TRANSPORTATION RESEARCH RECORD 943

Pavement Maintenance Prediction and Runway Repair Materials

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Damage Functions for Rutting, Fatigue Cracking, and Loss of Serviceability in Flexible Pavements

J. BRENT RAUHUT, R.L. LYTTON, P.R. JORDAHL, AND W.J. KENIS

Damage functions are required for development of load equivalence factors to be used in allocating cost responsibilities to various vehicle classes for use of highways. They are also required by pavement management systems for prediction of pavement damage or deterioration. The only damage function and set of load equivalence factors available have been those for loss of serviceability derived from the AASHO Road Test. This work is 20 years old and resulted from accelerated testing in one environment and essentially for one subgrade, so new damage functions over the range of distresses significant to deterioration of flexible pavements were needed by FHWA to respond to a Congressional mandate for new cost-allocation recommendations. Damage functions for rutting and fatigue cracking and a new damage function for loss of serviceability are provided. These damage functions resulted from multiple regressions on 216 separate sets of predicted distresses for each of four environmental zones. The predictions were obtained with an improved version of the VESYS flexible pavement model calibrated through comparisons with measured values from 15 test sections throughout the United States. Damage predictions from the regression equations are also compared with the damage measured on 15 test sections representing a range of environmental, support, thickness design, and traffic conditions.

The Federal-Aid Highway Act in 1978 required that new cost-allocation studies be conducted to update or replace those conducted 20 years ago on the basis of results from the AASHO Road Test. Load equivalence factors to differentiate among types of damage caused by the various vehicle classes are critically important to the allocation of costs for highway construction and maintenance among these same vehicle classes. The development of load equivalence factors in turn is dependent on the availability of damage functions from which they may be derived.

It is important at this point to define both damage functions and load equivalence factors. Damage functions are mathematical equations that predict distress or reductions in performance measures [such as the present serviceability index (PSI) or the skid number] as a fraction of a referenced level of distress or reduction in performance established as a failure condition. The failure condition of interest is not necessarily structural failure but rather that level of distress or loss of performance that may be expected to generate major repair or rehabilitation. The form of the damage function or equation used in this project is similar to that used for the AASHO Road Test, where the damage function (g) is calculated as follows:

$$g = (N_{18}/\rho)^{\beta} \tag{1}$$

where

- g = damage function, which ranges from 0 to 1 with increasing damage;
- N_{18} = number of 18-kip equivalent single axle loads (ESALs) applied;
- ρ = ESALs required to produce a damage level defined as failure; and
- β = power that represents the rate of damage increase.

W represents the number of 18-kip ESALs at some time of interest, and ρ and β differ by type of distress and environmental zone and are functions of a variety of independent variables consistent with the form of distress or loss of performance under consideration.

Load equivalence factors are defined in the same way as those resulting from the AASHO Road Test, but a more specific definition will be given, because the one published after the road test was somewhat confusing.

A load equivalence factor for an arbitrary axle load is the ratio of the number of standard axle loads (18-kip single axles as in the AASHO Road Test) to produce a predefined level of distress or reduction in performance to the number of the arbitrary axle loads necessary to produce the same level of distress or reduction in performance. As for the AASHO Road Test, these ratios are calculated at the predefined failure level when damage is 1.0. It can be seen then that the ratio represents the relative number of standard versus any other axle load necessary to produce an equivalent damage. As found at the AASHO Road Test, these load equivalence factors are dependent on the definition of failure or level of damage on which they are based.

It was not feasible from either a time or a cost viewpoint to organize, perform, and analyze results from another road test (or series of road tests in different environments and for different conditions), so FHWA selected an approach that used either empirical or mechanistic models for development of the needed damage functions and consequent load equivalence factors. After review of available empirical models and mechanistic models, it was decided to use the VESYS flexible pavement model with certain improvements after its predictions for rut depths, fatigue cracking, and loss of serviceability had been calibrated against measured data from 15 test sections in four environmental zones.

Three other types of distress were considered significant in generating major repair and rehabilitation for flexible pavements but were found to be essentially independent of axle load magnitudes and are not the subject of this paper. These types of distress are low-temperature (or thermal) cracking, roughness due to differential volume change in the subgrade, and reduced skid resistance.

In this paper a description is given of how the VESYS model was used to generate a data bank and how multiple regression techniques were used to develop damage functions from this data bank; the specific damage functions developed and their applications are given. It should be noted that such damage functions not only are useful for cost-allocation studies but also are critically important to the prediction of pavement deterioration for pavement management systems.

METHODOLOGY FOR DEVELOPMENT OF DAMAGE FUNCTIONS

VESYS III-A, as received from FHWA in December 1980, lacked several capabilities that were considered necessary for the development of damage functions. Primary among these was the explicit provision for considering tandem axles. Accuracy of predictions could also be improved by introduction of capabilities to generate and modify fatigue constants by asphalt concrete modulus or temperature and to input layer moduli and permanent deformation coefficients separately for each axle load considered. These capabilities were added to produce computer program

VESYS III-B, and this mechanistic model was used in the project. The theory and capabilities of the VESYS flexible pavement model have been discussed in detail elsewhere (1-4).

Calibrating VESYS III-B

Because the simulative abilities of mechanistic models are still quite limited when compared with the staggering array of conditions offered by nature, it was decided to improve the VESYS III-B predictive capabilities by predicting distress and loss of serviceability for 15 test sections and arriving at some means of calibration after comparing the predicted and measured results. In order to include environmental effects, test sections were selected in New York, Colorado, Texas, and Florida representing wet-freeze, wet no-freeze, dry-freeze, and dry no-freeze environmental zones. Each state was visited, detailed data were collected for each test section, a condition survey was conducted, and core samples were obtained for materials characterization testing. Both resilient modulus and permanent deformation testing were conducted on asphalt concrete and subgrade samples. A detailed study was conducted to establish axle load distributions and volume as accurately as possible. All this information was then organized as input to computer program VESYS III-B and predictions were obtained for rutting, fatigue cracking, and loss of serviceability.

These comparisons of predicted and measured distresses indicated, as expected, the need for modifying the predictions from the models in some rational fashion to improve their predictive accuracy. Although it would have been ideal at this time to conduct an extensive study aimed at developing modifying functions to operate on the predicted results, time was not available because of the necessity of supporting the FHWA reponse to Congress on cost allocations, and the expedient approach of simply developing constant multipliers that would correct the predicted values to better approximate the measured values was used.

Ratios between measured and predicted values were developed for area cracked (transformed to the distress index) and rut depths. These ratios were then the discrete multipliers that would transform the predicted values into the correct measured values. Similar ratios were developed for PSI, except that the ratios represented relative reductions in PSI with traffic rather than relative values of PSI. Once a set of multipliers was available for each test section, they were compared in detail for trends with the environmental zones. As would be expected, the multipliers selected were compromises aimed at the best approximations of predicted values overall.

Development of Damage Functions

The data for the flexible pavement damage functions for rutting, cracking (distress index), and index of loss of serviceability were generated by using the computer model VESYS III-B. The input data represented a full factorial of the following number of variables:

1. Four environmental regions,
2. Three subgrade moduli,
3. Three thicknesses of surface course,
4. Three structural numbers, and
5. Eight load levels, of which four were single-axle loads and four were tandem-axle loads.

In each environmental region, 216 computer runs were made in which distress index, rutting, and loss-of-serviceability index were computed for 10 levels of load application during the life of the pavement. With this array of data it was possible to determine ρ- and β-values for each of the 27 pavement sections and for each of the eight load levels. The computer runs represented, in effect, separate miniature versions of the AASHO Road Test in each of the four climatic regions with the important distinction that three different subgrades were used instead of one as at the Road Test.

Further regression analysis was conducted to determine the manner in which the values of ρ and β depend on the variables that were used in the analysis. Many forms of equations could be assumed and tested to obtain the best relationships, but the seven shown as follows were selected as the best candidates for the detailed regression analyses and were tested and evaluated separately:

$$y = c + a(L_1 + L_2)^b (E_s)^c (SN)^d (T)^e (L_2)^f \quad (2)$$

$$y = c + a(L_1 + L_2)^{b_1+b_2T+b_3T^2} (L_2)^{c_1+c_2T+c_3T^2} (E_s)^d (SN)^e (T)^f \quad (3)$$

$$y = c + a(L_1 + L_2)^{b_1+b_2SN+b_3SN^2} (L_2)^{c_1+c_2SN+c_3SN^2} (E_s)^d (SN)^e (T)^f \quad (4)$$

$$y = c + a(L_1 + L_2)^{b_1+b_2T+b_3T^2+e_2SN+e_3SN^2} \times (L_2)^{c_1+c_2T+c_3T^2+g_2SN+g_3SN^2} (E_s)^d (SN)^e (T)^f \quad (5)$$

$$y = c + a(L_1 + L_2)^{b_1+b_2T+b_3T^2+e_2E_s+e_3E_s^2} \times (L_2)^{c_1+c_2T+c_3T^2+g_2E_s+g_3E_s^2} (E_s)^d (SN)^e (T)^f \quad (6)$$

$$y = c + a(L_1 + L_2)^{b_1+b_2T+b_3T^2+e_2E_s+e_3E_s^2+e_4(T*E_s)} \times (L_2)^{c_1+c_2T+c_3T^2+g_2E_s+g_3F_s+g_3E_s^2+g_4(T*E_s)} (E_s)^d (SN)^e (T)^f \quad (7)$$

$$y = c + a(L_1 + L_2)^{b_0+(b_1T_1+b_2T_1^2+b_3T_2)E_s} \times (L_2)^{c_0+(c_1T_1+c_2T_1^2+c_3T_2)E_s} (E_s)^d (SN)^e (T_1)^f \quad (8)$$

where

y = ρ or β;
L_1 = load on one single-axle or one tandem-axle set (kips);
L_2 = axle code (1 for single axle and 2 for tandem axle);
SN = structural number;
$T = T_1$ = thickness of asphalt concrete layer (in.);
T_2 = thickness of granular base layer (in.);
E_s = subgrade modulus of elasticity (psi) (selected for stress state around 18 to 20 in. below top of subgrade to be representative of entire subgrade); and
a, a_1, and so on = coefficients from the regressions, whose values depend on type of distress, environmental zone, and whether y is ρ or β.

It should be remembered that the load equivalence factors are determined at damage levels of 1.0 and are the ratios ρ_{18}/ρ_j; thus, only the independent variables appearing in the exponents for $(L_1 + L_2)$ or L_2 affect the load equivalence factors because all other terms cancel out in division. Equation 2, for instance, might offer reasonable predictions, but its load equivalence factors would be dependent only on axle loads.

Equation 6 was selected as offering the best fit and most useful mix of significant independent variables for distress predictions and load equivalence factors. The values of the coefficients for this equation and the coefficients of determination R^2 appear in Table 1.

Table 1. Regression coefficients for Equation 6.

y	c	a	b_1	b_2	b_3	e_2	e_3	c_1	c_2	c_3	g_2	g_3	d	e	f	R^2
Wet Freeze zone																
DI ρ	.00	1.84E+4	-0.44	-.555	.032	1.50E-5	-1.66E-10	-0.70	.686	-.038	-2.01E-5	2.24E-10	0.19	-0.05	5.59	.993
RD β	.00	1.45E-1	0.10	-.019	.001	1.10E-7	-4.24E-12	-0.08	.013	.000	-9.55E-7	1.61E-11	-0.00	0.18	-0.22	.929
RD ρ	.00	1.88E-5	-4.19	-.732	.031	6.78E-6	-4.39E-10	1.95	.577	-.037	1.18E-4	-2.15E-09	3.33	-1.76	14.87	.953
PSI β	.09	6.39E+2	-0.10	-.162	.014	6.54E-5	-1.32E-09	-1.49	.455	-.042	5.65E-6	-2.21E-10	-0.68	-2.81	0.55	.604
PSI ρ	.00	6.78E-8	-3.88	-.374	.008	-4.22E-5	4.27E-10	2.56	-.061	.023	1.47E-4	-2.64E-09	3.54	3.56	8.37	.864
Dry Freeze zone																
DI ρ	.00	1.83E+4	-0.26	-.581	.033	1.22E-5	-7.11E-11	-0.86	.697	-.038	-1.80E-5	1.41E-10	0.21	-0.00	5.49	.991
RD β	.00	1.12E-1	0.12	-.017	.001	-1.69E-6	2.76E-11	-0.08	.009	.000	-9.42E-7	3.92E-11	0.02	0.23	-0.24	.913
RD ρ	.00	8.32E-1	-4.75	-.683	.032	6.07E-6	-1.78E-10	2.29	.585	-.042	1.31E-4	-2.86E-09	2.88	-4.23	17.39	.947
PSI β	.08	1.55E+2	0.24	-.219	.018	4.61E-5	-8.51E-10	-1.50	.502	-.045	1.27E-5	-8.16E-10	-0.62	-2.07	0.61	.450
PSI ρ	.00	8.38E-9	-3.91	-.101	-.015	-9.03E-5	1.52E-09	3.99	-.673	.078	1.16E-4	-1.76E-09	3.73	3.94	8.25	.788
Wet No-Freeze zone																
DI ρ	.00	1.11E+4	-0.33	-.611	.035	1.66E-5	-1.80E-10	-0.97	.794	-.044	-1.93E-5	1.67E-10	0.17	-0.13	6.09	.994
RD β	.00	1.51E-1	-0.01	.001	-.000	2.74E-7	-2.03E-12	0.02	-.003	.000	-1.30E-7	1.58E-12	-0.02	-0.08	0.07	.964
RD ρ	.00	1.76E+1	-5.11	-.705	.041	7.08E-5	-1.31E-09	1.87	.744	-.057	9.80E-5	-2.20E-09	3.34	1.42	4.84	.989
PSI β	.07	6.57E+1	0.07	-.216	.016	5.87E-5	-1.12E-09	-1.48	.407	-.032	1.15E-5	-5.20E-10	-0.60	-2.88	1.62	.799
PSI ρ	.00	1.00E-5	-5.53	-.172	.011	-3.43E-5	5.46E-10	3.17	-.022	-.001	1.84E-4	-3.37E-09	4.18	5.04	1.18	.915
Dry No-Freeze zone																
DI ρ	.00	1.14E+4	-0.15	-.630	.036	1.48E-5	-1.19E-10	-1.09	.785	-.043	-1.98E-5	1.60E-10	0.19	-0.11	5.90	.992
RD β	.00	1.59E-1	-0.09	.018	-.001	6.78E-7	-1.32E-11	0.09	-.020	.001	-2.03E-7	6.67E-12	-0.02	-0.06	0.00	.974
RD ρ	.00	3.16E+7	-3.10	-1.069	.060	8.34E-5	-1.30E-09	-0.10	1.198	-.001	5.89E-5	-1.62E-09	2.37	1.03	4.93	.977
PSI β	.08	1.14E+3	0.00	-.194	.014	7.59E-5	-1.45E-09	-1.19	.337	-.026	-1.06E-5	-1.32E-10	-0.94	-2.92	1.66	.742
PSI ρ	.00	5.15E-6	-5.45	-.038	.004	-9.77E-5	1.95E-09	3.15	-.004	-.004	1.97E-4	-3.27E-09	4.59	5.91	-0.80	.876

It is important to note that this form of equation shows that load equivalence factors as defined previously depend on the stiffness of the subgrade. It is not surprising that pavement structure (represented by SN or asphalt thickness) is significant because it was found to be significant for the AASHO Road Test load equivalence factors. However, the dependence of load equivalence factors on the stiffness of the subgrade is new. This dependence was not found at the AASHO Road Test because the entire set of test sections was placed on essentially the same subgrade soil. Equation 2 is the form of equation that is most similar to the AASHO Road Test equation and it produces uniformly the lowest values of R^2 that were found for any of the equations tested.

EVALUATION OF DAMAGE FUNCTIONS

The most obvious approach to evaluating a damage function is to convert its calculated results to distress or loss of serviceability and to compare the calculated distress or loss of serviceability with actual measured values. In the case of PSI loss, a second comparison may also be made with predicted serviceability loss calculated by the AASHO equation. Such comparisons have been made with the measured data from the 15 test sections described earlier.

Reduction in PSI

As stated earlier, the damage equation in this project is similar to that used for the AASHO Road Test. The differences between the equations are primarily in the equations for ρ and β [AASHO equations are given elsewhere (5); see Table 1 for Brent Rauhut Engineering, Inc. (BRE) equations]. Also, the initial value of PSI assumed for the AASHO Road Test equation and for the BRE VESYS regression models was 4.2, but the terminal serviceability for the AASHO Road Test equation was 1.5, whereas 2.5 was used in the VESYS regression on the basis of observations that federal-aid highways rarely are allowed to reach a lower serviceability level.

The damage functions discussed previously may be used to convert to predicted loss of PSI by the following relationships:

AASHO:

$$\Delta PSI = 2.7 \cdot 10^{G_t} \tag{9}$$

where

G_t = $\beta(\log N_{18} - \log \rho_{18}) = \log (N_{18}/\rho_{18})^{\beta_{18}}$,
N_{18} = total 18-kip ESALs to the time at which ΔPSI is calculated, and
ρ_{18}, β_{18} = calculated ρ and β for $L_1 = 18$, $L_2 = 1$.

VESYS III-B regression:

$$\Delta PSI = 1.7g \tag{10}$$

where

$$g = (N_{18}/\rho_{18})^{\beta_{18}} \tag{11}$$

and the difference in the coefficients (1.7 versus 2.7) arises because the AASHO Road Test defined damage equal to 1 at PSI = 1.5, whereas the VESYS III-B regression results are based on a terminal PSI (damage equal to 1) of 2.5.

Measured distress and serviceability data from the 15 primary test sections used in the calibration runs for the VESYS III-B computer program were also used for comparison of these two predictive models. The soil support values for the AASHO Road Test equations were obtained by using values of California bearing ratio (CBR) or Texas triaxial class available from the data-collection effort for subgrade soils. The values of subgrade stiffness (E_s) used for the VESYS regression models were those developed for the calibration runs. Other values of the independent variables were available from the data-collection effort or calculated in the usual manner. The relationships between soil support and CBR or Texas triaxial class were those presented in Report 128 of the National Cooperative Highway Research Program (NCHRP) (6).

In both sets of calculations 18-kip ESALs were used to convert mixed traffic to standard axle loads, but those for the AASHO Road Test equation were derived from the AASHO Road Test tables (5), and those for the VESYS regression model were derived from load equivalence factors calculated as the ratio ρ_{18}/ρ.

Figures 1 through 4 show plots of measured values

Figure 1. Measured and predicted PSIs from PSI loss predictions by using AASHO and BRE equations, Texas test sections, dry no-freeze zone.

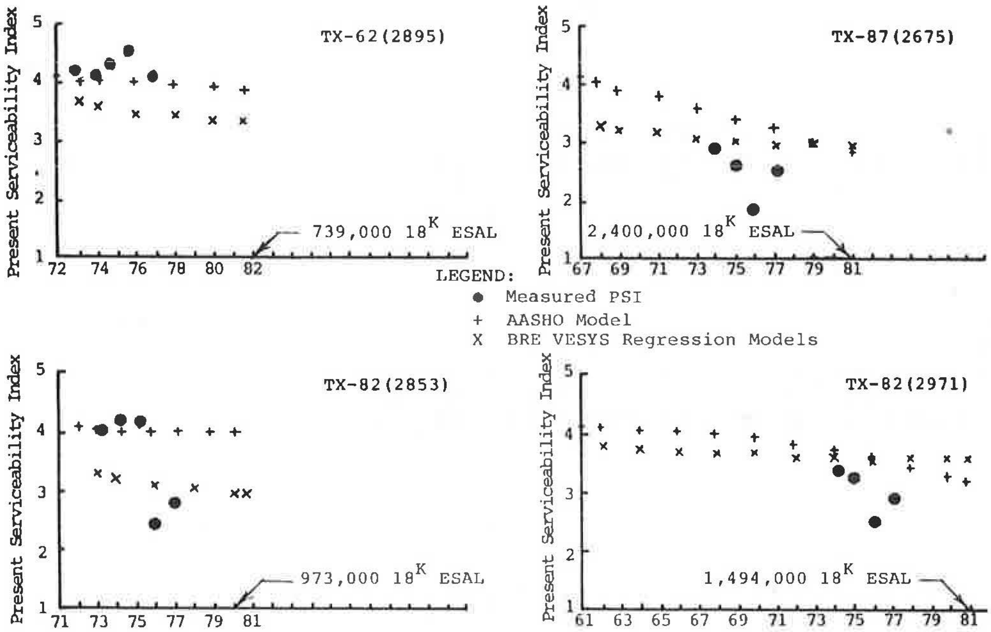


Figure 2. Measured and predicted PSIs from PSI loss predictions by using AASHO and BRE equations, Colorado test sections, dry no-freeze zone.

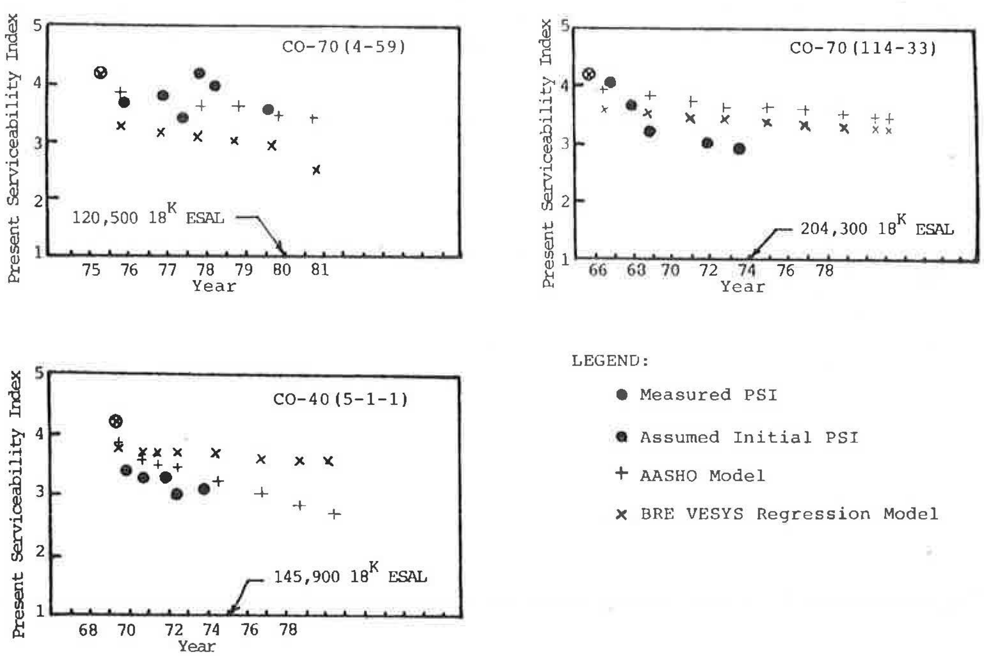


Figure 3. Measured and predicted PSIs from PSI loss predictions by using AASHO and BRE equations, Florida test sections, wet no-freeze zone.

