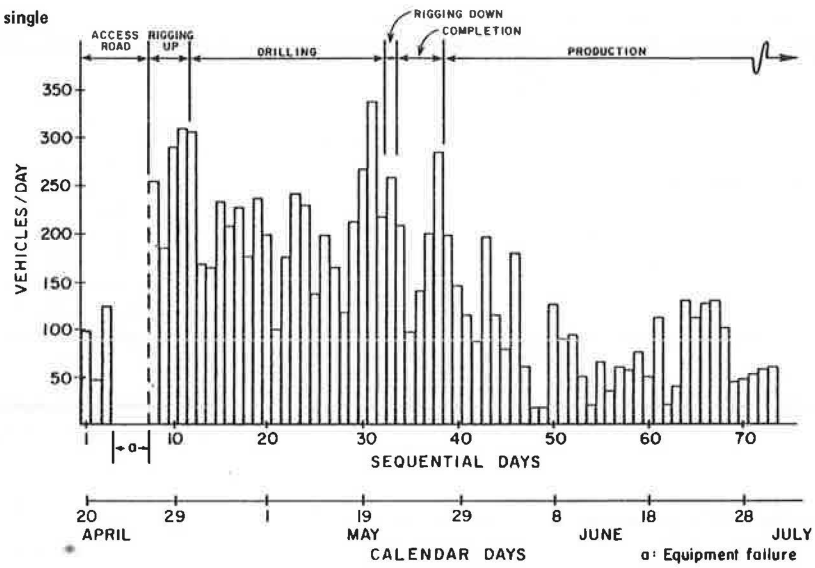


Table 3. Cumulative ADT and 80-kN ESAL repetitions for intended use and intended use plus oil wells starting in month 36.

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.

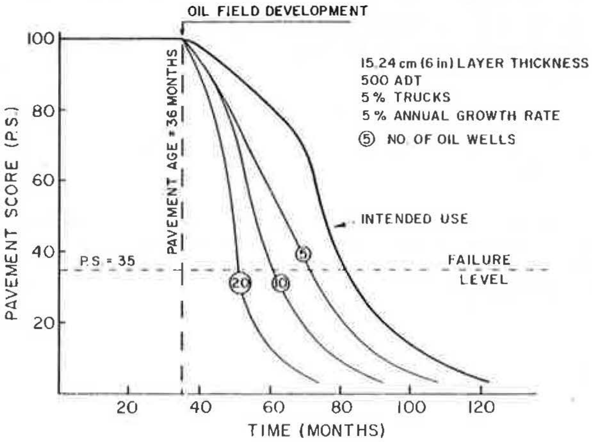


Table 4. Predicted pavement life for various levels of oil field activity.

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.

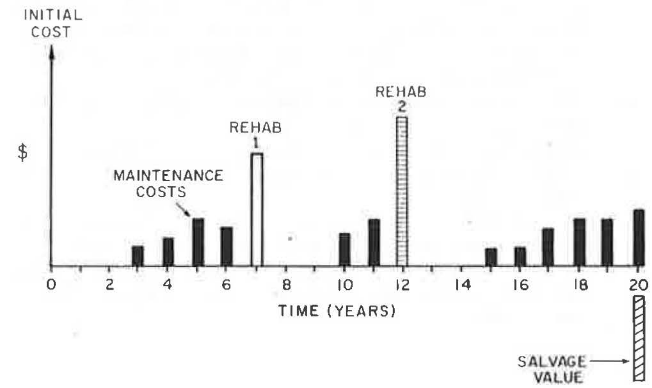
CONCLUSIONS

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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REFERENCES

1. J.M. Mason, M.J. Metyko, and N.J. Rowan. The Effects of Oil Field Development on Rural Highways. Texas Transportation Institute, Texas A&M Univ., College Station, Res. Rept. 299-1, Feb. 1982.
2. The AASHO Road Test: Report 5, Pavement Research. HRB, Special Rept. 61E, 1962.
3. AASHO Interim Guide for Design of Pavement Structures--1972. AASHO, Washington, D.C., 1972.
4. J.P. Mahoney and R.L. Lytton. Measurements of Pavement Performance Using Statistical Sampling Techniques. Texas Transportation Institute, Texas A&M Univ., College Station, Res. Rept. 207-2, March 1978.
5. J.A. Epps, A.H. Meyer, I.E. Larriemore, and H.L. Jones. Roadway Maintenance Evaluation Users Manual. Texas Transportation Institute, Texas A&M Univ., College Station, Res. Rept. 151-2, Sept. 1974.
6. M.G. Sharma, W.J. Kenis, T.D. Larson, and W.L. Gramling. Evaluation of Flexible Pavement Design Methodology Based Upon Field Observations at PSU Test Track. Proc., Fourth International Conference on Structural Design of Asphalt Pavements, Ann Arbor, Vol. 1, 1977, p. 158.
7. M.I. Darter, M.B. Snyder, and R.E. Smith. Development of a System for Nationwide Evaluation of PCC Pavement. NCHRP, Rept. 268, Dec. 1983.
8. R.L. Lytton and A. Garcia-Diaz. Evaluation of AASHO Road Test Satellite and Environmental Studies. NCHRP, Rept. 266, Dec. 1983.

9. R.L. Lytton, C.H. Michalak, and T. Scullion. The Texas Flexible Pavement Design System. Paper Presented at the Fourth International Conference on Structural Design of Asphalt Pavements, Netherlands, Aug. 1982.
10. SAS Users Guide 1979 Edition. SAS Institute Inc., Cary, N.C., 1979.
11. Collection and Use of Pavement Condition Data. NCHRP, Synthesis of Highway Practice 76, July 1981, 74 pp.
12. R.L. Lytton and others. Rehabilitation and Maintenance System: State Cost Estimating Program. Texas Transportation Institute, Texas A&M Univ., College Station, Res. Rept. 239-4, Dec. 1980.
13. Maintenance Management System: Pavement Evaluation System. Detailed Design Document, Texas State Department of Highways and Public Transportation, Austin, Aug. 1979.
14. R. Haas and W.R. Hudson. Pavement Management Systems. McGraw-Hill, New York, 1978.

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

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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