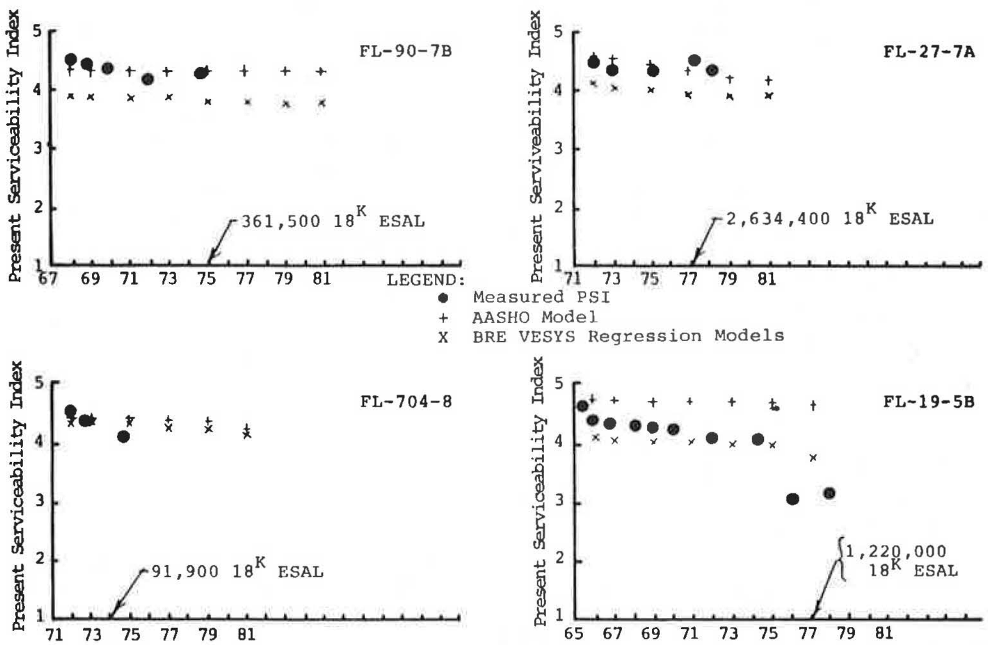
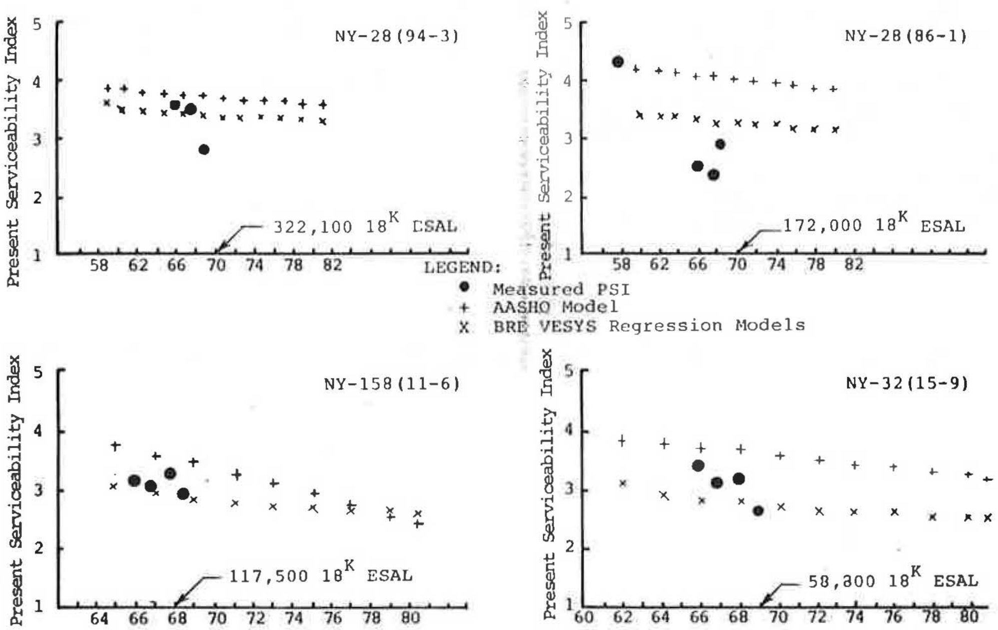


Figure 4. Measured and predicted PSIs from PSI loss predictions by using AASHO and BRE equations, New York test sections, wet-freeze zone.



of serviceability for the 15 test sections, PSIs predicted by using the AASHO Road Test equations, and PSIs predicted by the BRE regression models. For reasons discussed by Rauhut et al. (7), the regression equation (like that in VESYS III-B) predicts unrealistically rapid loss of serviceability in the first two years or so after construction or rehabilitation but continues to predict serviceability loss at a reduced rate such that the resulting PSI becomes more accurate as time (and number of cumulative axle loads) increases. Therefore, it should be remembered when these figures are studied that the unrealistically rapid loss of serviceability early in the life of the pavement is primarily the result of the choice of model and that only comparisons during, say, the last half of the analysis period are valid.

Comparisons of the predicted reductions in serviceability for the AASHO and BRE equations with the measured reductions are discussed in detail by test section by Rauhut et al. (7). As a general observation, the BRE regression equations appear to predict more serviceability loss than the AASHO equation, but not always. It appears that of the 15 comparisons for the individual test sections the AASHO equation predicted best on five, the BRE equations on seven, and three were essentially the same. The apparent conclusion to be drawn from these comparisons is that the BRE VESYS regression models (one for each of four environmental zones) predict loss of serviceability at least as well as the AASHO model with environmental zones represented by regional factors.

An attempt to further improve the BRE equations was made through applying multiplying functions to more closely predict the measured values for the test sections. Although this was successful for Texas test sections with a multiplying function developed from Texas data and Florida test sections with Florida data, the multiplying functions were not stable across a reasonable range of variables, so further improvements must await a broader data bank. The procedure used for development of multiplying functions is described in detail for rutting in the next section.

Rut Depth

The BRE damage functions for rut depth appear in Table 1 and include the four sets of regression coefficients for each of the four environmental zones. The damage function for rut depth can be converted to predict rut depth by simply multiplying the calculated damage by one-half. This conversion reflects the selection of 0.5 in. of rut depth as the failure level (damage equal to 1.0).

Figures 5 and 6 show plots of measured values of rut depth for four test sections in Texas and four in New York but also include as dashed lines the predicted rut depths obtained by using the regression equations from the VESYS factorial. As for PSI, these regression equations overpredict rut depth in the first two or three years, but the rates of rutting are reduced greatly and accuracy improves thereafter. Less rutting was generally predicted than that measured [reasons for this are discussed in detail by Rauhut et al. (7)].

Although time was not available earlier in the project to develop multiplying functions and constant multipliers were used instead, it was decided subsequently to develop and apply multiplying functions to the regression equations in order to modify their form and increase their accuracy.

The approach to developing the multiplying functions was to divide the measured rut depth by the calculated rut depth as was done for the constant multipliers previously described. However, this was done at several different times (and consequently number of axle loads) in this development. The desired multiplying function (RF) was then regressed with various forms of equations, sets of data (separate environmental zones and combinations of environmental zones), and combinations of independent variables. The final regression equations for calculating RF were as follows:

Freeze zone:

$$RF = 0.0991\ (N_{18}^{0.823}/SN^{2.70}) \qquad (12)$$

No-freeze zone:

$$RF = 0.1332\ (N_{18}^{0.2112}\ T^{0.4619}/SN^{0.7187}) \qquad (13)$$

The improved predicted equations for rut depth are then the rut-depth damage function (Table 1) multiplied by 1/2RF for each of the freeze and no-freeze zones. The resulting predictions from this improved equation also appear in Figures 5 and 6 as solid lines. As can be seen, these improved BRE equations for predicting rut depth are much more accurate and simulate the rate of increase in rut depth with time much better. The predictions for the Colorado and Florida test sections also compare quite well with the measured data.

Fatigue Cracking

Like those for rut depth, the BRE damage functions for class 2 and 3 fatigue cracking (alligator cracking) with associated regression coefficients for each environmental zone appear in Table 1. The con-

Figure 5. Measured rut depths, those calculated with regression equations, and those calculated with improved BRE equations, Texas test sections, dry no-freeze zone.

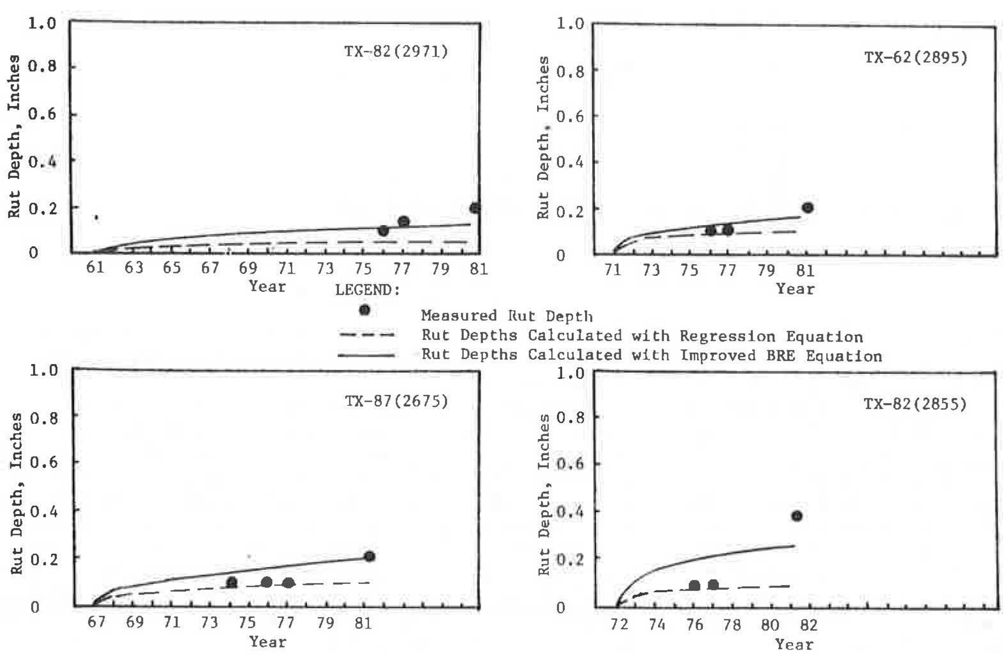
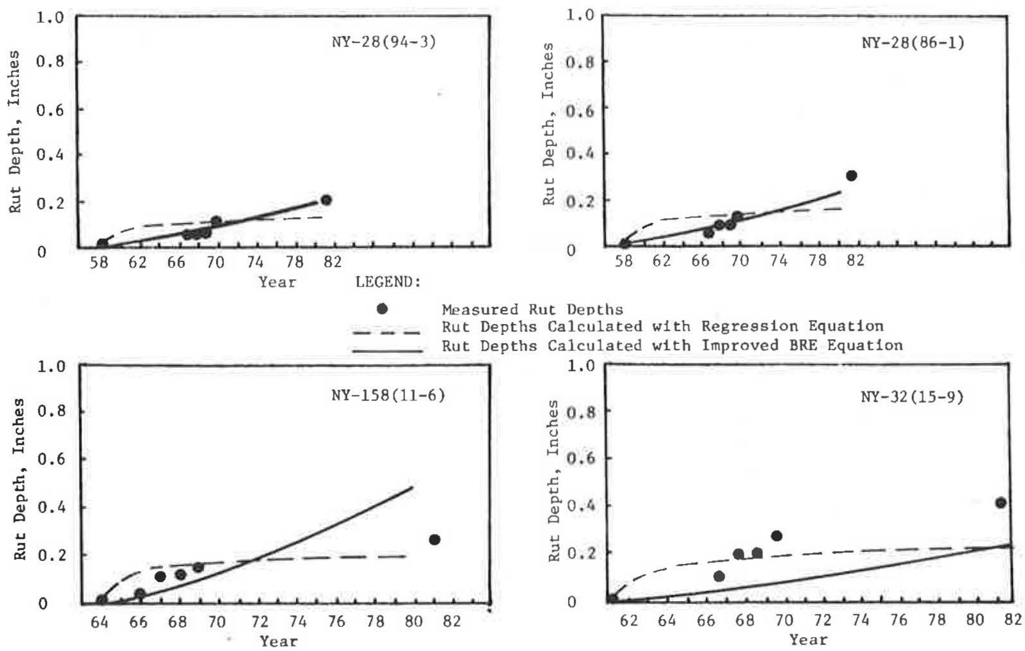


Figure 6. Measured rut depths, those calculated with regression equations, and those calculated with improved BRE equations, New York test sections, wet-freeze zone.



version to percentage of area cracked is made by using the following relationship:

$$A_c = 0.19 \exp (3.96DI) \tag{14}$$

where A_c is percentage of area cracked and DI is the damage index (or damage function) with the failure level (DI = 1) assumed as 10 percent of class 2 and 3 alligator cracking.

Equation 14 was developed during this project by using fatigue relationship data developed by Finn et al. (8). The development of this relationship is described by Rauhut et al. (9) and by Rauhut and Kennedy (10).

Figures 7 and 8 show plots of measured percentage of class 2 and 3 cracking for the four sections in Texas and the four in Florida (virtually no cracking had occurred in the test sections for Colorado and New York). Predicted fatigue cracking by using the BRE damage function equations and Equation 14 for conversion appears for comparison as solid lines in Figures 7 and 8. The predictions from the BRE regression equations reflected the characteristics of the VESYS model as modified by the constant multiplier of 0.5 on the distress index reasonably well. Similar to the VESYS predictions, minor alligator cracking occurred where minor cracking had been predicted, but some major failures had been predicted where none actually occurred.

It is extremely difficult to predict tensile cracking of any kind in asphalt concrete because of the myriad of mix and construction variables that affect the formation of cracks. In view of this and the general nature of the regression equations (they represent typical rather than specific mixes and include only the most significant independent variables), the BRE equations for fatigue cracking appear adequate for the development of load equivalence factors. They are also believed to be adequate for gross predictions of fatigue cracking damage where use of other more accurate models such as VESYS and the associated sophisticated material characterizations are not practical.

CONCLUSIONS

The damage functions developed for rutting and loss of serviceability predicted damage in a specific form rather than in the variety of forms in which this damage actually occurs in nature. A procedure was developed to produce modifying functions to transform the predicted rut depths into a form more consistent with that found from measured data for 15 test sections. It is believed that the rut-damage equation thus modified will provide reasonable predictions for typical pavement structures and pavement materials. An attempt to similarly modify the damage equation for PSI was not successful because the multiplying functions developed were unstable at extremes of structural number. Nevertheless, it is believed that this may be accomplished when the attempt is based on sufficient data.

The prediction of fatigue cracking damage (or indeed any tensile cracking damage) is problematical because of the many highly variable parameters that control crack initiation and propagation. Even in nature the variability in cracking damage is great for apparently identical pavement structures and conditions. It is believed that the damage function adopted for class 2 and 3 fatigue cracking is quite good for the typical pavement materials and conditions it represents. Nevertheless, it should be remembered that the equation for transforming distress index or damage into area cracked is applicable only to the typical fatigue relationship used in this study.

A number of problems and limitations were encountered for both the AASHO Road Test and this computer-based road test. Each had advantages and disadvantages. The one characteristic shared by both is the difficulty in transforming the data through multiple regressions into truly representative relationships. It is our belief that the results of this work, like those from the AASHO Road Test, will not prove to be final answers. However, our capabilities for predicting damage have been greatly broadened.

Figure 7. Measured and predicted fatigue cracking (classes 2 and 3) for Texas sections, dry no-freeze zone.

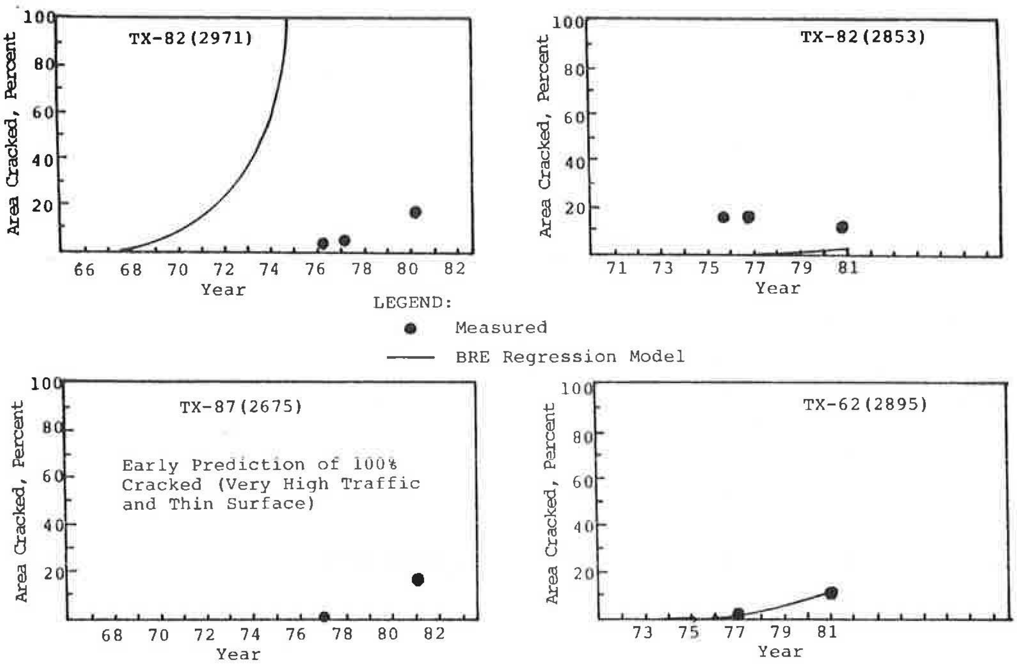
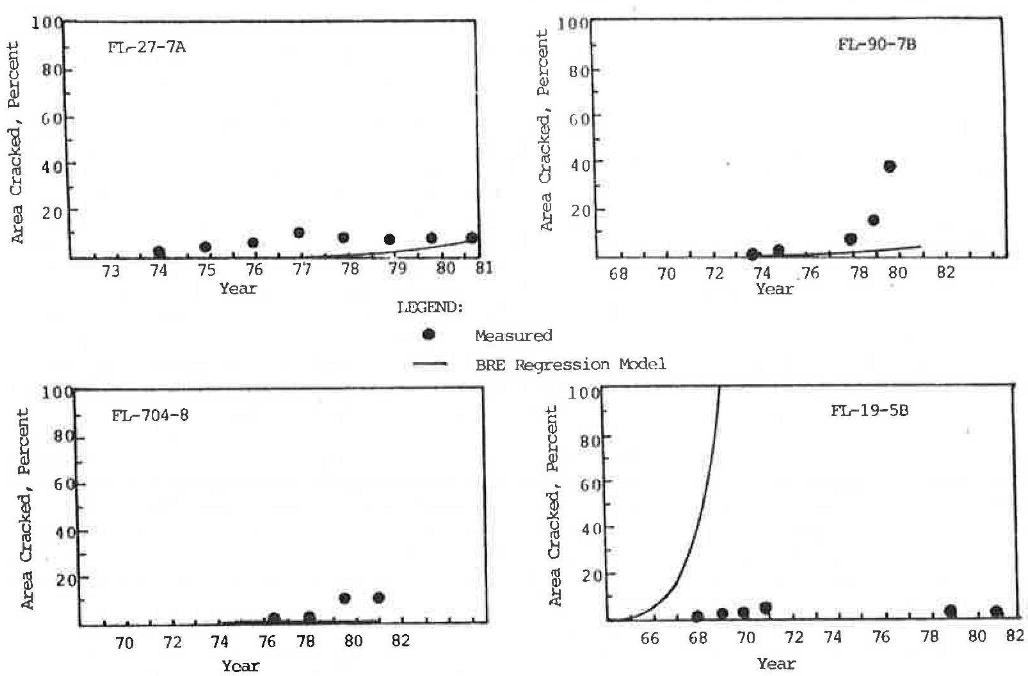


Figure 8. Measured and predicted fatigue cracking (classes 2 and 3) for Florida sections, wet no-freeze zone.



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Abridgment

Estimation of Network Rehabilitation and Maintenance Costs over an Extended Planning Horizon

ALBERTO GARCIA-DIAZ, R.L. LYTTON, AND JACK T. ALLISON

RENU is a computerized procedure to estimate funding levels required for rehabilitation, preventive maintenance, and routine maintenance of a highway network. The overall highway network consists of one or more pavement systems and each system may include several types of pavements. RENU can also be used to estimate the cost impact of changes in the legal axle load limits. The model uses a serviceability and distress approach to determine the timing for rehabilitation. Performance, distress, and survivor curves are generated based on pavement data collected from the Texas pavement system. RENU has the capability of generating survivor curves for different desired performance levels; it also contains the options of multiple overlays and the addition of new mileage during the planning horizon. A particularly interesting feature of the program is the estimation of rehabilitation costs associated with the upgrading of all pavements already in critical condition at the beginning of the analysis period. In addition to these costs, RENU also estimates the costs of keeping the network at a desired performance level during a specified planning horizon.

The basic objective of the RENU model is to estimate the cost impact of vehicle loadings on a given pavement network; this impact is measured in terms of rehabilitation and maintenance costs for all subsystems of the pavement network during a specified planning horizon. One of the most important contributions of the model is the development of a combined serviceability and distress approach to investigate the effect of a change of legal axle load limits on the life-cycle costs of highways.

Past work on procedures for estimating road rehabilitation requirements due to changes in axle load limits has resulted in the development of computerized methods such as REHAB (1,2) and NULOAD (3,4). The overall development of the RENU procedure was undertaken in three phases. The objective of the first phase of the study was to perform a comparison between REHAB and NULOAD and propose an improved methodology that would take into consideration certain requirements concerning pavement classification, data availability, and district organization of the overall highway system. The results of the first phase of the study were summarized in a series of reports (5-7).

The second phase was the development of a computerized procedure to evaluate the effects on costs and pavement condition of changing legal axle load limits that would overcome the limitations in the REHAB and NULOAD programs. The results of the second phase are summarized elsewhere (8,9).

In the third phase the scope of RENU was expanded to include a budgeting mechanism that would consider the cost of upgrading the pavement network to a specified performance level in addition to the maintenance (routine and preventive) and rehabilitation costs needed to keep the network at that performance level (10).

OVERVIEW OF RENU PROCEDURE

Briefly, the overall methodology can be summarized in four steps:

1. Incorporation of a load-distribution procedure to investigate the shift in a traffic stream toward higher loads if a new legal axle load limit is established,
2. Generation of pavement performance functions based on statistical analyses of observed data to predict riding conditions and pavement distress,
3. Generation of survivor curves to forecast the extent of pavement rehabilitation requirements in each of the periods of a planning horizon, and
4. Determination of rehabilitation costs considering the life cycles of representative sections of pavement under both the current and new legal axle load limits.

In order to decide what factor is causing the need for rehabilitation, a terminal serviceability index is compared against a specified minimum present serviceability index (PSI) value that normally is not reached before pavement distress becomes serious. If the terminal value is below the specified value, it is assumed that the worsening riding condition of the pavement is not the reason for rehabilitation. In this case each of several distress types is checked to see which one may be the cause. As a result of this analysis, pavement rehabilitation may be necessary because a critical value of either area or severity has been reached for one or more of the following types of distress: rutting, raveling, flushing, corrugations, alligator cracking, longitudinal cracking, transverse cracking, patching, and failures per mile. On the other hand, if the terminal PSI value is not below the specified value, it is assumed that rehabilitation is needed because of the deterioration of the pavement riding conditions.

PAVEMENT PERFORMANCE EQUATIONS

The input information required for the development of the flexible pavement performance equations used in the RENU model can be classified as follows:

1. Traffic factors: average daily traffic, 18-kip equivalent single axle loads (ESALs), average annual growth of traffic;
2. Climatic factors: temperature, annual average freeze-thaw cycles, annual average rainfall, wet-freeze index, Thornthwaite index;
3. Material properties: asphalt content, maximum deflection, liquid limit, plasticity index, percent passing No. 200 sieve, Texas triaxial class, volume of Dynaflect basin; and
4. Design and miscellaneous factors: initial PSI, final PSI, design period, condition surveys, structural number, layer thicknesses.

After field data concerning flexible pavement performance had been examined, the following function was postulated to represent the relative loss in serviceability index for Texas highways (9):

$$g(W) = \exp\left[-(K/W)^n\right] \tag{1}$$

where K and n are parameters and W is the traffic load in 18-kip ESALs. The damage function [g(W)] can also be expressed as the ratio of the loss in serviceability after W 18-kip ESALs to a specified maximum design loss. Let P_i be the initial PSI (at W = 0), P_t be the PSI after W_t 18-kip ESALs, and P_f be a lower bound on the PSI. Then the relative loss after W_t ESALs can be expressed as follows:

$$g_t = (P_i - P_t)/(P_i - P_f) \tag{2}$$

From Equation 2, it is possible to express P_t as a function of g_t: $P_t = P_i - (P_i - P_f)g_t$. This equation can be further rewritten after using Equation 1. The final result is as follows:

$$P_t = P_i - (P_i - P_f) \exp\left[-(K/W)^n\right] \tag{3}$$

The representation of P_t as a function of W is shown in Figure 1.

Typical equations for K, P_f, and n have been developed elsewhere (11). As an illustration, the equations for asphaltic concrete pavements with P_i = 4.70 are

$$K = 3.51 + 0.0092SN - 0.0042(TI + 50) + 0.014BASE - 0.023FTC + 0.0026PI - 0.18(TM - 50) \tag{4}$$

$$P = 2.06 \tag{5}$$

$$n = 2.06 \tag{6}$$

where

SN = structural number,
FTC = annual freeze-thaw cycles,
TI = Thornthwaite index,
TM = mean annual temperature,
PI = subgrade plasticity index, and
BASE = thickness of the flexible base.

When P_f is higher than P_t, the analysis of pavement distress can be accomplished by examining the degree to which a type of distress is extended (expressed as the percentage of the pavement surface area in need of repair) and the seriousness of the distress (expressed as crack width, crack depth, relative displacement at a joint, and so on). Usually the severity of a given type of distress can be subjectively estimated by comparing the observed distress with photographs of different levels of severity, such as slight, moderate, or severe, and then choosing numbers between zero and 1 (or 0 and 100 percent) to quantify the seriousness of surface failures.

The distress equations developed for Texas flexible pavements are of the same form as the PSI equations:

$$a = \exp\left[-(a_0/W)^n\right] \tag{7}$$

$$s = \exp\left[-(a_1/W)^n\right] \tag{8}$$

where

a = percentage of pavement surface covered by distress,
s = severity of distress expressed in numerical form,
a_0, a_1 = deterioration rates,
n = shape parameter, and
W = traffic level expressed in 18-kip ESALs.

Typical equations for a_0, a_1, and n have been developed (11). Sample equations for transverse cracking severity in asphalt-concrete pavements are

$$a = 1.40 - 0.094(TM - 50) - 0.0088FTC + 0.17H + 0.010PI \tag{9}$$

$$n = 3.28 \tag{10}$$

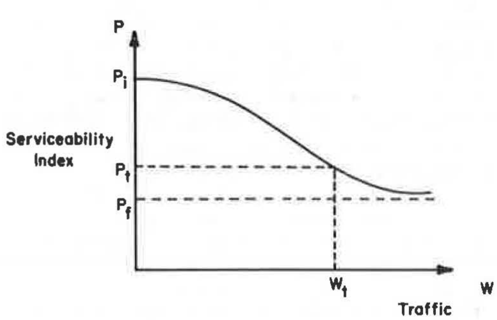
Similar equations for alligator cracking are

$$a = -0.87 + 0.88SN + 0.011(TM - 50) - 0.376H \tag{11}$$

$$n = 2.27 - 0.072PI - 0.015(TI + 50) + 0.92H \tag{12}$$

where H is the thickness of the surface.

Figure 1. Pavement performance relationship.



SURVIVOR CURVES FOR FLEXIBLE PAVEMENTS

Survivor curves are empirical probability functions used to predict the percentage of pavement mileage of a specific age that will not need rehabilitation in the near future. This in turn can be used to estimate the percentage of mileage that will need rehabilitation in the near future. This information, complemented with data on existing mileage and rehabilitation cost, can be used to estimate the funds needed in each period of a specified planning horizon.

The survivor functions developed for RENU can generally be written as follows:

$$V = 1 - \exp(-q/W^r) \quad (13)$$

where

V = percentage of surviving mileage,
q = parameter affecting the location of the survivor curve,
r = parameter affecting the shape of the curve, and
W = traffic level since construction or last rehabilitation.

For each pavement system of a large-scale network, survivor curves have been developed for typical pavements that are rehabilitated or reconstructed at several different performance levels; these performance levels are defined as follows:

Highway System	PSI for Performance Level: High	Medium	Low
Interstate	3.2	3.2-2.3	2.3
U.S. and state	3.0	3.0-2.1	2.1
Farm to market	2.9	2.9-2.0	2.0

Table 1 gives typical values of the parameters q and r for 10 types of flexible pavement. A graphical representation of the three survivor curves used by RENU in the distress option is shown in Figure 2. As can be seen, the percentage of pavement surviving after a given traffic volume is less when the performance standard is higher.

SPECIAL FEATURES

The RENU program computes the initial cost required to upgrade the system to one of the three levels of performance and the cost of keeping the pavement at the specified level with the corresponding survivor curves. The following particularly interesting features can be used for identifying meaningful scenarios for the RENU procedure.

New Options for Pavement Below Critical Performance Levels

According to this feature, the user of RENU can input a strategy for upgrading in a prescribed number of years all pavements in critical condition at the beginning of the analysis period. The corresponding mileage is reduced uniformly over the specified period by placing overlays of thicknesses that may be different from those used in ordinary cases. Critical pavements are referred to as pavements older than terminal serviceability (POTTS) in the RENU program.

Multiple Overlays

The first overlay of a pavement is placed at the time specified by the survivor curve. Additional overlays are placed at time intervals prescribed by the user.

Routine Maintenance

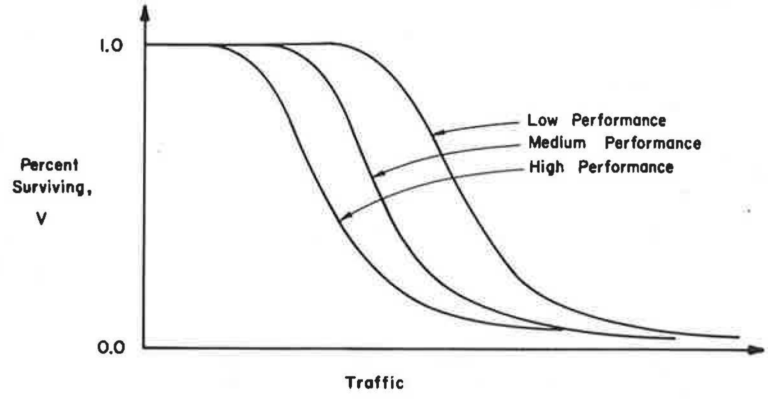
The POTTS mileage is considered part of the mileage that should receive routine maintenance. The corresponding rehabilitation costs are estimated by using the EAROMAR procedure (12) and cost information provided by the user. Three maintenance activities

Table 1. Constants for survivor curves for flexible pavements (distress option).

Type of Pavement	Constant by Performance Level: High r	High q	Medium r	Medium q	Low r	Low q
Rural, high-traffic HMAC	2.5	0.44×10^{16}	2.5	0.69×10^{16}	2.5	0.10×10^{17}
Rural, low-traffic HMAC	2.5	0.10×10^{14}	2.5	0.16×10^{14}	2.5	0.23×10^{14}
Urban, high-traffic HMAC	3.2	0.24×10^{22}	3.2	0.43×10^{22}	3.2	0.69×10^{22}
Urban, low-traffic HMAC	3.0	0.11×10^{18}	3.0	0.19×10^{18}	3.0	0.29×10^{18}
Rural, high-traffic overlay	2.5	0.21×10^{16}	2.5	0.43×10^{16}	2.5	0.76×10^{16}
Rural, low-traffic overlay	3.0	0.27×10^{16}	3.0	0.62×10^{16}	3.0	0.12×10^{17}
Urban, high-traffic overlay	3.1	0.19×10^{21}	3.1	0.46×10^{21}	3.1	0.91×10^{21}
Urban, low-traffic overlay	2.9	0.12×10^{17}	2.9	0.26×10^{17}	2.9	0.50×10^{17}
Urban, surface treated	2.3	0.12×10^{11}	2.3	0.25×10^{11}	2.3	0.45×10^{11}
Rural, surface treated	2.3	0.92×10^{9}	2.3	0.19×10^{10}	2.3	0.33×10^{10}

Note: HMAC = hot-mix asphalt concrete.

Figure 2. Survivor curves for three different performance levels.



are included in the analysis: patching, crack sealing, and base and surface repairs.

Preventive Maintenance

For each of the three levels of performance for which survivor curves were developed, preventive maintenance is provided in terms of seal coats. The user of the program specifies the time between seal coats and the cost per lane mile. This option is also available for pavements in POTTS.

New Mileage

New mileage can be added to the highway network as a result of reconstruction or new construction. This new mileage is considered by maintenance and rehabilitation in the same manner as existing pavements.

USE OF THE MODEL

The application of RENU reported in this paper can be summarized as follows. The Texas highway network was classified by system (Interstate, U.S., state, and farm to market), by pavement type (surface treated, asphaltic concrete, overlaid asphaltic concrete, and concrete), and by traffic level (low and high) for the five regional areas of the state. Age versus lane-mile distributions were identified for each of the previous classifications by using the state's road inventory file.

Overlay thicknesses were calculated for three possible desired standards of performance. These were based on typical deflection data and traffic levels. A sample of the values used in the procedure is given below:

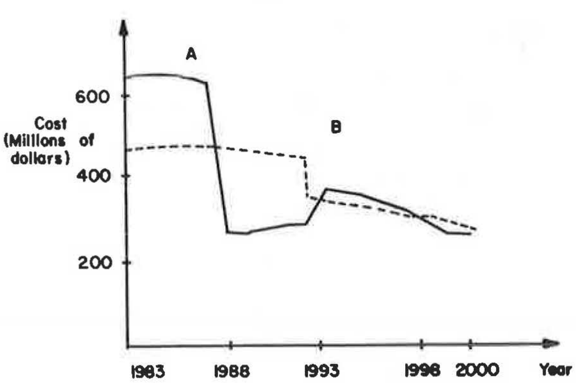
Highway Type	Overlay Thickness (in.) by Performance Level		
	High	Medium	Low
U.S. Interstate	1.75	3.50	4.50
U.S. or state	1.25	2.40	3.30
State or farm to market	0.40	1.40	1.90

Cost information on pavement rehabilitation and maintenance was obtained from the districts throughout the state. Average costs used in the application being discussed are summarized as follows: pothole repair, $125/yd^3$; crack sealing, $0.25/linear ft; base and surface repair, $12.50/yd^2$; seal coating, $3,200.00/lane-mile; and asphalt-concrete overlay, $69.00/yd^3$.

By using the RENU model, the Texas State Department of Highways and Public Transportation was able to compare the monetary requirements for different desired levels of performance and different strategies for upgrading pavements for which rehabilitation was overdue. Figure 3 shows a comparison of some of the possible strategies. Curve A shows the total annual rehabilitation, preventive maintenance, and routine maintenance costs, assuming that all critical pavements will be upgraded in 5 years. Curve B, the dashed line, shows the same total costs if the same pavements are upgraded in 10 years.

It may be noted in Figure 3 that there exists a considerable backlog of pavements due for rehabilitation and that once they are brought up to a specified performance standard, the total costs of maintenance and rehabilitation level off. From a budgetary point of view, funds need to be increased during the first portion of the analysis period to provide a desired pavement system quality; from then on, a reasonably constant budget will be required to maintain that level.

Figure 3. Total annual maintenance and rehabilitation costs.



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Prediction of Pavement Performance by Using Nondestructive Test Results

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The possibility of using nondestructive test results to predict pavement performance is examined. Preliminary analysis found that nondestructive test results and pavement age parameters correlate well with the pavement condition. Also, depending on the pavement type, other factors such as weighted traffic counts correlated with the pavement condition. The data used in the analysis were collected at a military installation located in Virginia. Pavement condition was rated by using the pavement condition index developed by the U.S. Army Corps of Engineers. Nondestructive testing was performed by using a falling-weight deflectometer. The preliminary results indicate that pavement performance can be predicted by using nondestructive testing data.

U.S. pavements are deteriorating rapidly and have an unoptimistic future. Maintenance and incidental user costs are increasing as maintenance and rehabilitation funds fail to keep pavements at an acceptable level of serviceability. Because a dramatic increase in funding is an unrealistic prospect, using available funds to the best advantage is imperative. A pavement management system (PMS) is precisely the tool needed to aid in performing such a task. Optimal use, priority ranking of projects, and pavement system inventory are all immediate benefits of any well-organized PMS.

There are many PMSs available and the key element to all workable systems is a consistent method for rating the condition of the pavement. These pavement condition ratings provide the necessary criteria to establish an effective maintenance policy by relating condition to maintenance needs. However, these rating systems provide only a measure of the current condition of the pavement and not the future condition. To make management benefits maximal, it is necessary to have a reliable method for predicting the future condition of the pavement. Developing a pavement performance model requires that a number of variables be considered, including traffic and structural capacity. The impetus of this paper is to establish whether nondestructive test (NDT) results (as a measure of pavement strength), in combination with other variables such as age and traffic, can be used as predictors of pavement performance.

Nondestructive testing is now being used for structural evaluation of pavements and is a common component in overlay design. Nondestructive testing can be performed quite rapidly and the test results can be applied in a PMS to select the optimum repair alternative for a given project. To have the additional ability to use nondestructive testing results in pavement performance prediction models is a distinct advantage to any PMS.

TEST PROGRAM AND DATA COLLECTION

In a continuing effort to improve the PMS called PAVER, the U.S. Army Corps of Engineers Construction Engineering Research Laboratory has collected NDT results for the pavements at a military installation in Virginia. The installation is currently using the PAVER system as its PMS. The U.S. Army Corps of Engineers Waterways Experiment Station performed the nondestructive testing with a Dynatest 8000 falling-weight deflectometer (FWD). All pavement sections included in the PAVER system were tested (191 sections).

The FWD was selected because of its modeling of moving loads. This has been documented by Hoffman and Thompson (1,2). A future phase of the project will be to compare the test results of the FWD and the model 2008 road rater to ensure that prediction models are not device dependent (road rater testing was performed concurrently with FWD tests).

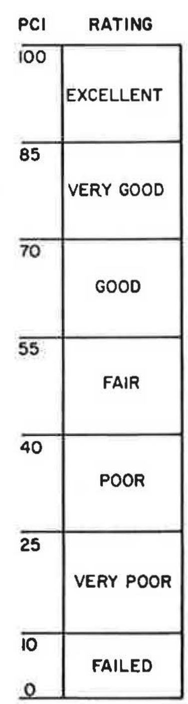
The actual FWD test scheme consisted of the following:

1. Three test locations in a section;
2. Three impulse-load levels of approximately 5, 9, and 15 kips per test; and
3. Three deflection measurements per load obtained with geophones located at 0, 12, and 36 in. from the center of the load plate per test.

For a given pavement section and load level, the average deflection was calculated for each of the geophone locations. Corrections were made for the temperature at the time of testing. Load versus deflection and deflection-basin characteristics for each section were calculated based on the average responses.

In addition to the NDT data, other information on the pavement sections was obtained from the existing PAVER data base. This included information on pavement structure, pavement layer ages, traffic counts, and pavement condition. The condition rating being used at the military installation in Virginia is the pavement condition index (PCI) developed by the U.S. Army Corps of Engineers. It is an objective rating method based on measuring the quantity and severity of each distress type present in the pavement. The PCI is a numerical indicator that uses a scale of 0 to 100; the scale and associated ratings are shown in Figure 1. The PCI has been proven reproducible

Figure 1. PCI rating scale.



by field testing and correlates well wih the collective judgment of experienced pavement engineers. Additional information and documentation concerning the PCI are provided elsewhere (3).

All data were computer encoded into a data file for use with the Statistical Package for the Social Sciences (SPSS) computer software. The next section summarizes the analysis of data and describes the developed pavement performance prediction model.

MODEL DEVELOPMENT

Variables used for prediction model development include the PCI; pavement inspection date; pavement layer material types; pavement layer thicknesses; pavement layer construction dates; current traffic volume and classification; pavement distress types, quantities, and severities; and FWD test data, temperature, time, load, and deflections for each pavement section. These variables are grouped into six general categories representing specific variable classes:

1. Pavement type,
2. PCI and pavement distress data,
3. NDT information,
4. Pavement construction or inspection dates,
5. Traffic information, and
6. Pavement layer thicknesses.

Table 1 presents a typical variable from each category, the range of that variable, and a typical value.

Table 1. Typical variables for general data categories.

Typical Variable	Typical Value	Range High	Range Low
Pavement type	Asphalt concrete, no overlay	NA	NA
PCI	85	100	42
Maximum FWD deflection (mils), high load level	36.6	97.4	12.7
Surface construction date	June 1952	July 1974	February 1935
Current traffic volume (vehicles/day), type A	1,925	17,616	50
First overlay thickness (in.)	1.5	4.3	0.5

Note: NA = not applicable.

Table 2. Variables for prediction model.

Variable	Mean	Range High	Range Low	Standard Deviation
PCI	84.9	100	42	10.6
AGE	7.0	29	0	5.0
AGESOL	22.4	40	4	8.9
AGETOT	29.4	44	17	7.7
AGECOL	16.4	40	1.0	8.9
PMTOT	6,891	47,642	7.5	11,354
LPMTOT	3.16	4.68	0.875	0.928
DIFF	0.344	1.0	0.151	0.106
AREA	60.2	164.6	23.0	22.4
LOLTHICK	1.26	2.0	0.70	0.29
TOLTHICK	1.96	5.3	0.70	1.09
SURTHICK	2.37	5.0	1.00	1.06
TOTHICK	4.35	8.5	2.30	1.62

Note: AGE = age of pavement since last overlay (yr), AGESOL = age of pavement to last overlay (yr), AGETOT = total age of pavement (yr), AGECOL = age of previous construction to last overlay (yr), PMTOT = weighted traffic total (vehicles/day), LPMTOT = log of weighted traffic total (vehicles/day), DIFF = normalized deflection basin slope, AREA = area of FWD deflection basin at the high load level (in.^{2} x 10^{-3}), LOLTHICK = last overlay thickness (in.), TOLTHICK = total overlay thickness (in.), SURTHICK = surface thickness (in.), TOTHICK = total pavement thickness (in.).

In developing the performance prediction model, linear stepwise regression methods were used to analyze the data. The first step in this analysis was to divide the data by pavement type and run a general correlation matrix with the collected variables and pavement condition. The division of the data by pavement type indicated that there are sufficient cases (population) only to have a statistically significant prediction model for asphalt-concrete pavements with asphalt-concrete overlays. This lack of data does not signify that pavement performance cannot be predicted for other pavement types. In fact, preliminary analysis by using the limited data on other pavement types suggests that pavement performance can be predicted.

The preliminary correlation matrix for the asphalt-concrete overlay sections was used to select variables for further analysis. Variable selection was accomplished by minimizing the linear dependence or correlation between the independent variables (predictor variables). Once this selection process was complete, initial model development was possible. The specific variables considered during the model development are summarized in Table 2, in which the mean, range of values, and standard deviation for each of the variables are also given. Not all of the variables were found to correlate with PCI and these are not included in the prediction model. The model presented in Figure 2 includes pavement layer ages, a weighted traffic variable, and NDT parameters. The specific variables included in the model are described in subsequent paragraphs.

Statistics for the developed performance prediction model give a correlation coefficient equal to 0.765 and a standard deviation of 6.9. The relative significance of each variable group was approximately 60 percent for the age variables, 30 percent for the NDT variables, and 10 percent for the traffic variables. A plot showing the actual PCI versus the predicted PCI is given in Figure 3. These statistics show a significant correlation between predicted PCI and actual PCI, which indicates that NDT results used in conjunction with traffic and pavement age variables can be used to predict pavement performance.

Actual PCI values ranged from 42 to 100. However, the mean PCI value is 85 and as can be seen from the plot in Figure 3, most of the actual PCI values are above 60. Aside from indicating that a good functional pavement network exists at the installation, this data range does not allow low PCIs to be accurately predicted; therefore, a limitation of the current prediction model is that PCI values

Figure 2. Performance prediction model.

PERFORMANCE PREDICTION MODEL FOR AC PAVEMENTS WITH AC OVERLAYS

$$
\begin{aligned}
\mathrm{PCIP} = 96.6 - [&(.00156 * \mathrm{AGE}^{1/2} * \mathrm{AGETOT} * \mathrm{LPMTOT} * \mathrm{DIFF} * \mathrm{AREA}) \\
&+ (.03062 * \mathrm{AGE}^{1/4} * \mathrm{AGESOL}^2 * \mathrm{DIFF}^2) \\
&+ (.0005728 * \mathrm{AGE}^2 * \mathrm{LPMTOT} * \mathrm{DIFF} * \mathrm{AREA})]
\end{aligned}
$$

WHERE

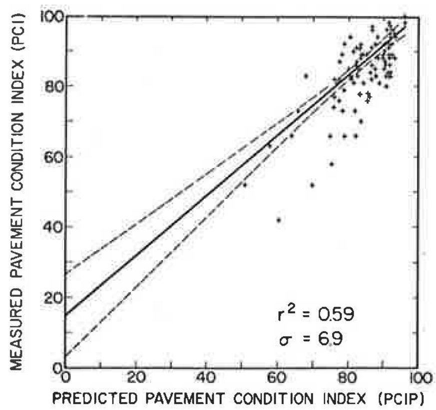
PCIP = PREDICTED PAVEMENT CONDITION INDEX

STATISTICS

CORRELATION COEFFICIENT (r) 0.765

STANDARD ERROR OF ESTIMATE (σ) 6.88

Figure 3. Measured versus predicted PCI.



of less than 60 cannot be predicted with a reasonable degree of confidence.

Two parameters calculated from the NDT deflection measurements are used in the performance prediction model. These parameters are determined by using the deflection measurements taken at 0 and 12 in. from the point of loading. Statistical analysis indicates that using only the deflection measurements taken at these two locations provides the best correlations. The parameters are a normalized deflection factor that gives a measure of the slope of the deflection basin (DIFF) and a measure of the deflection-basin area (AREA) at a given load level. Figure 4 presents the DIFF and AREA variables. Although not exactly the same variables are found in other literature on nondestructive testing, both DIFF and AREA correlate strongly with the variables defined by Hoffman and Thompson (1,2). Theoretically DIFF can vary from 0 to 1.0; stiffer pavements have lower values. For AREA the larger the value the less stiff the pavement.

These two variables, AREA and DIFF, were used in the performance model because they were found to have the best correlation with PCI. During the initial development of the performance prediction model, it was considered critical that NDT parameters having engineering significance and best correlation with PCI be used.

Thickness was not found to have a high correlation in the regression analysis. A number of thickness variables were examined, such as total pavement thickness, asphalt-concrete thickness, and a weighted total pavement thickness, yet none of these variables was found significant. However, both thickness and NDT results are used as measures of pavement strength, so it is not surprising that thickness was not found to be a significant variable. In fact, if the thickness were included in the model, it would weaken the effect of the NDT variables in predicting pavement performance.

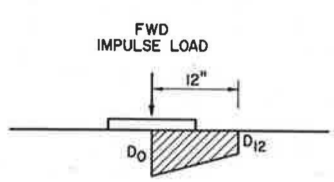
As is expected, pavement age is an important variable in the prediction of pavement performance. Figure 5 gives the age variables with PCI. Three pavement-layer age variables are included in the prediction model--AGE, AGESOL, and AGETOT. AGE is the time since the last overlay, AGESOL is the time from construction to first overlay, and AGETOT is the total pavement age. Total pavement ages ranged from 17 to 44 yr; the time since the last overlay ranged from 0 to 29 yr. This represents a good distribution of ages for the given pavement type; therefore, pavement age is not expected to present itself as a limiting factor in the performance prediction model.

Finally, a weighted traffic variable (LPMTOT) is included in the prediction model. LPMTOT is the natural logarithm of a weighted current traffic count. The current traffic counts are divided into different categories based on the vehicle size and load. Because heavier vehicles (trucks) cause more damage, the traffic counts must be weighted to account for this differential. Coefficients used in weighting the different traffic categories were obtained based on information from Yoder (4) and other researchers (5). The traffic types and weighting are presented in Figure 6.

The prediction model was developed from data obtained at one location and is applicable to that location only. Climatic effects limiting the use of the model to one geographical area have not been considered. A general model will require that data be obtained from different locations, so the number of freeze-thaw cycles and mean annual temperature can be considered.

A major assumption in using NDT data to predict pavement performance is that the NDT response does

Figure 4. NDT variables used in regression analysis.



$$AREA = 12 \times (D_0 + D_{12})$$

$$DIFF = \left(\frac{D_0 - D_{12}}{D_0}\right)$$

Figure 5. Pavement age variables.

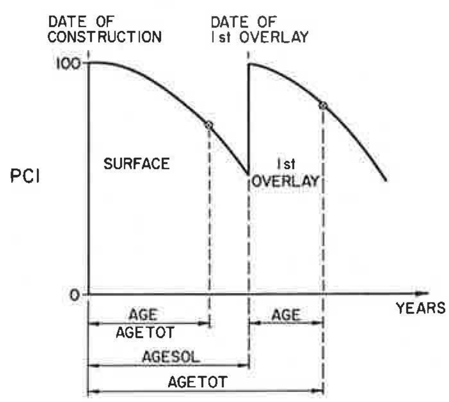


Figure 6. Traffic types and weighting.

TRAFFIC TYPE	DESCRIPTION
TYPE A	PASSENGER, PANEL AND PICKUPS
TYPE B	TWO AXLE TRUCKS AND BUSES
TYPE C	TRUCKS WITH THREE OR MORE AXLES
TYPE D	60^K TRACK VEHICLES AND 15^K FORKLIFTS
TYPE E	90^K TRACK VEHICLES AND 20^K FORKLIFTS

TRAFFIC WEIGHTING

PMTOT = .15 * TYPE A + 80 * TYPE B + 500 * (TYPE C + TYPE D + TYPE E) not change with time if other variables such as time of testing and temperature remain constant. Based on evidence from several researchers (6), this appears to be a reasonable assumption if the tests are performed on sound sections of the pavement. The response is expected to remain constant until the pavement starts to fail structurally. This concept is shown in Figure 7. Because the purpose of the predictions is to outline maintenance and rehabilitation requirements, pavement sections should not reach the level where NDT response would change drastically.

The performance prediction model as currently developed has some limitations. However, the statistics from the model indicate that NDT results can be used in combination with other variables to predict pavement performance. Continuing model development should remove the limitations now associated with the prediction model and allow it (or similar models for a given locality) to be incorporated into the pavement system PAVER.

USE OF MODEL IN PMS

Pavement performance prediction models will greatly enhance the usefulness of the PMS. On both the network and the project levels, pavement prediction models assist in the selection of the optimum maintenance strategies. Budget planning, priority ranking of projects, and inspection scheduling can be organized in such a manner as to maximize user benefits. In addition, at the project level, the prediction model information (NDT data, traffic counts, and so on) can be used to select and aid in design of the most advantageous maintenance and rehabilitation (M&R) alternative.

Effective use of the prediction model requires that the necessary information be included in the PMS. For the pavement network considered, nondestructive testing will have to be performed, traffic counts collected, and construction history determined for each pavement section. This information would then have to be stored in the PMS data bank.

A workable network-level application of the prediction model would be to enable optimal budget expenditures. As Figure 8 shows, the PCI has been found to correlate well with the needed level of M&R (3). Conceptually, as can be seen in Figure 9, the required level of M&R dollars for a given pavement decreases with increasing PCI. The prediction model can be used to determine those pavements that will most need repair or be deteriorating rapidly. By applying timely maintenance to those sections before more costly alternatives are required, spending of available monies can be made optimal. Also, the prediction models can be used for identification of critical pavement sections to plan future pavement inspection schedules.

At the project level, the available information can be used for pavement sections identified as needing repair. Available NDT, traffic, and pavement age data can be used in the design and selection of specific M&R alternatives for a given pavement section. This would allow the engineer to select the alternative that will maximize benefits.

Pavement performance prediction models have important engineering and management applications at both the project and the network levels. The degree of usefulness of the prediction models and the data associated with them, however, depends on how well the engineer uses the information available.

Figure 8. Relationship between PCI and maintenance requirements.

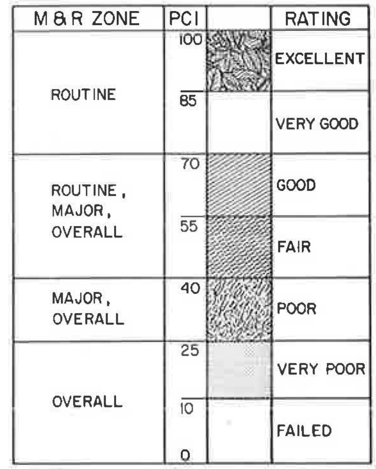


Figure 9. Maintenance cost as a function of pavement condition.

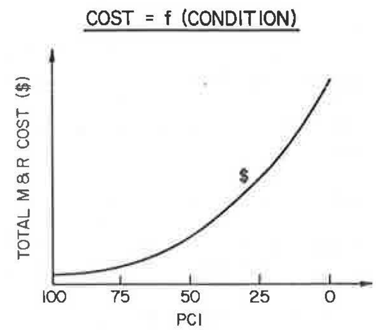
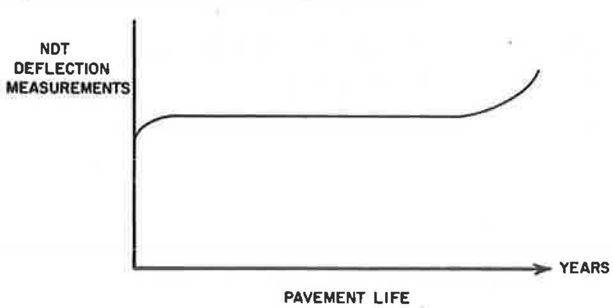


Figure 7. Pavement deflection versus pavement life.



SUMMARY

Based on the findings presented in this paper, it is believed that NDT results do correlate with pavement condition and can be used as a predictor of pavement performance. Using NDT results in addition to other pavement data to accurately predict PCI will allow user benefits to be maximized. Including prediction models based on NDT results in the PMS will permit optimum planning, priority ranking, and scheduling on a long-term basis.

However, before the performance models presented in this paper become practical, further development is required. The possibility of improving the prediction model based on the concept of performance and strength level will be explored. Another application is to simply apply the proven form of the model to data from different locations and redetermine the coefficients for that location. This concept may also be applied to develop a family of curves for performance prediction. Additional data are required for the inclusion of climate factors and other pavement types. Comparisons must be completed to assure that the relationships developed between PCI and NDT results are not device dependent. Once sufficient data have been compiled and analyzed, an accurate pavement performance prediction model will be a welcome addition to any PMS.

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A Model for Predicting Service Life of Flexible Pavement and Its Impact on Rehabilitation Decisions

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A procedure has been developed to estimate the service life of a flexible pavement based on a combination of predicted ride and distress conditions. These conditions are calculated by using equations developed for Texas, taking into consideration measurable values of material properties, climatic conditions, and design factors. Predicted pavement lives were correlated with actual Texas data and acceptable results were obtained. The most significant contributing distress types that affect the service life were identified by using a discriminant-analysis approach. Discriminant functions were developed for each of the prevalent Texas flexible pavements to determine whether the probability of needing rehabilitation is high for calculated levels of ride and distress. An analysis is provided to assess the cost of a delay in rehabilitation once the predicted life has been reached. In this analysis maintenance, user, and rehabilitation costs are taken into consideration. Rehabilitation costs and strategies dependent on pavement condition are modified from those developed for the California pavement management system.

The development and use of a procedure for estimating the service life of an existing flexible pavement in Texas are described; the estimation of service life is based on predicted values of serviceability and distress. A discriminant-analysis approach is used in the development of the model to define the terminal point for rehabilitation.

The study also includes an analysis for assessing the cost of a delay in rehabilitation once the predicted life has been reached. A present-worth and benefit-cost analysis in which rehabilitation, maintenance, and user costs are considered is used in this assessment.

BACKGROUND

The Texas State Department of Highways and Public Transportation sponsored a project to estimate the remaining service life of a flexible pavement. For the purposes of this paper, service life is defined as the total number of equivalent axle loads or the total number of years that the pavement surface lasts, i.e., time or loads between resurfacings. Similarly, the service life of a surface-treated pavement is taken as the time or loads between seals or surface treatments. Following previous work done on flexible pavements in Texas, pavements are classified as asphaltic concrete, overlay, or surface treated for developing the life-prediction models.

An examination of actual data on flexible pavement performance has suggested the following function to represent the loss in serviceability or percentage of distress:

$$g(N) = \exp[-(\rho/N)^{\beta}] \tag{1}$$

where ρ and β are deterioration-rate constants and N is the number of 18-kip equivalent single axle loads (ESALs). Equations for each of the pavement categories have been developed (1) to estimate the deterioration-rate constants for predicting levels of distress and serviceability based on environmental, material, and design properties. The performance equations predict the affected area or degree of severity for each of the following types of distress: rutting, raveling, flushing, corrugations, alligator cracking, longitudinal cracking, transverse cracking, and patching.

Periodic pavement condition surveys have been performed on selected pavement sections in Texas to monitor the serviceability index and the severity and extent of distress. Distress area and severity are rated as none, slight, moderate, and severe, corresponding to numerical ratings of 0, 1, 2, and 3, respectively. In addition these ratings can be converted into percentages of area or severity; for applications reported in this study, 16.6, 33, and 50 percent correspond to ratings of 1, 2, and 3, respectively. This relationship is used in the development of the service life prediction model to numerically express the extent of each type of distress. Once the extent of distress has been estimated, the service life of a pavement can be determined from Equation 1.

MODEL DEVELOPMENT

Discriminant analysis is a statistical technique in which an observation of unknown origin is assigned to one or more distinct groups based on the value of the observation (2). This technique is used to combine the effects of the different distress and performance types that produce a need for pavement rehabilitation.

Essentially the technique discriminates among groups by using a linear combination of the observations. The coefficients of the linear relation are chosen to maximize the ratio of the difference in the means of the linear combination in each of the two groups to its variance (3). Frequently the distance from each individual observation to each of the group centroids, commonly known as Mahalanobis' D^2-statistic (4), is used as the criterion for classification purposes. This smallest distance dictates the assignment rule and may be stated as follows:

$$D_j^2(x) = (x - \bar{x}_j)^T S_j^{-1} (x - \bar{x}_j) + \ln|S_j| - 2\ln(r_j) \quad (2)$$

where

$D_j^2(x)$ = generalized squared distance from observation x to group j,
x = vector of variables in an individual observation,
$\bar{x}_j$ = vector of means of variables in group j,
S_j^{-1} = inverse of covariance matrix for group j,
$|S_j|$ = determinant of covariance matrix for group j, and
r_j = prior probability of assignment to group j (proportion of observations in group j to total number of observations in all groups).

When the covariance matrices are equal, the quadratic terms cancel because of symmetry and linear equations result for the distance measure. The variables used in this study are the serviceability index and the area and severity of distress, which are shown by using values of 0, 1, 2, or 3. To obtain the observations for the analysis, a sample of sections was used for which condition survey information was available for the years 1973-1978 for each of the pavement types. The observations were classified into two groups: those that had been resurfaced during the 1973-1978 period and those that had not. Results from the 1977 condition survey or those of the years preceding a decision to rehabilitate (resurface) were used to describe each section.

Discriminant analysis was used to determine which of the types of distress or serviceability index were the best indicators of a decision to resurface and how they were weighted relative to one another. The decision to resurface in terms of discriminant analysis is a decision to assign a particular section of pavement to the group of pavements that need resurfacing.

In order to obtain an effective assignment rule--that is, one with a low error rate--the variables must provide information about the two populations, which enables assignments to be made. The complexity of the discriminant function may be reduced because the set of variables used is limited to those that contribute the most to the assignment of the observations into the two groups. A regression analogy, credited to Cramer (5) and applicable to discriminant analysis with two groups, allows the problem to be treated as a multiple regression with the creation of a dummy variable as indicator of group membership. To accomplish this, a new variable (y_i) is defined by one of the following equations:

$$y_1 = n_2/(n_1 + n_2) \quad \text{if } x_i \text{ is a member of group 1} \quad (3)$$

$$y_2 = n_1/(n_1 + n_2) \quad \text{if } x_i \text{ is a member of group 2} \quad (4)$$

where

y_i = dependent variable for observation i,
n_1 = number of observations in group 1, and
n_2 = number of observations in group 2.

This substitute variable made it possible to examine all of the linear regression relations among the dependent and independent variables. The model with the smallest mean-square error was chosen to provide the set of variables (distress types or serviceability index) that are used in the discriminant function. An alternative approach to this one could have used a forward or backward stepwise regression model, available in many standard computer software packages. However, the procedure used here was believed to be superior to the stepwise procedure because the order that the variables enter into the model does not affect the final set of variables.

Table 1 gives the distress types that proved to be the best indicators of the need to resurface each of the three pavement types. The number of variables used in the model is greatly reduced for each of the pavement types. Interestingly, the pavement serviceability index (PSI) was chosen only for the overlaid pavements. This corresponds to the widely held opinion in Texas that pavements are rehabilitated mainly because of existing distress rather than the quality of the ride. The set of variables for each pavement type includes at least some of the most important distress types causing serious surface deterioration, such as alligator cracking and longitudinal and transverse cracking.

By using the variables listed in Table 1, discriminant functions are developed to identify pavement sections in need of resurfacing. Hypothesis testing of the covariance matrices of the two groups (resurfaced and not resurfaced) revealed that they are not statistically equal, resulting in quadratic discriminant functions, which are more appropriately handled by a computer program. The classification performance of the models is found to be acceptable by examining the number of correct assignments made with the test data. The results of this analysis are displayed in Table 2.

Linear approximations of the discriminant functions for each pavement type are given in Table 3. However, an examination of the number of correct predictions made by the linear functions, given in Table 4, shows the superiority of the quadratic functions in identifying sections that belong to the group of resurfaced pavements.

It may be noted that a limited number of observations existed for resurfacing in the asphalt-concrete and overlay categories. The resulting functions may be somewhat biased because of this. However, the results given in Table 2 demonstrate that the models are fairly good discriminators.

Table 1. Serviceability and distress types selected for discriminant analysis.

Pavement Type	Serviceability or Distress Type
Asphalt concrete	Alligator-cracking severity
	Longitudinal-cracking severity
	Longitudinal-cracking area
	Transverse-cracking severity
Overlay	Pavement serviceability index (PSI)
	Alligator-cracking area
	Longitudinal-cracking severity
	Longitudinal-cracking area
Surface treated	Rutting severity
	Rutting area
	Longitudinal-cracking severity
	Transverse-cracking area
	Patching area

Table 2. Number of observations correctly predicted by quadratic discriminant functions for three types of pavement.

Pavement Type	Group	No. of Cases	Correct Predictions No.	Correct Predictions Percent
Asphalt concrete	Resurfaced	5	4	80.0
	Not resurfaced	76	71	93.4
Total		81	75	92.6
Overlay	Resurfaced	16	10	62.5
	Not resurfaced	66	60	90.0
Total		82	70	85.4
Surface treated	Resurfaced	56	39	69.6
	Not resurfaced	77	62	80.5
Total		133	101	75.9

Table 3. Linearized discriminant functions for three types of pavement.

Pavement Type	Variable	Group Resurfaced	Group Not Resurfaced
Asphalt concrete	Constant	-8.1085	-0.5903
	Alligator-cracking severity	1.8279	0.4700
	Longitudinal-cracking severity	-4.6091	0.3768
	Longitudinal-cracking area	5.6596	0.3020
	Transverse-cracking severity	2.4684	0.3330
Overlay	Constant	-13.7967	-13.0364
	PSI	5.8827	6.6062
	Alligator-cracking area	2.0832	1.3395
	Longitudinal-cracking severity	-1.3448	0.6792
	Longitudinal-cracking area	3.4315	0.1462
Surface treated	Constant	-5.9554	-4.7224
	Rutting severity	3.2586	2.4665
	Rutting area	2.3691	3.0307
	Longitudinal-cracking area	1.4207	1.0717
	Transverse-cracking area	1.2046	0.4998
	Patching area	0.9275	0.4040

DESCRIPTION OF LIFE-PREDICTION MODEL

The serviceability and distress performance equations are used with the discriminant functions to predict the life of a section of pavement. As aging occurs or loads accumulate, signs of distress become evident and the serviceability index may decrease. At the point where the equations predict a change in the condition rating, the overall rating for each of the corresponding distress and serviceability variables is evaluated by the corresponding discriminant function. This process continues until the probability of being assigned to the group of pavements in need of resurfacing reaches or exceeds a specified value. Because the goal of the model is to determine when a pavement is in need of rehabilitation, which may be considered a critical decision, a relatively high assignment probability is warranted. The probabilities used in the model are 0.70, 0.70, and 0.80 for asphalt-concrete, overlaid, and surface-treated pavements, respectively. The probability for assigning an observation to a group was described by Eisenbeis and Avery (4) as follows:

$$P[j/x] = \exp[-0.5\, D_j^2(x)] / \sum_K \exp[-0.5\, D_K^2(x)] \quad (5)$$

In Equation 5, P[j/x] is the posterior probability that observation x belongs to group j.

The translation of pavement life from 18-kip ESALs into time is accomplished by using annual average daily traffic (AADT), estimated traffic growth, percentage of trucks, and truck traffic information from 1980 W-4 and W-5 tables with the AASHTO procedures (6). Assuming a linear traffic growth rate, the following expression relates time to the accumulated load:

$$A = N_0\, [I + 0.5\, (G)\, I\, (I - 1)] \quad (6)$$

where

N_0 = yearly 18-kip equivalent at time 0,
I = number of years,
G = annual growth rate, and
A = accumulated 18-kip ESALs.

Table 4. Number of observations correctly predicted by linear discriminant functions for three types of pavement.

Pavement Type	Group	No. of Cases	Correct Predictions No.	Correct Predictions Percent
Asphalt concrete	Resurfaced	5	3	60.0
	Not resurfaced	76	73	96.1
Total		81	76	93.9
Overlay	Resurfaced	16	8	50.00
	Not resurfaced	66	61	92.42
Total		82	69	84.1
Surface treated	Resurfaced	56	29	51.8
	Not resurfaced	77	64	83.1
Total		133	93	69.9

Results produced from the life-prediction model were correlated with actual data from Texas pavements. The statistical findings from regression and correlation analyses are shown in Figures 1, 2, and 3.

The resulting regression lines are close to the desired zero intercept with a slope of 1 (a 45-degree line on the graphs). With correlation coefficients in the range of 0.5 to 0.6, about 26 to 37 percent of the variation in the actual service life is accounted for by the linear relationship. However, an examination of the F-values (9.9 to 14.6) reveals that a significant amount of the variation in the response variable (actual life) is accounted for by the linear model. Although these results may not be extremely impressive, they are promising, especially because there are many variables in the decision process for determining when a pavement should be resurfaced, including the availability of funding, which may or may not be related to the need for resurfacing.

COST ANALYSIS

Two basic functions are accomplished by the cost analysis to be described: assessment of the cost of delaying the predicted rehabilitation by using a present-worth analysis and provision of a benefit-cost ratio to help justify the proposed rehabilitation. The analysis includes rehabilitation and maintenance costs and benefits due to savings in fuel consumption, travel time, and reduced maintenance.

Rehabilitation costs are dependent on the strategies used, which are dictated by the principal cause of the resurfacing. The strategies used in this model are customized versions of those suggested by the California pavement management system (7) and appear in Table 5. As part of the customizing, the alternatives have been stated in terms of the scores obtained from the condition survey. The alternative

Figure 1. Actual versus predicted performance for asphalt-concrete pavements.

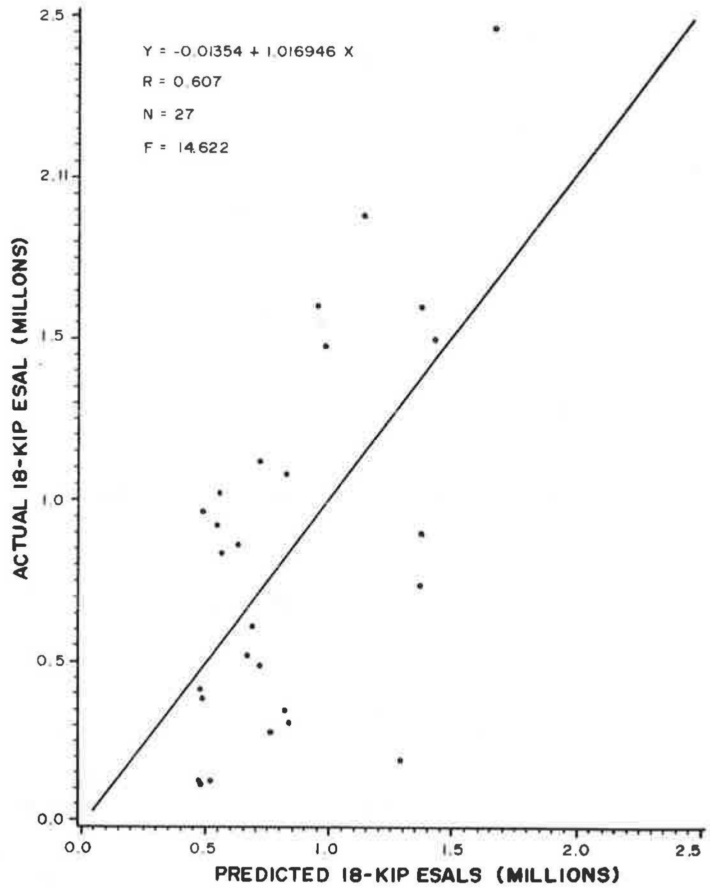


Figure 2. Actual versus predicted performance for overlaid flexible pavements.

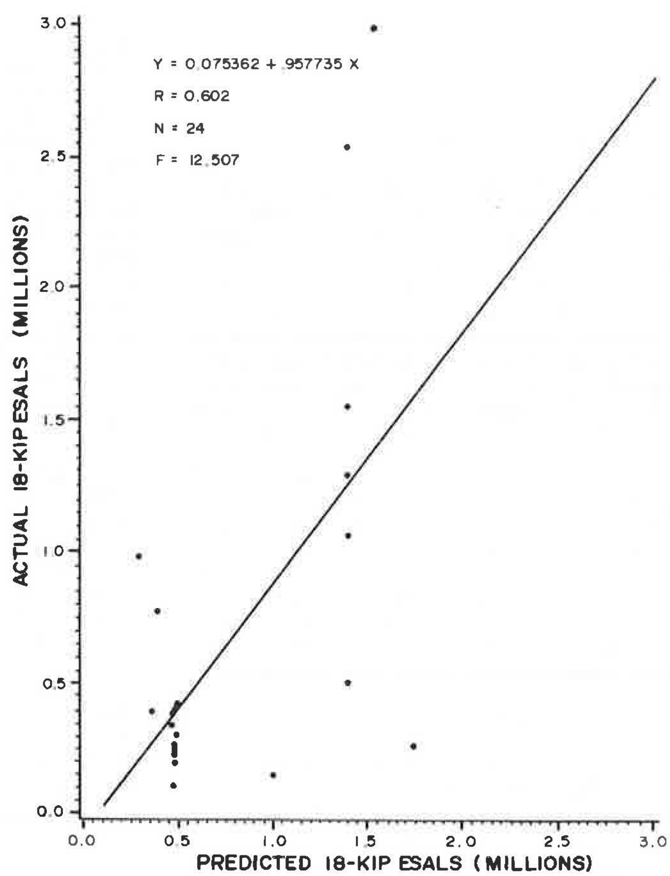


Figure 3. Actual versus predicted performance for farm-to-market surface-treated pavements.

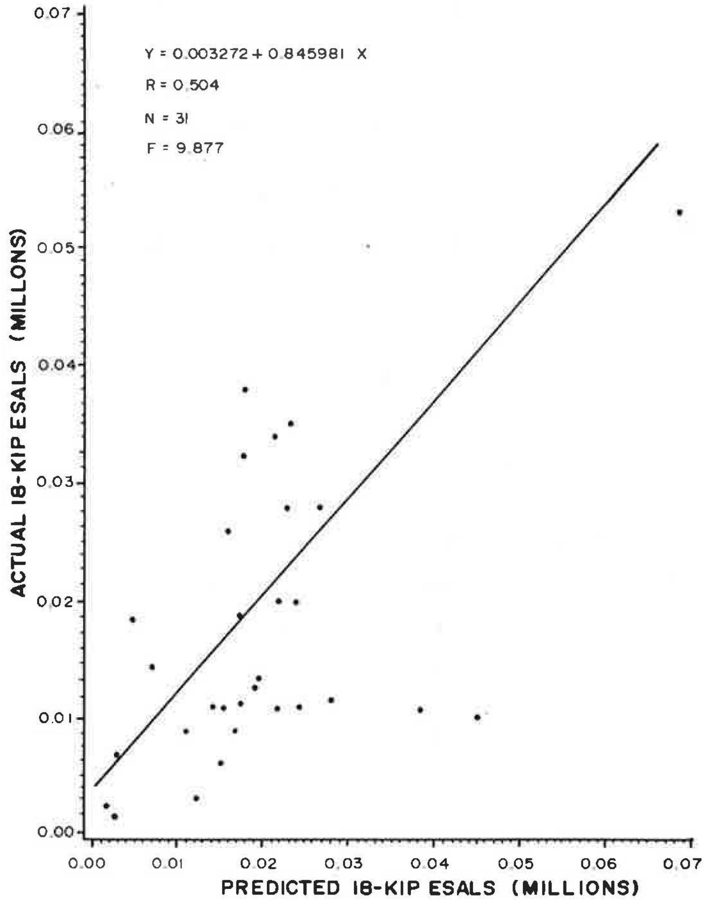


Table 5. Rehabilitation strategies for three types of pavement.

Pavement Type	Cause	Strategy by Condition: Slight	Moderate	Severe
Asphalt concrete	Alligator cracking	Fill cracks	1-in. overlay and local dig-out	5.0-in. overlay
	Longitudinal and transverse cracking	Do nothing	Fill cracks	Rubberized asphalt chip seal
Overlay	Alligator cracking	Fill cracks	1-in. overlay and local dig-out	5.0-in. overlay
	Longitudinal cracking	Do nothing	Fill cracks	Rubberized asphalt chip seal
	PSI < 2.9	Leveling and 1-in. overlay	Leveling and 1-in. overlay	Leveling and 1-in. overlay
Surface treated	Rutting	Seal coat	Double seal coat	Sectional reconstruction
	Longitudinal and transverse cracking	Do nothing	Do nothing	Fill cracks
	Patching	Do nothing	Seal coat	Double seal coat

is matched to the predicted condition for each applicable distress and serviceability type, and the most costly strategy is chosen to be the cost of rehabilitation.

Pavement maintenance costs are assumed to increase with pavement age. For lack of a more precise model developed for Texas, the EAROMAR (8) equations were used, even though they were developed to predict maintenance workloads for multilane freeways. The model is as follows:

$$C_t = (1{,}100C_1 + 1{,}000C_2 + 5C_3)/\left\{1 + \exp[(t - 10)/1.16]\right\} \quad (7)$$

where

C_t = annual maintenance cost (yr/lane mile),
C_1 = bituminous skin patching (\$3.47/yd²),
C_2 = crack sealing (\$0.25/linear ft), and
C_3 = bituminous base and surface repair (\$450/yd³).

For highway types other than freeways, the EAROMAR results are appropriately modified by multiplying them by a reduction coefficient reflecting past maintenance data for Texas. The results of a comparison of maintenance costs for farm-to-market and U.S. and state highways with those for Interstate routes in Texas (9) is as follows. (As an illustration, the maintenance cost on farm-to-market roads is 38.2 percent of the cost per lane mile computed by the EAROMAR equations.)

Highway System	No. of Observations	Avg Maintenance Cost per Lane Mile ($)	Percentage of Interstate Cost
Interstate	4	1,028.00	
Farm to market	23	391.00	38.2
U.S. and state	62	325.00	31.6

The present-worth analysis focuses on the rehabilitation strategy and on the annual maintenance during the analysis period, which corresponds to the service life of the rehabilitation strategy, which in turn is determined by the service-life prediction model. Costs of delaying rehabilitation beyond the predicted end of a pavement's life are calculated for delay periods from 1 to 5 years. In order to compare the alternatives over equal time spans, the unused value of the rehabilitated pavement is taken into consideration. The present worth of delaying rehabilitation may be expressed as follows:

$$PW = \sum_{n=1}^{r} [C_n \cdot (P/F_{i,n})] + \left\{ R_c \cdot (A/P_{i,m}) + \sum_{n=1}^{m} [C_n \cdot (P/F_{i,n}) \cdot (A/P_{i,m})] \right\} [(P/A_{i,m-r}) \cdot (P/F_{i,r})] \quad (8)$$

where

- i = interest rate,
- R_c = rehabilitation cost,
- m = analysis period,
- r = year in which rehabilitation occurs,
- C_n = maintenance cost in year n,
- $A/P_{i,m}$ = equal-payment-series capital recovery factor = $i(1 + i)^m/[(1 + i)^m - 1]$,
- $P/A_{i,n}$ = equal-payment-series present-worth factor = $1/(A/P_{i,m})$,
- $P/F_{i,n}$ = single-payment present-worth factor = $1/(1 + i)^n$.

The unused value may be expressed as follows:

$$U = R_c \cdot (A/P_{i,m}) \cdot (P/A_{i,r}) \quad (9)$$

BENEFIT-COST ANALYSIS

In this section, a model is constructed to evaluate the benefits resulting from reductions in user and maintenance costs due to increased serviceability. The resulting benefit-cost analysis is useful in relating the probability of a proposed alternative to its cost. Two types of user costs are considered, fuel consumption and travel time, because these represent disbursements on the part of the user in contrast to more subjective abstract costs, such as those for discomfort. In addition, accident costs and vehicle operating costs, often considered in an analysis of this type, were not included for lack of an adequate model to relate them to serviceability or the distress types mentioned previously. Fuel consumption costs and travel time costs were estimated for different levels of the serviceability index as predicted by the corresponding performance equation. To calculate the benefits derived from increasing the serviceability, a concept illustrated in Figure 4 is used (10). The underlying assumption is that costs increase with pavement age up to a point, and resurfacing (point G) updates the age and returns the cost structure to zero. The benefits would be the difference between the cost under the assumption that no improvements were made and the cost under the assumption that an improvement takes place. This benefit is represented by the region BDEG in Figure 4 for a time span of N years.

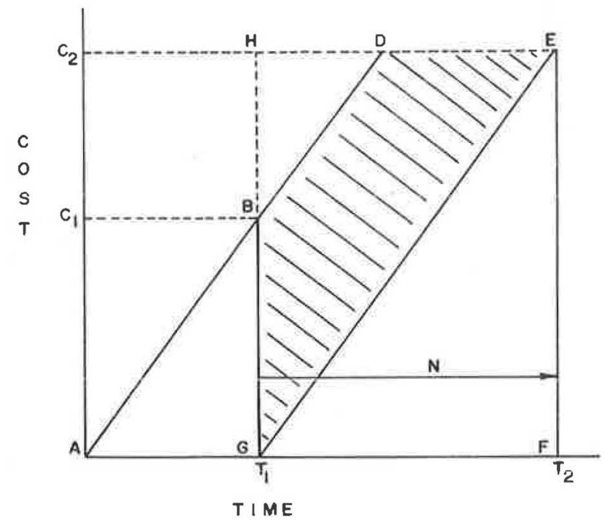
With this concept the following equation was developed to calculate the benefits derived from fuel savings:

$$B_f = \left\{ [(F_2 N)/2] - [(F_2 - F_1)/2] \, [N - N(F_1/F_2)] \right\} \times [(AADT)(L_s)(365)(1/12)(CG)(P/A_{i,n})]/N \quad (10)$$

where

- B_f = present worth of benefits from fuel savings due to resurfacing,

Figure 4. Concept of time versus cost (10).



- F_2 = maximum percentage of reduction in fuel costs (1.5 percent) due to resurfacing,
- F_1 = percentage of reduction in fuel costs based on PSI before resurfacing,
- N = service life,
- CG = cost of a gallon of gasoline, and
- L_s = length of section.

The percentage of reduction in fuel use as shown by Ross (11) is given by the following:

$$F_1 = 0.0001879 \cdot (PSI_A - PSI_B) / [0.043771 - (0.0001879 \cdot PSI_A)] \quad (11)$$

where PSI_A is the serviceability index after resurfacing and PSI_B is the serviceability index before resurfacing.

It should be noted that this equation yields results considerably different from those interpolated from Claffey's work (12); the maximum difference is about 30 percent. To illustrate the magnitude of benefits per mile derived from fuel savings, an AADT of 1,000 vehicles may be assumed together with a service life of 8 years, an annual interest rate of 10 percent, and a PSI before resurfacing of 2.5. The present worth of benefits due to fuel savings for this example would be calculated as follows:

$$F_1 = 0.0001879 \cdot (4.7 - 2.5)/[0.043771 - (0.0001879)(4.7)] = 0.0096 \quad (12)$$

$$B_f = \left([(0.015)(8)/2] - [(0.015 - 0.0096)/2] \left\{ 8 - [8(0.0096) \div 0.015] \right\} \right) \left\{ [(1{,}000)(365)(1.20)(5.3349)]/[(12)(8)] \right\} = \$1{,}271.00 \quad (13)$$

For time savings, the equation used for calculating benefits is as follows:

$$B_t = (T_a - T_b) \, AADT \cdot V \cdot (P/A_{i,n}) \quad (14)$$

where

- B_t = present worth of benefits from time savings due to resurfacing,
- T_b = travel time before resurfacing,
- T_a = travel time after resurfacing, and
- V = value of time per hour.

Speed increases due to resurfacing are as follows (10,13):

Speed Limit (mph)	Increases (mph) by PSI 2.6-2.11	2.1-1.81	1.8-0
25-30	0	0	2
35-40	0	2	4
45-50	2	4	6
55	4	6	8

As an illustration, for the previously stated example and a speed limit of 55 mph with a $6.00 delay cost per hour, the benefits due to time savings would be calculated as follows:

$$B_t = [(1/51) - (1/55)]\ (1{,}000)\ (6)\ (5.3349) = \$16{,}661.00 \qquad (15)$$

Benefits derived from reduced maintenance costs (B_m) are estimated by calculating the present worth of the difference between maintenance costs when there is no resurfacing and those when resurfacing takes place. Because maintenance costs are calculated as a function of pavement age, resurfacing updates the age of the pavement and thus reduces costs.

As an illustration, a two-lane state highway may be assumed with a 10-year-old pavement at the time of rehabilitation. It is further assumed that the new surface lasts 8 years. Savings in maintenance costs would be calculated as follows:

$$B_m = 2[1{,}100\,(3.47) + 1{,}000\,(0.25) + 5(450)] \times \left(\sum_{i=1}^{8} [1/1 + \exp(-i/1.16)] - \left\{1/1 + \exp[-(i - 10)/1.16]\right\}\right) = \$18{,}821.00 \qquad (16)$$

The total benefits for fuel savings, time savings, and reduced maintenance for this example are $36,753.00/mile.

Costs for the benefit/cost (B/C) ratio are those of the rehabilitation strategy discussed previously, which yields the following relationship:

$$B/C \text{ ratio} = (B_f + B_t + B_m)/R_c \qquad (17)$$

Assuming in the example that the principal cause of rehabilitation is a moderate level of alligator cracking, the cost of rehabilitation as modified from the California method is given by the following:

$$R_c = [L_s\,(N + 0.67) \cdot CJ + (L_s)\,(N)\,(0.05)\,(CE)] \cdot 1.2 \qquad (18)$$

where

L_s = length of project (1 mile),
N = number of lanes (two),
CJ = cost of 1-in. overlay per lane mile ($10,000.00), and
CE = cost of base repair and patching per lane mile ($140,000.00).

The resulting rehabilitation cost is $56,000.00 with a B/C ratio of 0.64. Delaying this project until a more costly rehabilitation strategy must be taken may result in a lower B/C ratio. However, if the same strategy applies, benefits will increase and the ratio will increase; this makes the project more competitive with other projects.

The negligible savings in fuel contributes to making this project unfavorable in the light of a benefit-cost analysis. Previous studies (10) and studies in other countries (14) suggest a stronger influence (larger benefits) of savings in fuel in the determination of total benefits.

SUMMARY

A model has been described that is capable of predicting the service life of a flexible pavement section and of evaluating the effects of a prompt or delayed decision in taking rehabilitation actions. The model combines discriminant analysis, a statistical technique, with performance equations developed for Texas conditions and produces results that compare favorably with actual data collected on sections of the state's highway system.

A framework is provided to perform present-worth analysis based on maintenance and rehabilitation costs. The period for this analysis is assumed to be the life of the surfacing, which in turn is predicted by the life-prediction model, and the rehabilitation strategies are also generated internally. A benefit-cost analysis is provided to supplement the present-worth analysis. This model is expected to aid the state in planning or programming future expenditures on pavement rehabilitation projects.

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Notice: The opinions expressed in this paper are those of the authors and not necessarily those of the Texas State Department of Highways and Public Transportation.

Abridgment

Field Investigation of Resource Requirements for State Highway Routine Maintenance Activities

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The first phase of a comprehensive study to identify potential cost and energy savings in routine maintenance activities on the state highway system in Indiana is described. In this phase the current highway routine maintenance standards of the Indiana Department of Highways were reviewed and updated based on data collected in the field, and guidelines for estimating equipment fuel consumption were established. The needs for different resources (materials, labor, and equipment) used in various routine maintenance activities (types, rates of consumption, and frequencies of use) were identified. Energy consumed in each activity was determined as the number of gallons of fuel required to produce one production unit of an activity. The preliminary data analysis indicated that there is a potential for considerable cost and energy savings through better assignment of equipment in different activities. The information developed in this phase can be used directly by the Indiana Department of Highways in preparing their annual maintenance program.

Inflation and price increases have significantly affected the routine maintenance expenditures for the state highway system in Indiana. For example, the total expenditure on routine maintenance activities in 1976 was $47 million, whereas in 1981 the expenditure increased to about $70 million (1,2).

The recent increase in price for all petroleum-related materials includes such derivatives as motor fuel, asphalts, and tars. Motor fuel is the material with the greatest price increase, and it is critical to any maintenance activity because of the dependence of the equipment fleet on it. For instance, the maintenance equipment fleet of the Indiana Department of Highways (IDOH) consumed about $2.6 million worth of motor fuel in 1976, and in 1981 this increased to about $6.0 million. In addition the portion of total material costs assigned to motor fuel has increased with time; for example, 18 percent of the total material costs was assigned to motor fuel in 1976 as opposed to 28 percent in 1981.

From the foregoing observations it is evident that motor fuel must be considered a special resource that needs to be controlled. This can be achieved only through detailed information on equipment use and associated fuel consumption. Many studies have been initiated in the past on the general topic of energy use by maintenance equipment (3-10). However, the information available does not provide either the degree of variability of fuel consumption among different equipment types or the variability of fuel consumption by the same equipment type when used in different maintenance activities. Furthermore, the current standards of equipment use by IDOH are measured by the number of hours or miles for which a piece of equipment is used. These measures cannot provide useful information about fuel consumption unless other supporting rates are developed. Such rates as miles per gallon and gallons per hour are useful in recognizing the amount of fuel consumed as well as the degree of use of a piece of equipment.

The objective of the study reported in this paper is to update the current standards of maintenance resource needs and to establish new standards for fuel consumption by maintenance equipment. This information can then be used in efforts to achieve maintenance cost and energy savings. The study was sponsored by the Federal Highway Administration and IDOH and the results obtained will be of use to IDOH in programming routine maintenance activities.

STUDY METHODOLOGY AND DATA-COLLECTION PROCEDURE

The existing system of maintenance data recording was used with some modifications. The current reporting system of IDOH consists of filing work records on a crew day card. Information recorded on such cards includes activity type, location, date, number of crew members and corresponding man hours, equipment used and corresponding miles or hours, materials used and corresponding quantities, and total accomplishment (production units).

For 6 weeks during October through November 1981, data were collected from selected subdistricts representing the six districts of IDOH. This period was considered unique in that most maintenance activities were performed during this time. Nevertheless, some activities could not be included: activities that are not applied at that time of the year (for example, snow and ice removal); activities with low occurrence, such as seal coating; and activities of administrative nature, such as training, stand-by time, and so on.

The current data-recording system by using crew day cards does not include any information about the amount of fuel consumed by different equipment types. Consequently the subdistrict managers were instructed to fill each piece of equipment with fuel before and after each job. The difference was then to be recorded on the same crew day card with other associated activity data.

*Deceased.

The gross sample size was about 1,400 jobs. After a screening process to check the validity of the data, about 200 jobs were excluded.

Forty-nine maintenance activities were covered in this phase of the study (see Table 1 for names of activities). Thirty-nine different materials were found to be the most frequently used in practice. The labor force was grouped into six categories. Seventy-nine different equipment types were used in routine maintenance. Units of measurement for types of fuel-consuming equipment were miles per gallon or gallons per hour. Production of other equipment types was measured by number of miles or hours.

RESULTS OF THE STUDY

Resource requirements for each routine maintenance activity in terms of materials, labor, and equipment were analyzed. In this effort three categories of information were developed:

1. The type of each resource element used in

Table 1. Summary of resource cost analysis (5, 11).

Activity	Unit of Measure	Man Hours per Production Unit	Cost per Man Hour[a] Material	Fuel	Total
Roadway and shoulder					
Shallow patching	Tons of mix	13.0	2.1	0.8	8.7
Deep patching	Tons of mix	5.3	5.2	1.4	12.8
Premix leveling	Tons of mix	3.2	8.3	1.2	15.6
Full-width shoulder sealing	Foot miles	3.7	21.0	2.6	29.7
Sealing longitudinal cracks and joints	Linear miles	8.8	4.2	1.1	11.6
Sealing cracks	Lane miles	25.6	4.1	1.1	11.0
Cutting relief joints	Linear feet	0.5	2.2	3.1	11.4
Spot repairing of unpaved shoulders	Tons of aggregate	1.2	3.4	1.4	10.8
Blading shoulders	Shoulder miles	1.7	0.0	2.1	8.1
Clipping unpaved shoulders	Shoulder miles	26.6	0.0	2.3	8.5
Reconditioning unpaved shoulders	Shoulder miles	27.7	14.7	2.7	23.5
Joint and bump burning	Bumps removed	3.9	0.05	0.7	6.7
Other	Man hours	1.0	4.5	0.7	11.0
Roadside					
Machine mowing	Swath miles	1.1	0.0	1.1	6.9
Brush cutting	Man hours	1.0	0.0	1.0	7.0
Herbicide treating	Man hours	1.0	15.9	1.1	22.9
Seeding and/or fertilizing	Man hours	1.0	0.3	0.3	6.5
Topping, trimming, or removing long trees	Trees	20.8	0.0	1.3	7.4
Stump removing	Stumps	3.7	0.0	1.6	7.6
Spot mowing and hand trimming	Man hours	1.0	0.0	1.1	7.0
Right-of-way fence repairing	Linear feet	0.3	7.2	0.6	13.5
Other	Man hours	1.0	0.0	1.3	7.2
Drainage					
Cleaning and reshaping ditches	Linear feet	0.1	0.0	1.7	6.9
Inspecting minor drainage structures	Structures	0.6	0.0	0.5	6.4
Pipe replacing	Location	58.6	11.5	1.4	18.9
Motor patrol ditching	Ditch mile	42.6	0.0	2.2	8.4
Cleaning minor drainage structures	Structures	4.8	0.0	1.2	7.2
Other	Man hours	1.0	5.4	1.0	12.3
Bridges					
Bridge repairing	Man hours	1.0	2.1	0.7	9.0
Bridge deck patching	Square feet	1.3	0.4	1.2	7.4
Traffic control					
Subdistrict sign maintenance	Man hours	1.0	6.1	1.3	13.6
Painting pavement messages and special markings	Man hours	1.0	2.9	0.9	9.8
Guardrail maintenance	Linear feet	2.0	3.3	0.9	10.2
Other	Man hours	1.0	0.0	1.4	7.3
Winter and emergency					
Emergency maintenance	Man hours	1.0	1.1	1.3	8.5
Stockpiling winter materials	Man hours	1.0	0.0	1.2	6.9
Other	Man hours	1.0	0.2	0.6	6.7
Public service					
Roadside park, rest area, and weigh station maintenance	Man hours	1.0	0.0	0.8	6.4
Work for state institutions	Man hours	1.0	4.7	0.8	11.3
Full-width litter pickup	Right-of-way miles of pass	3.7	0.0	0.5	6.2
Spot litter pickup	Man hours	1.0	0.0	1.1	6.8
Roadway cleaning	Man hours	1.0	0.0	1.1	5.7
Other	Man hours	1.0	0.0	0.6	6.3
Other					
Materials handling and storage	Man hours	1.0	0.0	3.6	9.6
Detour maintenance	Man hours	1.0	0.0	1.0	7.1
Other support activities	Man hours	1.0	0.0	2.7	8.8
Special maintenance	Man hours	1.0	38.8	2.2	47.1
Special maintenance	Man hours	1.0	9.6	0.8	16.7
Special maintenance	Man hours	1.0	16.0	2.4	24.7

[a]All costs are based on 1981-1982 prices.

each activity, for example, the type of materials or equipment that has been found to be used in the field in accomplishing an activity;

2. The frequency of use of each resource element when employed in an activity (use factor) (for example, a use factor of 0.5 means that the corresponding resource element is used 50 percent of the time); and

3. The rate of consumption of each resource element when used in an activity, for example, the number of units of a certain material required to produce one production unit of an activity. Man hours required to produce one production unit of an activity is used as the rate of consumption of the labor resources. The rate of consumption associated with equipment may be the number of gallons consumed by this equipment to produce one production unit of an activity or it may be given in terms of number of miles per gallon or gallons per hour consumed when this equipment is used.

Activity-Material Interactions

The frequency of use (use factor) was defined as follows:

$$f_{ij} = M_{ij}/N_j \tag{1}$$

where

f_{ij} = use factor of material i in activity j,
M_{ij} = total number of times material i was used in all jobs of activity j, and
N_j = total number of jobs of activity j.

The rate of consumption of a particular material to produce one unit of an activity was defined in terms of quantity. These rates were calculated as follows:

$$R_{ij} = m_{ij}/P_j \tag{2}$$

where

R_{ij} = rate of consumption of material i when used in activity j,
m_{ij} = total number of units of material i used in activity j, and
P_j = total number of units produced of activity j.

The average material cost per production unit of an activity was estimated from the following:

$$CM_j = \Sigma f_{ij} * R_{ij} * c_j \tag{3}$$

where

CM_j = average material cost to produce a unit of activity j,
f_{ij} = use factor,
R_{ij} = rate of consumption, and
c_j = unit cost of material j (in 1982 dollars).

Activity-Labor Interactions

As stated earlier, six labor categories were found to be used in maintenance activities. The frequency of use (use factor) of each category in each activity was given the value 1 or zero. A value of 1 was given if the category was included in the corresponding activity and the value of zero if not. Finally, the rate of consumption was determined in terms of number of workers in each labor category required to accomplish an activity and in terms of number of man hours needed for one production unit of an activity.

Activity-Equipment Interactions

One of the major thrusts of this study was to provide the maintenance division of IDOH with reliable information concerning equipment fuel consumption. The information is expected to provide the necessary background for developing new standards for equipment costs. The subsequent discussion in this paper will focus only on fuel-consuming equipment types.

The computation of the use factor of a piece of equipment in an activity was similar to the procedure expressed in Equation 1.

Two rates of consumption were considered. The first is the number of gallons of fuel consumed by a piece of equipment to produce one production unit of an activity. This rate was employed directly to calculate the average fuel cost per production unit.

The second rate is concerned with the operational aspect of the equipment. Miles per gallon or gallons per hour are conveniently used for this purpose. The results showed considerable variation between different equipment-activity combinations. That is, not only do different equipment types have different rates of consumption but also the consumption rates for the same equipment type may vary considerably when the equipment is used in different activities. These rates were calculated by using the same procedure presented in Equation 2.

Equation 3 was employed to calculate average fuel cost per production unit of an activity.

RESOURCE COST ANALYSIS

Number of man hours, material quantities, and number of gallons of fuel consumed by maintenance equipment types were estimated for each routine maintenance activity. These data were then used to estimate the cost of each of the resource elements to perform one production unit of each activity. In Table 1, a comparison of resource consumption by different activities is shown. Man hours was chosen as a common unit for this comparison. On this basis, full-width shoulder sealing, reconditioning unpaved shoulders, and herbicide treating are the most material-consuming activities. On the other hand, cutting relief joints, materials handling and storage, and other support activities are the most fuel-consuming activities.

Statistical tests showed that the average rates of consumption of labor, materials, and fuel are significantly different from one location (subdistrict) to another.

IMPLICATION OF RESULTS

The detailed information on resource requirements for various maintenance activities can be used for a systematic evaluation of areas in which cost and energy savings can be achieved. For example, the variability of fuel consumption between different equipment-activity combinations observed in this study indicates that considerable savings can be obtained through better management of equipment. An illustration is activity 272 (roadside park, rest area, and weigh station maintenance). The current standards specify that a dump truck be used for this activity. However, the field observations made in this study indicated that a dump trick was used in only 50 percent of the jobs, whereas a flatbed truck was used in the other 50 percent.

The average number of gallons consumed by a flatbed truck to produce one production unit of activity 272 is 0.5, whereas 0.25 gal is consumed by a dump truck for the same purpose. It is clear that 0.25 gal could have been saved per production unit each time a flatbed truck was used instead of a dump truck. Considering the total production for activity 272 in 1981 (21,056 man hours), about 2,600 gal could have been saved during fiscal year 1981. Similarly it was found that an extra 5,908 gal was consumed as a result of using a dump truck in 18 percent of all jobs of activity 276 (spot litter pickup), whereas a pickup truck could have been used in the operation of this activity.

These examples were two of many cases in which the actual frequencies of equipment use deviated from IDOH standards. The savings mentioned are based on these deviations. However, the deviation observed might have been caused by the inherent nature of the jobs performed. In this case the current equipment use standards may be updated to comply with the actual field requirement and to help in better monitoring and evaluation of field work.

ACKNOWLEDGMENT

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Abridgment

Determining Maintenance Needs of County Roads and City Streets

WES WELLS

Two types of street and road maintenance needs in the San Francisco Bay Area are documented: ongoing maintenance, or what is necessary on an annual basis to keep roads in adequate condition, and backlog costs, or what is necessary to bring roads back to adequate condition that had deteriorated due to deferred maintenance. Estimates of need for both types of maintenance are then compared with actual expenditures for the Bay Area's 9 counties and 92 cities to determine funding shortfalls. It was found that the local road system was not being adequately protected. Ongoing maintenance expenditures only covered about 60 percent of what was needed. Seventy-five percent of the shortfall was in preventive maintenance. This deferral of maintenance had led to a backlog of road deterioration by which 20 percent of the roads were classified as being in fair to poor condition. These findings led to three major recommendations: maintenance practices needed to be improved, the problem needed to be communicated to the public, and more revenue was required. Significant steps have subsequently been initiated for all three types of maintenance. A simple and straightforward method of measuring need is presented, not to generate project-level decisions but to provide ballpark estimates of aggregate revenue requirements. The methodological and technical study was extended to an action program to carry out the three recommendations. Popular summary reports, a slide show, legislative principles, and actions to improve maintenance practices have all been subsequently developed.

The San Francisco Bay Area includes roughly 5 million of California's 23 million people in 9 counties and 92 cities. The largest cities are San Jose in the South Bay, Oakland and Berkeley in the East Bay, and San Francisco in the West Bay. These four cities had dominated the Bay Area, sustaining almost 70 percent of the region's population through 1940. By contrast, the four northern counties have had the bulk of their growth occur in the last 30 years. Santa Clara County, in the south, has increased its population by 50 percent in just the last two decades.

The Bay Area has more than 17,000 miles of city streets and county roads. This represents more than 92 percent of all roads in the region after the 1,400 miles of state highways are included. Roughly one-fourth of the local system is contained in Santa Clara; another one-fourth in the four northern counties of Marin, Napa, Solano, and Sonoma; and the remainder in the four central counties of Alameda, Contra Costa, San Mateo, and San Francisco.

FINANCING LOCAL STREETS AND ROADS

Since 1963 Californians have been taxed 7 cents per gallon of gasoline. Roughly half of this amount is returned to cities and counties to be used for streets and roads. About one-third of total revenues comes from the gasoline tax, another one-fourth from cities' general funds, and the remainder from a variety of smaller sources.

Several factors have affected these revenues in recent years, all in a negative direction. Most important, the tax rate has not changed in 20 years. In addition, the distribution formula by which the local portion of the 7-cent/gal tax is allocated has also not changed in 20 years; the formula did not anticipate the tremendous shift in population in the 1950s, 1960s, and 1970s into cities, which negatively affected the cities to an even greater extent. Gasoline tax revenues have also been declining since FY 1978-1979 as California gasoline consumption dropped from 11.9 billion gal that year to 11.3 billion the next. This trend is projected to continue. Finally, in 1978 California taxpayers greatly curtailed property taxes and their rate of increase, which significantly reduced cities' general funds.

With the funding scenario just noted, significant pressure was placed on California legislators to help solve the funding shortage. However, no study had been done that attempted to specifically measure maintenance requirements in dollars and then compare these required costs against what was actually being spent.

In 1981 the California Legislature responded to the concerns being raised by introducing a bill to increase the gasoline tax by 2 cents/gal; 1 cent/gal was to come back to local jurisdictions for their road systems. This bill served as the catalyst in the mobilization of Bay Area public works directors both to support the bill and to actually document the real magnitude of the local street and road maintenance need. This effort led the Metropolitan Transportation Commission (MTC), in conjunction with other public works and engineering officials, to initiate the study discussed here. The purpose was threefold: to develop for the public and the legislature accurate information on local street and road conditions, including current and projected maintenance revenues and expenditures; to define ongoing and backlog maintenance requirements; and to provide a realistic assessment of the problem together with funding requirements.

STUDY APPROACH AND DESIGN

Major Analytic Tools

In order to determine what was being spent for road maintenance, what ought to be spent, and the resultant revenue gap or shortfall, it was necessary to acquire information from two major sources and to develop and cost a preventive maintenance program. A questionnaire was necessary to measure both what jurisdictions were currently spending for road maintenance and what they thought they should be spending. The questionnaire also measured existing street areas by functional type and existing preventive treatments and cycles and identified major revenue sources.

A pavement condition survey (PCS) was required in order to measure actual backlog costs. The intent of the survey was to develop reasonable estimates of the extent of road deterioration in sufficient jurisdictions to permit ballpark estimates of county and regional backlog costs. Six types of distress were recorded after visual inspection of pavements: transverse, longitudinal, and alligator cracking as well as raveling, maintenance patching, and rutting. Inspectors also recommended corrective treatments, if necessary, ranging from routine spot repairs through restoration.

In order to measure cyclical maintenance needs, a treatment process needed to be defined that would keep roads adequately maintained, much like the scheduled maintenance for an automobile. A standard process was developed for arterials, collectors, and local access roads. For arterials, a slurry or chip seal without a fabric interlayer is applied in the 7th, 21st, and 35th year. A 1.5-in. overlay with fabric is applied in the 14th and 28th years. And finally the arterial is restored in its 42nd year. Similar processes were developed for collectors and local access but with much longer life cycles, 63 and 105 years, respectively.

The standardized maintenance cycle was then applied to the inventory of the region's streets and roads. The estimated age of the streets and roads was used to determine the required treatment. In this way the actual cyclical maintenance costs in any one year could be calculated. In addition, by modifying the actual treatments or the cycles, alternative cyclical strategies and their associated costs were analyzed.

To facilitate the development of the questionnaire, the survey, and the cyclical treatment process, a technical advisory committee was established. This 24-member group was composed primarily of public works directors and local engineers knowledgeable in road maintenance. Besides helping to design the three major analytic tools just discussed, this group proved invaluable as a catalyst for the participation of other local departments in the data-gathering efforts. The working knowledge of the committee on how road description data, cyclical treatment programs, and maintenance revenue and expenditure data were recorded enabled the questionnaire to closely reflect local information files.

In order to accurately document the total road maintenance needs of Bay Area jurisdictions, it was necessary to measure both backlog and ongoing maintenance needs.

Backlog Maintenance Needs

To determine backlog maintenance needs, information from three major sources was required.

Actual Measurement of Pavement Condition

The PCSs were conducted in 11 Bay Area cities and counties. In all more than 7,200 street segments representing more than 1,500 miles of local roads were sampled. MTC handled all data processing. Training classes were conducted in early April 1981. All participating jurisdictions had returned their completed forms by the end of May.

A pavement condition index (PCI) was calculated by using the extent and severity of pavement distress scores from the six distress types. The range of scores varied as follows depending on the extent (percentage of area exhibiting distress) and severity (none, slight, moderate, severe):

1. Transverse cracking, 0-12;
2. Longitudinal cracking, 0-20;
3. Alligator cracking, 0-50;
4. Raveling or surface wear, 0-20;
5. Patching or maintenance repair, 0-10; and
6. Rutting or corrugations, 0-16.

Therefore, the best possible PCI was zero and the worst possible was 128.

Derivation of Recommended Maintenance Treatments

The field surveyors also assigned one of nine proposed maintenance treatments to each road segment. MTC, through an analysis of the initial correlation between PCI and recommended maintenance treatments, was able to assign a specific maintenance treatment based on each segment's PCI. This tended to reduce variations across jurisdictions. The assignment process was roughly as follows: no treatment, PCI 0-19; seal, PCI 20-39; overlay, PCI 40-69; and restoration, PCI 70+.

Derivation of Costs for Corrective Treatments

Costs were developed for each of the four major maintenance treatment categories based on the individual treatment costs developed by the technical committee. These estimates represented averages from current maintenance contracts.

Ongoing Maintenance Needs

To determine ongoing maintenance needs, information was needed on what was currently being spent and what ought to be spent.

What Was Being Spent

A questionnaire was sent to each of the Bay Area jurisdictions asking them to report actual and budgeted road maintenance expenditures. Sixty-four jurisdictions responded, representing all of the 9 counties and 55 of the 92 cities. This represented all of the county road mileage and 87 percent of the city street mileage. Expenditures were broken down into four general maintenance categories: cyclical, e.g., preventive maintenance such as seals, overlays, and restoration; routine, e.g., patching and crack repair; nonpavement, e.g., lighting and cleaning; and other, e.g., special programs and administration.

What Ought To Be Spent

The determination of desirable maintenance expenditures was made in two ways. For the latter three categories (routine, nonpavement, and other), average reported costs per mile were calculated for each reporting jurisdiction. These were defined as expenditures necessary for acceptable maintenance. These estimates are not based on standards and are therefore subject to the judgment of the public works directors.

The determination of desirable expenditure for cyclical maintenance was based on the development and application of the standard preventive maintenance treatment process. The MTC questionnaire asked jurisdictions to show the cyclical treatment techniques that they believed could be used to sustain their streets and roads in acceptable condition. The average procedure was then modified by the technical committee to create a standard maintenance cycle.

The treatments were applied to miles of pavement as dictated by pavement age. This required an estimate of pavement miles by functional class by year built. To determine required cyclical costs for 1980, for example, the cyclical treatments had to be matched to the estimated miles of roads built in specific years. Because all treatments were applied in 7-year cycles, required treatments were matched to the estimate of miles of road built in 1903 (11th treatment, or a slurry or chip seal without fabric for arterials and collectors and a rejuvenating seal for local access), 1910 (10th treatment, or a thin overlay with fabric for arterials and local access and a rejuvenating seal for collectors), and so on through the miles of road built in 1973, which would be receiving their first cyclical treatment. It is recognized that this process is highly idealized and ignores such inevitable real-life factors as severe climate changes, changes in traffic loads, budgetary constraints, modifications to old roads, and the like. Nevertheless, order-of-magnitude cost determinations are possible if this process is followed. More important, it permits the testing of modifications to this process such as extending the 7-year cycle to 10 years or substituting overlays for restoration.

Calculation of Maintenance Shortfalls

The ongoing maintenance shortfall represents the difference between what ought to be spent and what was actually being spent. The backlog maintenance shortfall represents the resultant costs calculated by applying the unit costs of the recommended maintenance for each surveyed road section multiplied by the square yards in each section. Because results were based on a 9 percent sample of all Bay Area segments, an extrapolation to the total 17,000-mile system was required. This resultant total cost was reduced by the required cyclical cost for that year to yield the backlog.

ANALYSIS AND RESULTS

Backlog Maintenance

Eleven Bay Area jurisdictions surveyed the condition of their pavements. Based on 1,500 miles of streets and roads surveyed, 56 percent of the road area was recommended to receive no treatment, 24 percent to receive seals, 15 percent overlays, and 5 percent restoration.

Maintenance backlogs for individual jurisdictions were converted to cost estimates and expanded to represent regional shortfalls. The total Bay Area maintenance backlog was estimated to be between $300 million and $500 million.

To illustrate roughly how this backlog would escalate over the next few years as well as to eventually link the combined maintenance needs to a proposed gasoline-tax increase, backlog costs were estimated through FY 1986-1987. The midrange of the regional estimate was used to simplify the illustration. A 10 percent inflation rate was added to this amount. In addition, because ongoing cyclical and routine shortfalls will become backlog costs, 85 percent (the pavement-related portion) of the projected ongoing maintenance shortfall was also added. Results are shown in Table 1. The findings indicate that Bay Area roads are deteriorating at a more rapid rate than they are being repaired. To focus on the actual magnitude of this deferral of maintenance, it was necessary to examine how jurisdictions are coping with the annual or ongoing maintenance needs.

Ongoing Maintenance

The previous section on study design indicated the

Table 1. Backlog costs estimated through FY 1986-1987.

Fiscal Year	Backlog ($000 000s) Initial	10 Percent Escalation	Pavement-Related Ongoing Shortfall	Total Future
1980-1981	400	40	99	539
1982-1983		54	120	713
1983-1984		71	148	932
1984-1985		93	173	1,198
1985-1986		120	185	1,503
1986-1987		150	205	1,858

Table 2. 1980-1981 Bay Area ongoing maintenance costs.

Maintenance Category	1980-1981 Budgeted Expenditure ($000 000s)	Estimated Essential Need ($000 000s)	Shortfall ($000 000s)
Cyclical	42	118[a]	76
Routine	30	40	10
Nonpavement			
Street lighting	29	33	4
Traffic safety	17	19	2
Street cleaning	10	13	3
Landscaping	11	15	4
Miscellaneous	10	10	0
Other (administrative)	18	20	2
Total	167	268	101

[a]Estimated by applying the standard cyclical maintenance treatment program. All other expenditures and needs are as reported on MTC's inventory and adjusted to average costs per mile for each line item.

method used to determine ongoing maintenance expenditures and needs and resultant shortfalls. In Table 2 the results for the base year, FY 1980-1981, are summarized.

Clearly the pavement-related categories are where the greatest shortfalls are occurring. Eighty-five percent of the $101 million shortfall occurs in these categories; 75 percent is in the important category of cyclical or preventive maintenance. Most of the nonpavement and other categories represent more of the fixed-cost type of expenditures, which are more difficult to cut back or defer in times of severe budget problems.

As illustrated with backlog costs, the future increases in maintenance shortfalls over those attributable to inflation are significant. The same is true for future ongoing maintenance shortfalls. The FY 1980-1981 shortfall has been projected to FY 1986-1987 to illustrate the rapid escalation of the cyclical maintenance costs in millions of dollars. The base-year shortfall of $101 million more than doubles to $241 million. This is occurring primarily because many streets and roads are reaching an age in this decade where more expensive treatments, e.g., overlays and restoration, are necessary.

RECOMMENDATIONS

The analysis and results discussed here have been published (1,2). Because of the magnitude of the deficits estimated in these reports, the technical study phase was extended in order to publicize the problem and work toward possible solutions. Toward this end 2,500 copies of a summary report were distributed, primarily to locally elected officials. A slide show was also prepared and presented to more than 50 groups in early 1982. Maintenance shortfalls were converted into gasoline-tax equivalents to illustrate how much increase would be required. These efforts helped foster a movement in the Bay Area by which 48 of 58 cities in four counties endorsed a 5-cent gasoline-tax increase to be placed on the ballot for voter approval.

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Abridgment

Field Tests of Rapid Repair Methods for Bomb-Damaged Runways

A.H. MEYER, D.W. FOWLER, AND B. FRANK McCULLOUGH

Rapid repair of bomb-damaged runways is of vital concern to the U.S. Air Force. The results of field tests conducted under the direction of the Air Force Engineering and Services Center at Tyndall Air Force Base are presented. These tests were of various rapid repair techniques that use methyl methacrylate (MMA) polymer concrete. This includes both user-formulated and commercially available MMA polymer concrete. Both spalls and craters were repaired. Full-depth polymer-concrete (PC) repairs, at-grade precast units, and precast units with PC caps are reported. The repairs were trafficked with both F-4 (27,000-lb single wheel) and C-141 (144,000-lb dual-tandem wheel) load carts. All of the crater repairs performed satisfactorily as did most of the spall repairs, which demonstrated the feasibility of using PC methods for the rapid repair of bomb-damaged runways.

The rapid repair of bomb-damaged runways is of vital concern to the U.S. Air Force. Airfield pavements must be repaired rapidly after attack so that aircraft can be launched. Current repair procedures, specified in Air Force Regulation (AFR) 93-2 (1), are based on North Atlantic Treaty Organization (NATO) damage criteria, which require that three 750-lb bomb craters be repaired in 4 hr. The repair procedures in AFR 93-2 include backfilling the crater with debris within 1 ft of the surface, removing upheaved concrete, filling the top of the crater with select fill, and then the placing and anchoring metal matting over the surface of the backfilled crater.

New developments in weapons technology have altered the repair criteria. The current threat includes many smaller weapons. As a result, instead of only a few large craters as envisioned in AFR 93-2, the repair procedures must also be able to handle many small or medium-sized craters.

The Air Force Engineering and Services Center is currently engaged in a research and development program to improve the rapid runway repair (RRR) capability. New materials and techniques are being investigated by the Engineering and Services Laboratory at Tyndall Air Force Base. The objective of this study is to develop rapid repair techniques that use methyl methacrylate (MMA) polymer concrete, which has been successfully used for repair of highway structures (2,3).

SPALL REPAIRS

A series of 15 simulated spalls was made in the Tyndall research pavement. The section of pavement used consisted of a clay subgrade covered by 12 in. of portland-cement concrete (PCC) overlaid with 4 in. of asphalt-cement concrete (ACC). This pavement cross section is similar to that which exists at several U.S. Air Force bases.

The series was made up of five sets of three spalls, each set having a small, medium, and large spall. Type A spalls were small, type B were medium, and type C were large (Figure 1). Three sets were repaired by using the user-formulated (UF) system and an in-line mixing gun. From this series of tests, the concept of the in-line mixing gun was verified. This concept allows the chemicals to be mixed at the point of application and minimizes exposure of personnel to the chemicals. The tests also verified that debris can be used for at least part, if not all, of the aggregates required, provided some select material is available for finishing. It should be noted that the debris used did not contain large chunks or a significant amount of asphaltic materials.

CRATER REPAIRS

Three methods of crater repair were tested at the pavement research facility at Tyndall Air Force Base. Three 20 x 20-ft test pits were prepared with clay as the base material.

The first two crater repairs (pits 2 and 3) were made with precast slabs. The precast slabs were prepared from normal PCC. The slabs were nominally 6 x 6 ft square and of two thicknesses, 12 in. and 8 in. All slabs had concave keyways on all sides. All slabs were cast with quick-release lifting eyes and threaded drag eyes. Concrete was placed in the wooden forms, consolidated, and allowed to cure more than 28 days before use.

The 6 x 6-ft size was selected to match the 20 x 20-ft pit to be repaired. Using nine slabs in the repair area resulted in a gap of approximately 6 in. between slabs and edges for the polymer-concrete (PC) bonding material. The 12-in.-thick slabs weighed about 5,200 lb each, and the 8-in.-thick slabs weighed about 3,500 lb. Both of these weights were well within the handling capabilities of the front-end loaders, small cranes, and forklifts usually available at most bases.

Pit 2 was repaired by using the at-grade precast slab method. The base of the pit was a clay layer some 21 in. below the grade surface. A 6-in. layer of crushed limestone was compacted over the clay, and a 3-in. sand leveling course was placed over the crushed limestone. Figure 2 gives a plan view and a cross section of pit 2.

Pit 2 was then tested with the load cart; 150 coverages of the F-4 load cart and 70 coverages of the C-141 load cart were used. A coverage is a function of the number of operations at full load, configuration of the wheels, and wander of the aircraft. One coverage of the F-4 load cart is roughly equivalent to 50 passes of the aircraft and one coverage of the C-141 load cart is roughly equal to 10 passes. The repair showed no visible signs of deterioration. The profile of the repair remained virtually unchanged from the beginning to the end of load-cart testing, and from the profile data the riding quality was excellent, because there was little change in elevation across the slab.

Pit 3 was repaired by using a combination of the precast slab and the cap methods of repair. The base of the pit was a clay layer [California bearing ratio (CBR) = 4] covered with 6 in. of compacted,

Figure 1. Typical spall repair sections.

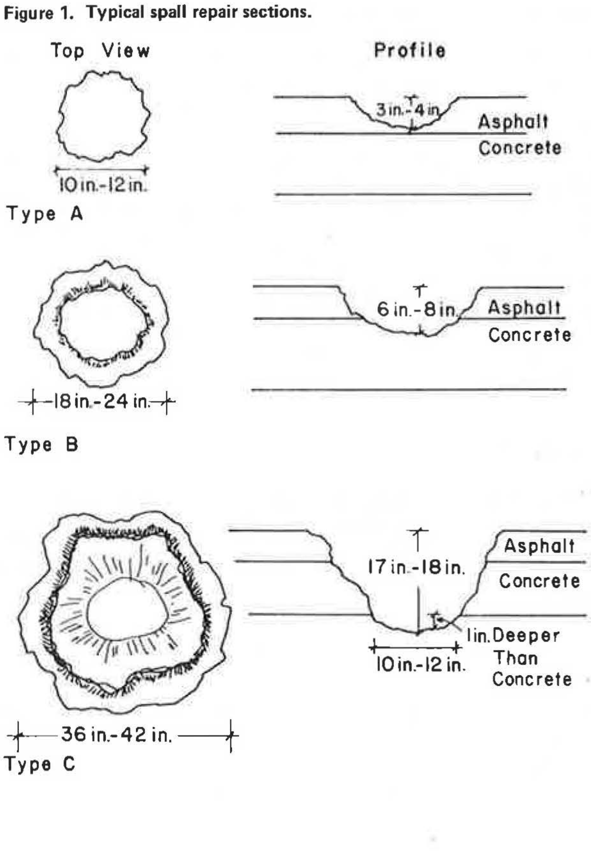
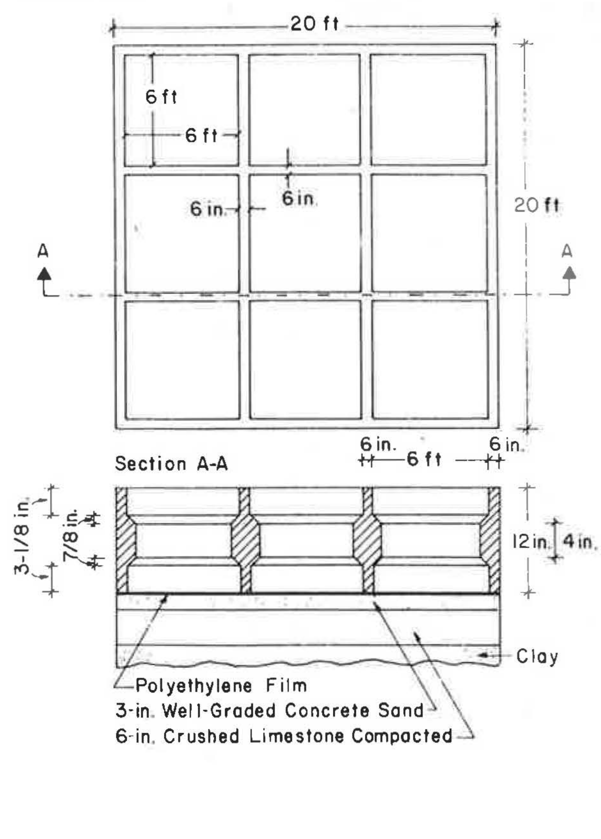


Figure 2. Pit 2.



crushed limestone. The 8-in. precast slabs were then placed on the crushed limestone. Polymer concrete was placed around and over the slabs to bring the repair to grade. The PC cap over the slab was nominally 2 in. thick (Figure 3).

Pit 3 was load-cart tested in the same manner as pit 2, and analysis of the profile data revealed essentially no change in profile from beginning to end of load-cart tests. The profile data also illustrate excellent riding quality in that the maximum change along any profile line was 0.09 ft (1.08 in.) in 20 ft.

Figure 3. Pit 3.

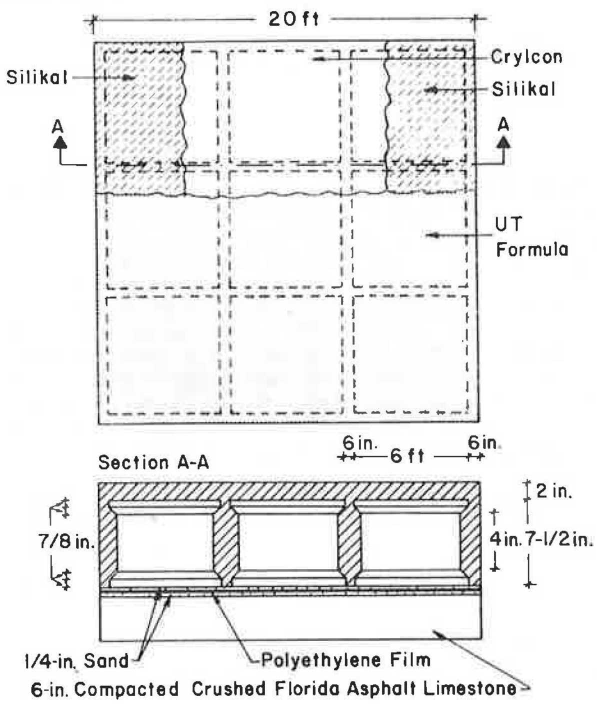
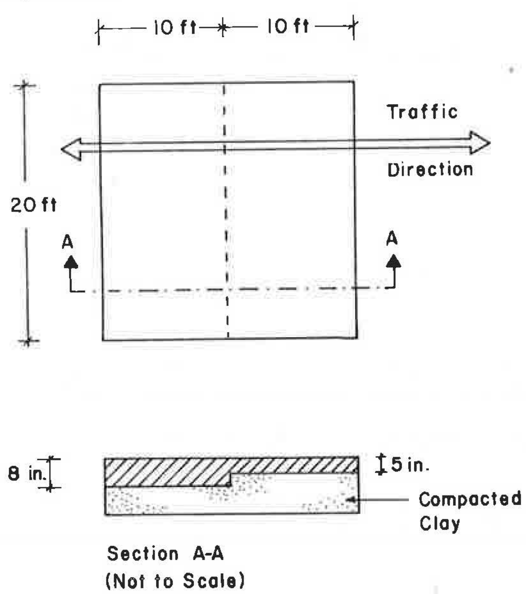


Figure 4. Pit 1.



This method of repair offers some distinct advantages:

1. The bulk of the surface layer (top 12 in.) of the repair is constructed of materials of known and controlled quality (the precast slabs);
2. The precast units do not have to be aligned as required with the at-grade precast repair; and
3. The method significantly reduces the volume of polymer concrete required compared with that for a full-depth PC repair. This lower volume reduces the storage time of chemicals with limited shelf lives.

Pit 1 was repaired by using UF polymer concrete. The base of the pit was clay. The repair consisted of two parts; half of the PC cap was nominally 8 in. thick and half of the repair was nominally 5 in. thick (the actual average thickness was 4.5 in.). A cross section of pit 1 is shown in Figure 4. The clay under the 8-in. section had a CBR of 6, and the clay under the 5-in. section had a CBR of 3.

The UF polymer concrete was placed in pit 1 by using the concrete mobile as in pit 3. After placement, the polymer concrete was screeded and finished. Thermocouple data from both the 5-in. and the 8-in. sections revealed expected values for both peak temperature (approximately 80 to 90°C) and time to peak exotherm (approximately 30 to 35 min).

The repair was load-cart tested with 150 coverages of the F-4 load cart and 70 coverages of the C-141 load cart. Analysis of the profile data reveals no significant changes in elevation from beginning to end of load-cart testing. The average change in elevation is 0.002 ft (0.03 in.); the maximum difference at any location is 0.03 ft (0.4 in.). Analysis of the profile lines reveals a maximum change of 0.08 ft (0.96 in.) in 20 ft. This indicates that the riding quality would be adequate for air traffic.

SUMMARY

In summary the following conclusions can be stated:

1. All repair methods and materials were adequate for the load-cart tests performed, including the 4.5-in.-thick UF PC slab over the CBR 3 clay subgrade;
2. Polymer concrete is structurally adequate for airfield pavement repairs;
3. Regarding time and equipment requirements, the cap method is probably the most efficient if the concrete mobile is used and can be continuously available; and
4. The UF PC system offers a distinct advantage in that the amount of chemicals can be adjusted to satisfy temperature demands in order to keep cure time relatively constant.

ACKNOWLEDGMENT

We would like to acknowledge the combined efforts and support of the Department of Civil Engineering at the University of Texas at Austin and the U.S. Air Force Engineering and Services Center, Tyndall Air Force Base, Florida.

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A Survey on the Use of Rapid-Setting Repair Materials

D.W. FOWLER, GEORGE P. BEER, AND A.H. MEYER

The current state of the art for rapid-setting materials used to repair concrete in Texas and selected other states is reviewed. Texas districts were surveyed for a listing of rapid-setting materials that they have used over the past 10 years. Twenty-seven materials were reported. The districts also provided an evaluation of the materials based on their use in different types of repairs, cost, use in different climatic conditions, durability, bond to concrete, and appearance. Nine states were asked to provide the same information requested of districts; eight responses were received. Districts and states were also asked to provide a ranking of material characteristics and properties.

Rapid-setting repair materials for portland-cement pavements and bridge decks are in great demand. The higher traffic volumes and the advancing age of many pavements and bridges have created serious maintenance problems for state highway forces.

A wide range of repair materials is available (1, pp. 115-160). The materials have been categorized as portland cement, other chemical-setting cements, thermosetting materials, thermoplastics, calcium sulfate, bituminous materials, composites, and additives used to alter mix characteristics (2).

Many different brands of materials are available, and considerable variation in properties is likely for each category from brand to brand. There is considerable variation in cost per unit weight, and the final in-place cost must take into account the ratio of binder to aggregate. Some materials are designed for temporary repairs and others are designed for permanent repairs. Some are to be used in limited ambient temperature ranges, and some cannot be used in wet weather. Some can be used at feathered edges, but most require a chipped or saw-cut boundary.

There is a pressing need for information on which to base selection of rapid-setting materials for different applications. However, there is a serious lack of reliable information from manufacturers and users. Mechanical and durability properties, when available from the manufacturer, are often given without reference to the test methods. The continuing introduction of new products and the modification of old ones makes evaluation and selection more difficult. There has been a paucity of performance information from users.

SCOPE OF STUDY

A research study was begun in September 1981 with the following objectives: identify candidate materials, evaluate selected materials in the laboratory, determine optimum placement methods, test materials and methods in the field, and disseminate results. The first part of this study, a survey of the Texas State Department of Highways and Public Transportation (SDHPT) and transportation departments of selected states to determine their experience with rapid-setting repair materials, is summarized here. No attempt is made in this paper to recommend materials. Future research in this study will provide a basis for methods of evaluation of rapid-setting materials.

USE OF RAPID-SETTING REPAIR MATERIALS IN TEXAS

Many rapid-setting repair materials have been used by SDHPT. Most districts have used one or more of these materials to repair concrete. The Materials and Tests Division (D-9) has tested many of the materials used by the districts. Each district was asked to provide information on the use of rapid-setting materials and D-9 was asked to provide specifications and test results on materials tested. Their response is summarized in this paper.

Survey of Districts

Each SDHPT district in Texas was sent a questionnaire to obtain their experience with rapid-setting repair materials. The questionnaire, which is included in a report by Fowler et al. (3), asked for (a) a ranking of characteristics and mechanical properties of repair materials in order of performance and (b) for each repair material used, the volume per year, relative performance for different types of repairs and weather conditions, appeal to workers, and relative appearance. All but four districts responded to the survey. The materials and their respective ratings by the districts are summarized in this section. The rankings of characteristics and mechanical properties are given later in this paper.

Materials Used

Table 1 summarizes by district the use of rapid-setting materials. All materials reported are shown. The amount, if any, indicated by each district is shown by a symbol representing the range of the amount in pounds per year. The absence of a symbol indicates that no use of the material was reported by a district. The questionnaire asked for all materials used in the past 10 years; 27 materials were reported.

Evaluation of Use and Performance of Materials

Districts were asked to rank the materials on a scale from 1 to 5, in which 5 indicated highest or best, for performance in different types of repairs; cost; mixing, placing, and finishing; durability;

Table 1. Materials used in districts.

District[a]	Alcrete	Bostik 276	Cel-Set	Crylcon	Duracal	B-102 Epoxy	Fast Fix	Ferrolith G	Fondu Calcium Aluminate	Gilco Rapid Patch	Gilcrete	Hub Chem Emulsified Asphalt	Horn 240	Hydraset	Mite 150	Neco-Crete	Plexicrete	Polymer Concrete (UT)[b]	Quik Crete	Set 45	Set 45 (Hot Weather)	Sika Set	Silikal	Speed Crete	Tapecrete	Tiger Crete	Zip-Crete
1														B													
2			A				X							B					X	X				B			
3														B				C		A							A
4					C																						
5		A				A																					A
7																											
8						C												A						C			
9			B			B																B					
10			C																								
11									X								X	X							X		
12																											
13	C			A	C						C																
15																				X			A			X	
16						B																				A	
17																		A									
18				X						A			A					C		B	C		C				
19															B												
20																				B				B			
22																											
23								A										A									
25												A						A		A							
26																B		C		C			C				

Legend: A: 10 to 1000 lb ; B: 1000 to 25,000 lb ; C: More than 25,000 lb ; X: Amount not reported

[a] Questionnaires not received from Districts 6, 14, 21, and 24

[b] UT Polymer Concrete is material developed by Center for Transportation Research

appeal to workers; bond to concrete; and appearance. Table 2 summarizes the evaluation. The numerical rating is an average of the ratings provided by each district and is not weighted for the amount of material. From Table 1 the quantities of each material can be determined. It should be noted that the evaluations for materials that have been used only in small quantities by one or two districts may not be meaningful.

OTHER STATES' EXPERIENCE WITH RAPID-SETTING MATERIALS

Questionnaires similar to those sent to districts were sent to nine states. Most states did not provide an evaluation of materials. Some provided specifications, lists of approved materials, or general comments. A summary of the responses of each of the states is given in the following.

California

California had one of its 11 highway districts fill out the material evaluation questionnaire. They reported using three materials for bridge-deck spalls: Set 45, Horn 240, and Fondu calcium aluminate (C_3A). Table 3 is a summary of California material evaluations.

Florida

Florida is currently in the process of evaluating five rapid-setting materials; final acceptance or rejection of these products has not yet been made.

Georgia

Georgia has used seven rapid-setting repair materials. Limited testing has been performed on these materials; Table 4 presents Georgia's evaluation.

Iowa

Iowa has no special provisions for repairs of pavements and bridges except to use concretes with high cement contents and to use calcium chloride as an accelerator.

Kansas

Kansas has no standard practice for rapid repairs of pavements or bridge decks. It has tested many materials but none has proved entirely satisfactory.

New York

New York currently uses epoxies for repair of pavements and bridge decks. The New York State Department of Transportation specifications cover the details of repairs and epoxies. New York also has made some repairs with polymer concrete, which is covered by a special specification. The Highway Maintenance Division also has a list of approved products for repairs of pavements and structures (3).

Oregon

Oregon does not have a standard practice for rapid repair of pavements and bridge decks. It reports the use of five separate repair materials, of which a summary is given in Table 5.

Pennsylvania

Pennsylvania currently uses a broad range of materials for repair of bridge decks and pavements. It uses epoxy mortars, polymer concretes, polymer-modified mortars, and magnesia phosphate. These products are covered in the Pennsylvania Department of Transportation specification for rapid-setting repair materials. The Pennsylvania Department of Transpor-

Table 2. Summary of material evaluations.

Material	Type of Repair					Cost	Mixing, Placing and Finishing				Durability	Appeal to Workers	Bond to Concrete		Appearance
	Wide Cracks	Bridge Deck Spalls	Pier or Abutment Spalls	Pavement Spalls	Punchouts		Normal Temp	Low Temp	High Temp	Use in Wet Weather			Horizontal Surface	Vertical Surface	
Alcrete	NR[a]	2.0	3.0	3.0	3.0	3.0	4.3	2.0	3.3	1.5	2.7	3.7	3.0	3.3	4.0
Bostik 276	NR	5.0	NR	NR	NR	5.0	2.0	NR	NR	NR	5.0	4.0	5.0	NR	2.0
Cel Set	3.0	NR	5.0	5.0	2.0	3.3	4.7	3.3	4.3	2.0	4.0	4.3	3.7	3.3	4.3
Crylcon	NR	3.0	4.0	5.0	4.5	5.0	4.5	2.5	3.0	1.0	4.5	2.5	5.0	5.0	5.0
Duracal	5.0	4.5	4.0	4.5	3.7	4.3	3.8	2.9	2.7	3.5	4.0	4.2	4.3	4.3	4.5
B-102 Epoxy	3.7	3.3	3.0	4.0	5.0	3.5	3.5	2.7	2.3	1.5	2.5	3.0	3.8	3.0	3.7
Fast Fix	2.0	2.0	NR	2.0	NR	3.0	4.0	3.0	2.0	1.0	3.0	3.0	3.0	NR	NR
Ferrolith-G	NR	3.0	NR	NR	NR	3.0	4.0	2.0	4.0	2.0	4.0	3.0	4.0	NR	3.0
Fondu C_3A	NR	NR	NR	4.0	NR	2.0	5.0	4.0	4.0	5.0	4.0	4.0	4.0	4.0	4.0
Gilco Rapid Patch	NR	NR	NR	NR	4.0	2.0	5.0	NR	NR	5.0	5.0	5.0	3.0	NR	4.0
Gilcrete	NR	NR	NR	2.0	3.0	3.3	3.7	1.7	2.0	1.0	2.3	2.7	2.0	2.7	3.0
Hubchem Emulsified Asphalt	NR	NR	NR	4.0	4.0	3.0	4.0	4.0	4.0	5.0	5.0	5.0	3.0	2.0	3.0
Horn 240	NR	NR	NR	NR	4.0	4.0	4.0	NR	NR	1.0	3.0	4.0	3.0	NR	NR
Hydraset	NR	3.0	NR	3.0	4.5	2.3	4.0	3.7	3.3	2.7	4.7	4.0	4.7	3.7	5.0
Mite 150	NR	NR	NR	5.0	5.0	3.0	2.0	2.0	2.0	NR	5.0	5.0	NR	NR	5.0
Neco-Crete	NR	3.1	3.1	3.1	3.3	NR	NR	NR	NR	NR	NR	NR	NR	NR	NR
Plexicrete	4.0	NR	NR	4.0	NR	5.0	5.0	4.0	4.0	4.0	5.0	5.0	4.0	4.0	5.0
Polymer Concrete (U.T.)	3.2	3.7	3.3	3.3	4.3	3.4	4.6	2.5	3.8	1.0	4.3	2.7	3.9	3.8	4.1
Quik Crete	2.0	2.0	NR	2.0	NR	3.0	4.0	3.0	2.0	1.0	3.0	NR	3.0	NR	NR
Set 45	4.5	4.0	3.0	4.2	3.9	3.7	4.2	4.3	2.8	4.0	4.5	4.8	4.3	3.3	4.3
Set 45 (Hot Weather)	4.0	4.0	NR	NR	4.0	2.0	5.0	5.0	4.0	5.0	4.0	5.0	3.0	NR	5.0
Sikaset	3.0	NR	NR	NR	4.0	5.0	5.0	3.0	2.0	1.0	4.0	5.0	2.0	1.0	4.0
Silikal	3.7	4.2	3.3	4.2	4.4	5.0	4.0	4.0	2.5	1.0	4.5	2.0	4.5	4.0	4.5
Speed Crete	1.0	2.0	2.0	1.5	2.5	2.0	5.0	5.0	5.0	1.7	1.7	4.3	2.0	2.0	3.7
Tape Crete	NR	NR	NR	5.0	NR	3.0	5.0	4.0	4.0	5.0	5.0	5.0	5.0	5.0	5.0
Tiger Crete	5.0	3.0	1.0	1.0	NR	4.5	5.0	4.0	4.0	5.0	3.0	4.0	1.0	5.0	4.0
Zip-Crete	NR	2.0	4.0	NR	NR	2.5	4.0	1.0	4.0	NR	3.5	5.0	4.5	3.0	5.0

Evaluations are based on a subjective scale of 1 to 5 with 5 representing the best performance or highest cost

[a] NR indicates no response

Table 3. Evaluation of rapid-setting materials by the state of California.

Material	Usage lb /yr	Bridge Deck Spalls	Cost	Mixing, Placing and Finishing				Durability	Appeal to Workers	Bond to Concrete		Appearance
				Normal Temp	Low Temp	High Temp	Use in Wet Weather			Horizontal Surface	Vertical Surface	
Fondu C_3A	15,000	3.0	3.0	4.0	5.0	3.0	NR	3.0	3.0	3.0	NR	2.0
Horn 240[a]	10,000	5.0	4.0	4.0	5.0	2.0	NR	4.0	NR	5.0	NR	4.0
Set 45	25,000	4.0	3.0	4.0	5.0	3.0	NR	4.0	4.0	4.0	NR	5.0

[a] Same as Darex 240

tation also has a list of approved commercial rapid-setting materials (3).

DESIRED CHARACTERISTICS AND MECHANICAL PROPERTIES

Districts and other states were asked to rank characteristics and mechanical properties of rapid-setting repair materials. Eight characteristics and eight properties were listed, and other items could be added.

Response of Districts

The ranking of characteristics and properties by districts is as follows:

Characteristics:

1. Setting time;
2. Performance (durability);
3. Working time;
4. Ease of mixing, placing, and finishing;
5. Use over wide temperature range;
6. Use in wet weather;
7. Cost;
8. Similarity to color of adjacent concrete; and
9. Availability.

Properties:

1. Bond strength to concrete,
2. Flexural strength,

Table 4. Evaluation of rapid-setting materials by the state of Georgia.

							Cost	Mixing, Placing and Finishing						Bond to Surface		
Material	Usage lb/yr	Wide Cracks	Bridge Deck Spalls	Pier or Abutment Spalls	Pavement Spalls	Punchouts	Joint Repairs	Normal Temp	Low Temp	High Temp	Use in Wet Weather	Durability	Appeal to Workers	Horizontal Surface	Vertical Surface	Appearance
Duracal	10,000	NR	3.0	NR	2.0	NR	3.0	3.0	3.0	3.0	3.0	2.0	4.0	2.0	1.0	3.0
Epoxy	NR	2.0	3.0	NR	2.0	3.0	5.0	2.0	2.0	1.0	2.0	3.0	1.0	4.0	4.0	NR
Horn 240	1,000	NR	NR	NR	4.0	NR	4.0	3.0	4.0	1.0	1.0	4.0	3.0	3.0	3.0	3.0
Polymer Concrete	10,000	NR	3.0	NR	4.0	NR	5.0	3.0	3.0	2.0	1.0	5.0	2.0	4.0	4.0	4.0
Roadpatch	1,500	NR	NR	NR	4.0	NR	3.0	3.0	2.0	1.0	3.0	3.0	3.0	3.0	3.0	3.0
Set 45	100,000	NR	NR	NR	3.0	NR	4.0	3.0	5.0	1.0	1.0	2.0	2.0	2.0	2.0	3.0
Speed Crete	NR	NR	NR	NR	1.0	NR	2.0	3.0	2.0	2.0	3.0	1.0	3.0	1.0	1.0	3.0

Table 5. Evaluation of rapid-setting materials by the state of Oregon.

	Type of Repairs						Mixing, Placing and Finishing						Bond to Concrete			Cost
Material	Wide Cracks	Bridge Deck Spalls	Pier or Abutment Spalls	Pavement Spalls	Punchouts	Joint Repairs	Normal Temp	Low Temp	High Temp	Use in Wet Weather	Durability	Appeal to Workers	Horizontal Surface	Vertical Surface	Appearance	
Concreasive 2020 Polymer	NR	5.0	NR	NR	NR	5.0	5.0	3.0	2.0	1.0	5.0	5.0	5.0	5.0	5.0	5.0
Crylcon	NR	5.0	NR	NR	NR	5.0	5.0	3.0	2.0	1.0	5.0	5.0	5.0	5.0	5.0	5.0
Niklepoxy Product #4	NR	4.0	NR	1.0	NR	NR	4.0	2.0	2.0	1.0	4.0	3.0	4.0	4.0	3.0	5.0
Type III Portland Cement Concrete (w/c = 0.30)	NR	4.0	NR	3.0	NR	NR	3.0	3.0	3.0	2.0	5.0	3.0	4.0	2.0	5.0	1.0
Set 45	NR	2.0	NR	2.0	NR	NR	5.0	2.0	2.0	3.0	2.0	3.0	3.0	2.0	3.0	3.0

3. Shrinkage,
4. Compressive strength,
5. Ductility,
6. Wear resistance,
7. Coefficient of thermal expansion, and
8. Stiffness (modulus of elasticity).

Setting time, performance (durability), and working time were rated the top three characteristics. Bond strength to concrete, flexural strength, and shrinkage were rated the top three mechanical properties.

Response of Other States

The ranking of characteristics and properties by other states is as follows:

Characteristics:

1. Performance (durability);
2. Ease of mixing, placing, and finishing;
3. Cost;
4. Setting time;
5. Working time;
6. Use over wide temperature range; and
7. Use in wet weather and similarity to color of adjacent concrete (tie).

Properties:

1. Bond strength to concrete,
2. Compressive strength,
3. Shrinkage,
4. Flexural strength and coefficient of thermal expansion (tie),
5. Wear resistance,
6. Ductility, and
7. Stiffness (modulus of elasticity).

Other states ranked performance (durability), ease of mixing and placing, and cost as the top three characteristics. The top three mechanical properties were bond strength to concrete, compressive strength, and shrinkage. The states ranked the same four mechanical properties at the top of the list as the districts did, although the order was slightly different.

CONCLUSIONS

There is an urgent need for dependable rapid-setting materials for the repair of concrete pavements and bridge decks. Many types and brands are currently available, but the selection of an appropriate material is complicated by the lack of reliable data from manufacturers and users. There is no standard evaluation procedure for these materials.

All of the SDHPT districts in Texas were surveyed to determine their experience and evaluation of rapid-setting repair materials. Quantities of each repair material used per year were obtained. Evaluations of each material were made on the basis of types of repair, cost, climatic conditions, durability, bond to concrete, and appearance. Considerable variation was noted for the 27 materials reported.

Other selected states were surveyed to determine their current experience. Six of the eight states responding listed specific materials that were currently being used. Three states provided an evaluation similar to that provided by the SDHPT districts.

The SDHPT districts provided a priority order for characteristics and mechanical properties. Setting time, performance (durability), and working time were ranked as the top three characteristics, whereas bond strength to concrete, flexural strength, and shrinkage were rated the top three mechanical properties.

The survey of the other states indicated performance (durability); ease of mixing, placing, and finishing; and cost as the top three characteristics. Bond strength to concrete, compressive strength, and shrinkage were given as the top mechanical properties.

RECOMMENDATIONS

It is recommended that further research be conducted to establish appropriate evaluation procedures for rapid-setting repair materials, evaluate the most common materials, and determine the field test performance of different types of repairs.

ACKNOWLEDGMENT

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Design of Polymer-Concrete Runway Repairs

B. FRANK McCULLOUGH, A.H. MEYER, AND D.W. FOWLER

Portland-cement-concrete airfield pavements with polymer-concrete (PC) repairs were analytically modeled to develop design criteria for determining the required repair thickness. A previously developed computer program for analyzing discontinuous orthotropic plates and pavement slabs was used to analyze the pavement. Two representative aircraft, the F-4 and the C-141, were used. Different repair sizes, support values, and runway thicknesses were tested. A sensitivity analysis was performed to determine which variables have the greatest effect on the stresses. For the purpose of developing design charts, the critical positions of the wheel loads for the different size repairs were found. The magnitude of the existing runway support (K-value) outside the repair section was found to have little effect on the stresses in the PC repair, although the existing runway thickness did. Because of the emergency nature of the repairs, the repair support values and thicknesses may be significantly different from those for the existing pavement. Consequently, these values have a significant impact on the repair results. Design charts were prepared that give the flexural stress as a function of repair thickness for three repair sizes, two repair support values, and two runway thicknesses. The allowable stress level for the polymer concrete has been reduced for the number of loading repetitions.

The U.S. Air Force, through the University of Texas, has recently studied the rapid repair of runways by using polymer-concrete (PC) materials (1). In some cases a section of a runway can be partitioned off and rapid repairs made so that the field can continue to serve its functional purposes. The results of this study are believed to be applicable to all runway types and thus the information is presented to add to the status of knowledge. In this paper the primary concern is the design aspects; for the material properties, see the papers by Meyer et al. and Fowler et al. in this Record.

Polymer concrete has been shown to be an effective material for rapid repair of bridge decks, pavements, and runways. PC materials consist of aggregate with a polymer binder instead of portland cement. Polymer concrete made with methyl methacrylate develops a strength of 6,000 psi in 1 hr or less (almost the ultimate value), is more ductile than portland-cement concrete (PCC), and bonds well to normal concrete.

In this paper the mechanistic modeling of concrete pavements with PC repairs is described to develop criteria for determining repair thicknesses. The behavior of the repairs was predicted for a wide range of support and loading conditions expected at North Atlantic Treaty Organization (NATO) bases in Europe. Then field tests were made at Tyndall Air Force Base in Florida to experimentally verify the boundary conditions on the charts. Design charts are presented for quickly determining the thickness of repairs required for the anticipated conditions.

It is essential in any analytical approach that techniques be used to properly model the load, geometry, and material properties to reliably predict the stresses in the pavement. In this study the SLAB 49 program developed by Matlock and Hudson for problems such as this (2) was used for developing the design charts.

Figure 1 presents a plan view and a longitudinal cross section of a runway pavement slab containing a repair section. A typical slab, 24 x 36 ft, was selected for study. In previous studies this size has been found to be adequate for studying the stress and deflection distribution for aircraft loadings. In this study the F-4 and C-141 were selected as the representative aircraft.

For the PC patches, three sizes were selected: 5 x 5 ft, 17 x 17 ft, and 24 x 36 ft. A preliminary study indicated that the shape would have only a minor influence on the stresses (less than 1 percent) except for extreme conditions. The thickness of the PC repair ranged from 5 to 10 in. in 0.5-in. increments. The support values selected were 50, 100, 200, and 300 pci; 50 pci represents a backfill with a minimum compaction. These values were guidelines provided by the U.S. Air Force based on previous testing.

For the existing PCC pavement, thicknesses of 8, 12, and 16 in. were considered. The support values selected were 100, 300, and 500 pci.

The following parameters were held constant in the study based on extensive laboratory tests conducted as a part of this study and reported elsewhere (1, paper by Meyer et al. in this Record):

$$E_{PC} = 2 \times 10^6 \text{ psi} \quad (1)$$

$$\nu_{PC} = 0.30 \quad (2)$$

$$E_{PCC} = 4 \times 10^6 \text{ psi} \quad (3)$$

$$\nu_{PCC} = 0.15 \quad (4)$$

where E_{PC} and E_{PCC} are the moduli of polymer concrete and portland-cement concrete, respectively, and ν_{PC} and ν_{PCC} are the Poisson ratios for the same materials.

Figure 1. Runway repair plan and section.

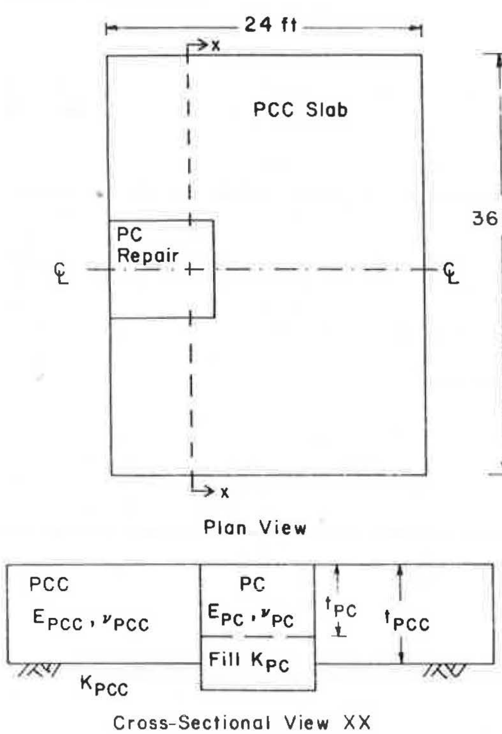


Figure 2 shows a footprint of the gear configurations of both aircraft with the weights and the tire pressures. Previous investigations have indicated that the stress will vary depending on the placement of the wheel relative to the edge, corner, and interior (3). Figure 3 shows the slab loading conditions expected in the field for the various aircraft types and repair conditions. Basically the conditions range from an interior location, represented by position 1, to a corner-edge condition, repre-

Figure 2. Aircraft load and gear configuration.

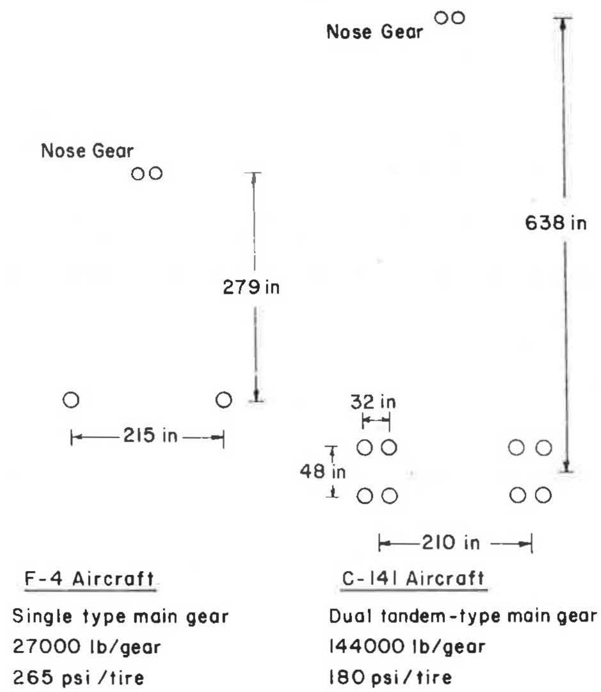
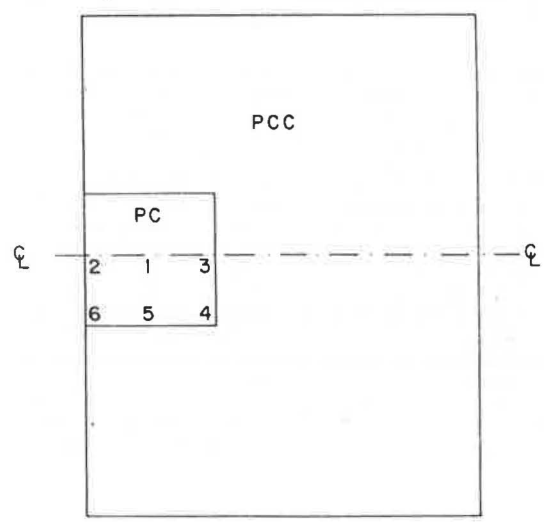


Figure 3. Horizontal load position for F-4 and C-141 aircraft.



Position 1 - Center of Patch
Position 2 - Outside Edge
Position 3 - Horizontal PC-PCC Interface
Position 4 - Corner PC-PCC Interface
Position 5 - Vertical PC-PCC Interface
Position 6 - Outside Corner

sented by position 6. The maximum flexural stress beneath the gear must be determined for all the factors considered to ascertain the critical condition. For the C-141 aircraft, the load placements for positions 1, 2, and 3 were varied slightly, as shown in Figure 4, because the tire-gear configuration was tandem. For one placement the front wheels were placed directly on the repair-section centerline, and for the other position the center of the gear was placed at the center of the repair, as shown in Figure 4.

SENSITIVITY STUDY OF VARIABLES

The nature of this study presented a wide range of conditions for investigation, and as the study progressed, it was obvious that some of the variables had no significant influence on the results. These factors are discussed briefly to provide background for the reasons that some variables are not considered in the analysis.

The presence or absence of the nose gear did not significantly influence the maximum stress conditions; this may be attributed to the wheel-base distances of 279 and 636 in. for the F-4 and the C-141, respectively. For most of the calculations, the nose gear was not considered. In addition, the maximum stress was not influenced by the adjacent gear because the wheel-tread distances were 210 and 215 in., respectively. The stresses, given in the next section, result from studies on one gear and thus would not be different if the entire configuration had been considered.

Considering the horizontal position of the gear, it was found (Figure 3) that positions 1, 2, and 3 were critical, whereas the other positions were less critical, because they received support from the stiffness of the surrounding PCC. Therefore the information reported in later sections is relative to positions 1, 2, and 3.

In the case of the 5 x 5-ft PC repair, positions 1, 2, and 3 were close together; subsequently only position 2 (edge) was considered, because it was the most critical. In the case of the 17 x 17-ft PC repair and the 24 x 36-ft PC repair, the study showed that position 3 had a low value of flexural stress compared with positions 1 and 2 for the F-4 aircraft load. Thus, this position was considered to have relatively little influence on the life of the repair. The loading with the C-141 showed that position 3 gave a higher stress than position 1. This is attributed to the wide load distribution of the C-141 main gear, which, unlike the F-4 main gear, is not influenced by the surrounding PCC. Although flexural stresses were found for position 1 that were not significantly lower than those obtained for position 3, it was decided that the design for position 3 would satisfy the condition of position 1.

In summary, only position 2 (representing the edge condition) and position 1 (representing the interior condition) were considered significant for the F-4 aircraft. Only position 2 (representing edge condition) and position 3 (representing the interior condition) were considered significant for the C-141.

For the C-141 aircraft, which has a dual-tandem gear, the two positions shown in Figure 4 were analyzed and the results indicated that loading the gear tires on the center of the repair gave the most critical stresses. This position was used for further analysis. Another problem investigated for the C-141 dual-tandem gear was the location of the maximum flexural stresses, which were found to occur directly under the tire.

The results given in Figures 5 and 6 indicate that varying the existing runway support has no influence on the repair section flexural stresses. The runway support was then fixed at 300 pci, which is slightly above the average (250 pci) existing at many bases. Although the 300 pci does not influence the results, it permits the user to consider repair support K_{PC}'s up to 300 pci, because of the primary assumption, i.e., $K_{repair} \leq K_{runway}$.

For a 17 x 17-ft repair, varying the existing runway thickness did not influence the flexural stresses in the repair section significantly (Figure

Figure 4. C-141 aircraft vertical gear position.

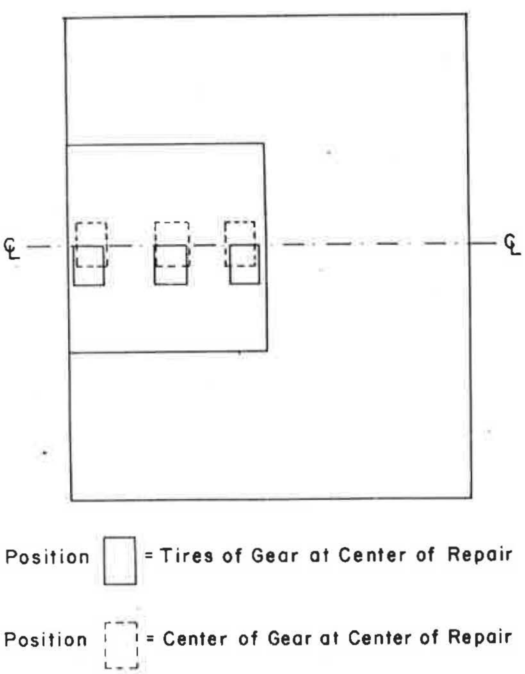
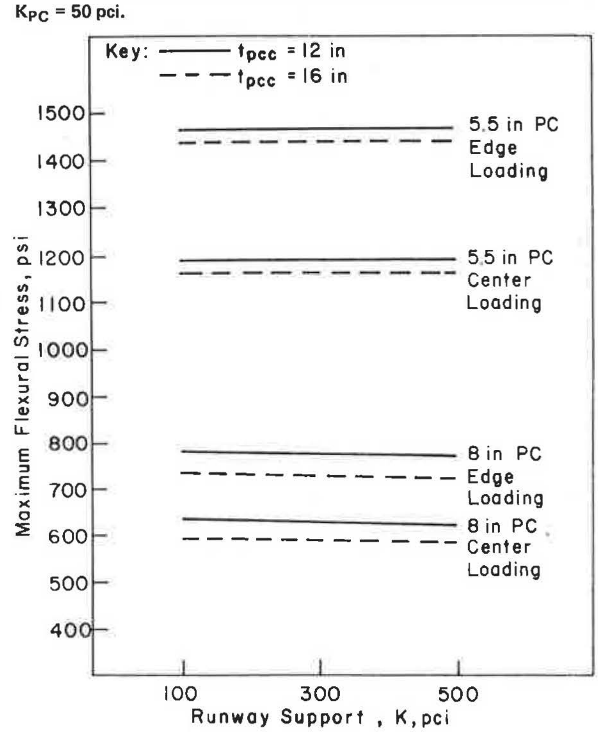


Figure 5. Stress versus runway support for F-4 loading, 5 x 4.5-ft repair, and K_{PC} = 50 pci.



7). The runway thickness was then fixed at 12 in., because this thickness represents a typical thickness encountered in the field and does not influence the results. For a 5 x 5-ft repair, however, the runway thickness has a definite influence. The values of 12 and 16 in. were used to represent thin and thick pavements, respectively, whereas 8 in. was rejected because high stresses were obtained by loading an 8-in. PCC slab, which indicated that the runway was underdesigned. The repair support has an influence on the results, and the values of 50 pci and 300 pci were selected as representing the extremes (poor and strong support).

The maximum stresses resulting from all conditions where K_{PC} = 300 pci and position 2 (edge) was loaded were plotted, and the boundaries of the stress envelopes for the F-4 and the C-141 are shown in Figure 8. Because the range of stress was small, only the upper boundary for the F-4 and the C-141 was used to represent the edge loading; K_{PC} = 300 pci for all conditions, which reduced the scope of analysis.

Figure 6. Maximum flexural stress versus runway support for C-141 loading and 5 x 4.5-ft repair.

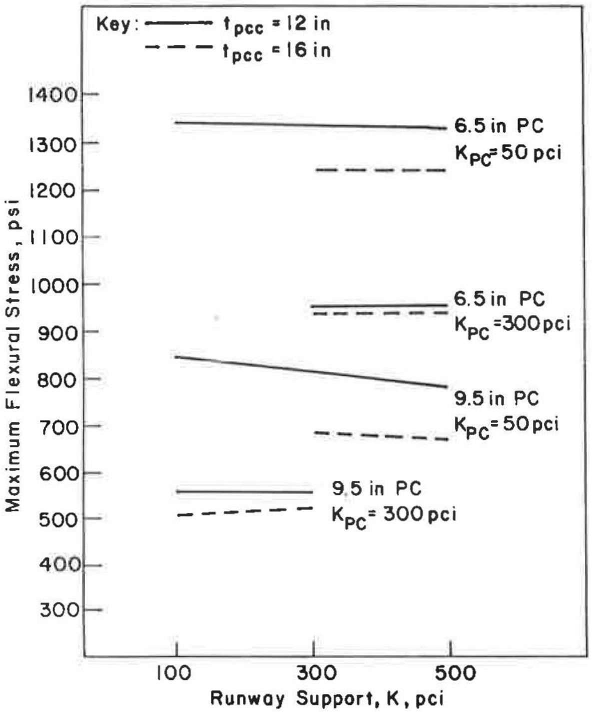
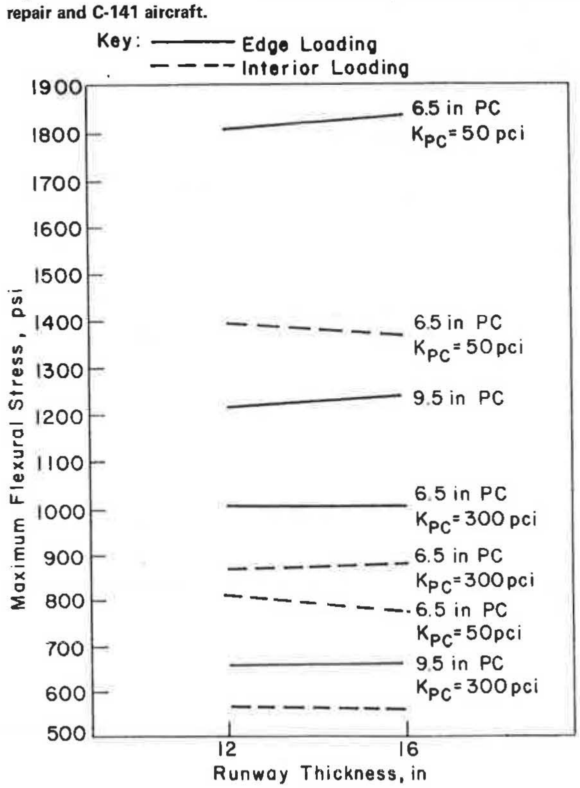


Figure 7. Maximum flexural stress versus runway thickness for 17 x 17-ft repair and C-141 aircraft.



DESIGN CHARTS

After the sensitivity study had been completed, design charts were prepared containing only the significant variables as defined by the sensitivity study. Figures 9 through 12 are design charts that present the maximum flexural stress for the repair section in terms of the PC repair depth for a range of conditions. On the charts, qualitative variables are used, whereas the previous information has been developed in terms of quantitative factors. The qualitative factors are as follows:

Small repair size: 25 to 299 ft^2,
Large repair size: 300 to 999 ft^2,
Major replacement: 1,000 ft^2 or greater,
Strong repair support: 300 pci or more,
Poor repair support: 50 pci,
Thick existing runway: 16 in. thick, and
Thin existing runway: 12 in. thick.

Figures 9 and 10 present the maximum flexural stress in polymer concrete versus the PC repaired-

Figure 8. Flexural stress versus PC depth for edge loading and K_{PC} = 300 pci.

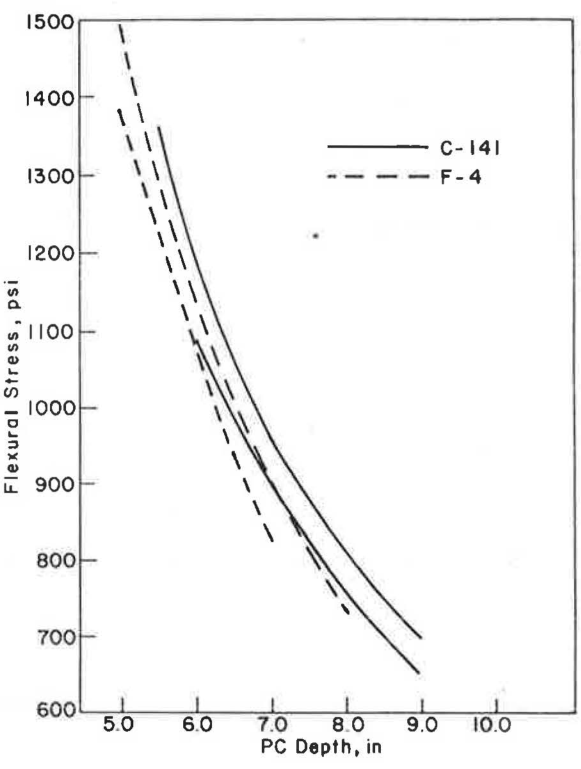


Figure 9. PC thickness design chart for F-4 aircraft and edge loading.

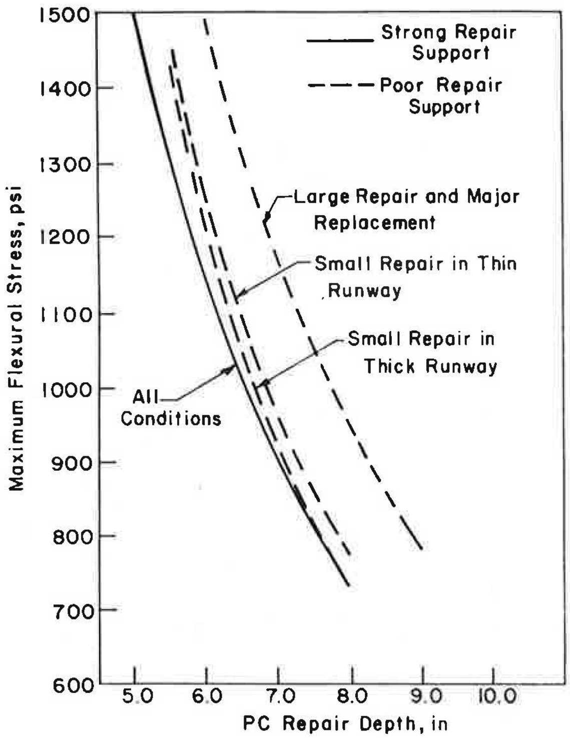


Figure 10. PC thickness design chart for F-4 aircraft and interior loading.

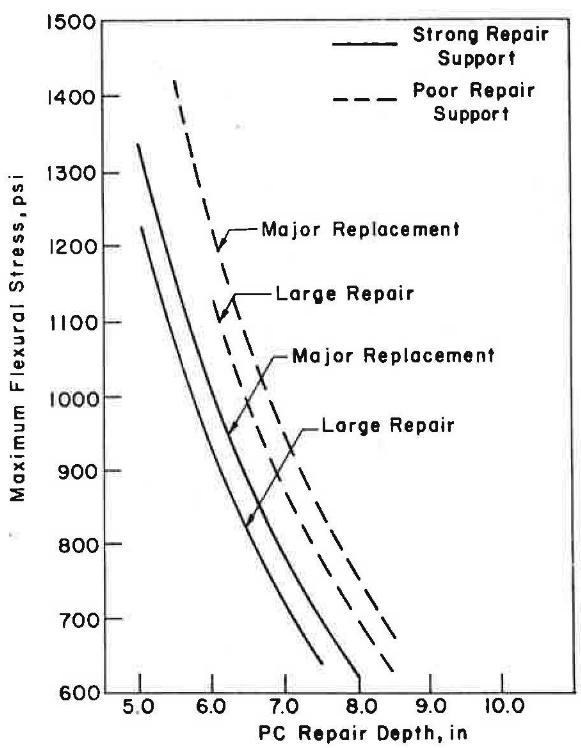


Figure 11. PC thickness design chart for C-141 aircraft and edge loading.

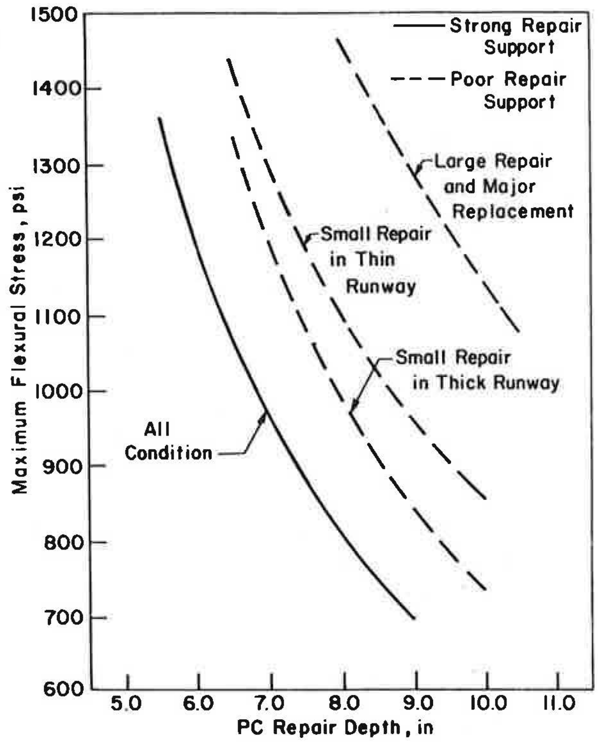
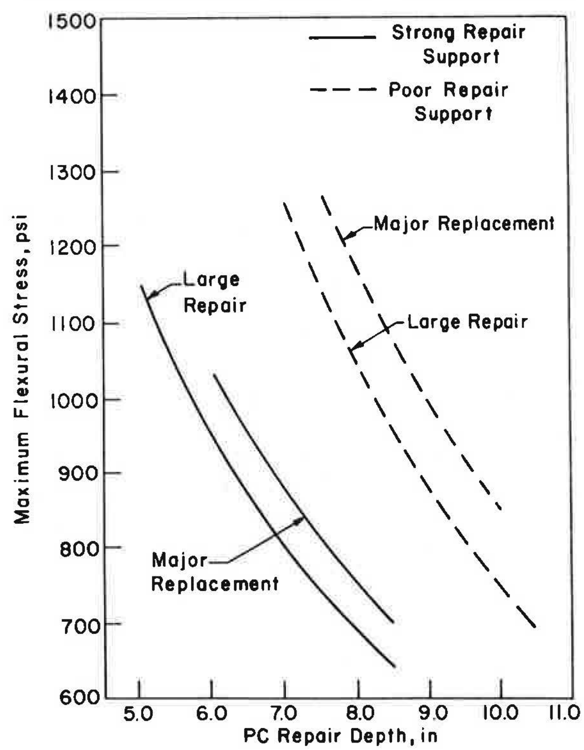


Figure 12. PC thickness design chart for C-141 aircraft and interior loading.



section thickness for the F-4 aircraft for edge and interior loading, respectively. Figures 11 and 12 give the same information for the C-141 aircraft for the edge and interior loading, respectively.

For simple use of the charts, only the edge loading should be considered for the small repairs, whereas for larger repairs and major replacement both edge and interior loading conditions should be considered. In using the chart, the value of the repair support may be interpolated. For the repair size, a size is determined and then the range that encompasses the appropriate figure is found. For example, if the repair area is 225 ft², the user would select the larger repair size for determining the PC thickness. If the repair section encompasses the edge, the edge condition should be selected. The edge condition is selected for longitudinal joints without load transfer or for edge conditions that have been selected because of least-damaged regions. If the repair section is surrounded by the existing pavement, an interior loading condition is used.

The allowable stress level to be used in the equation must be derived as a function of the number of repetitions. Therefore, the fatigue concept is applicable here. The following equation represents a typical fatigue equation that is used for PCC:

$$N = A\,(f/\sigma)^{B} \quad (5)$$

where

- σ = the stress in the concrete due to the appropriate aircraft loading and other conditions,
- f = the flexural strength of the polymer concrete,
- N = the allowable number of repetitions for the strength and stress conditions, and
- A, B = coefficients for testing specific materials.

This fatigue-equation format has been used for the design of PCC (strain in lieu of stress) and asphalt concrete for a substantial period of time and is felt to be applicable here. The coefficients A and B have not yet been developed for polymer concrete. Because all static tests on polymer concrete have indicated that this concrete is vastly superior to the normal PCC, it is believed that the use of coefficients developed for PCC will be conservative. Therefore, Equation 5 is defined as follows for polymer concrete, based on previous studies:

$$N = 23{,}400\,(f/\sigma)^{3.21} \quad (6)$$

DESIGN PROCEDURE

The following is a sequential procedure that may be used to determine the thickness of PC repair:

1. The thickness of the existing concrete pavement and relative applications of each of the two design aircraft (C-141 and F-4) are determined. It is decided what compaction condition is to be used in the field or repair section (e.g., poor or good) and for what length of time design applications are to be applied.
2. The user surveys the repair section and makes qualitative decisions as to small, large, or major repair replacement, and it is decided whether the repair is a free edge (i.e., whether the repair is surrounded by existing pavement) or zero load transfer at a longitudinal joint.
3. [continued] puted for each aircraft type by using the following equation:

$$\sigma = f\,(23{,}440/N)^{0.31} \quad (7)$$

4. The user then determines the PC repair by entering the appropriate value from Figures 9 through 12 to represent the aircraft type and loading condition. The allowable stress from step 3 is entered at the vertical axis projected horizontally to the appropriate repair section and support condition. At the intersection, the line is projected vertically and the thickness is read on the horizontal scale.

CONCLUSIONS

A design procedure for PC repairs for runways has been presented here that was developed for bomb damage repairs, but the concepts are applicable to any emergency repair. The repair was modeled with a computer program with a large number of variables based on aircraft type, size and location of repair, location of wheel on repair, support stiffness, and repair thickness. Based on the computer analysis, design charts are presented to permit the required PC thickness to be determined. The charts were results from the tests in the field and found to model the field conditions (1).

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The following acronyms are used without definition in Record papers:

AASHO	**American Association of State Highway Officials**
AASHTO	**American Association of State Highway and Transportation Officials (formerly AASHO)**
ASCE	**American Society of Civil Engineers**
ASME	**American Society of Mechanical Engineers**
ASTM	**American Society for Testing and Materials**
FAA	**Federal Aviation Administration**
FHWA	**Federal Highway Administration**
FRA	**Federal Railroad Administration**
IEEE	**Institute of Electrical and Electronics Engineers**
ITE	**Institute of Transportation Engineers**
NCHRP	**National Cooperative Highway Research Program**
NCTRP	**National Cooperative Transit Research and Development Program**
NHTSA	**National Highway Traffic Safety Administration**
SAE	**Society of Automotive Engineers**
TRB	**Transportation Research Board**
UMTA	**Urban Mass Transportation Administration**