

# Overlay Design Method for Flexible Pavements in Arizona

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A rational overlay design method for flexible pavements in Arizona, which includes roughness, fatigue, and plastic deformation models, has been developed. The roughness model is based on an analysis of the change in roughness because of overlay and the rate of change in roughness after overlay for typical Arizona roads. The fatigue model uses a shift factor to adjust existing fatigue criteria for the rate of crack development of pavements in the state. The plastic deformation model ensures that the overlay is thick enough so that expected traffic loads will not induce significant plastic deformations in underlying layers. The three design models are incorporated into a microcomputer program for computing the optimum overlay thickness, estimating the remaining life of an existing pavement, evaluating the life of a user-specified overlay, and performing economic analyses. Twenty in-service pavement sites were selected from Arizona highways covering various geographical and environmental regions, soil types, pavement conditions, and traffic volumes. Nondestructive tests were performed on these sites using the falling weight deflectometer at several stress levels, as well as Dynaflect tests. The layer elastic moduli were back calculated and adjusted for temperature. Asphalt concrete cores and undisturbed subgrade samples were collected and tested in the lab for modulus and other properties. The overlay design procedure has been verified with typical pavement sections in the state. Although the procedure in its current form applies to conditions in Arizona primarily, the design concepts can be used with conditions in other states.

For a variety of reasons, the number of new highway construction projects is steadily decreasing. As a consequence, a higher percentage of highway agency resources is being devoted to upgrading and maintaining existing highways. Thus, overlay design has moved into the forefront of pavement engineering.

The primary goal in overlay design is to provide a pavement that can withstand the applied traffic loads, throughout the design life, without failure, such as excessive cracking, rutting, or loss in serviceability. Fundamental engineering decisions include assessing which sections of highway require overlaying and how much overlay is needed. Mechanistically based design methods should be developed to close the gap between theory and practice and to upgrade the performance of the existing highway system.

The most commonly used overlay design approaches are (a) engineering judgment, (b) standard thickness, (c) empirical, and (d) mechanistic or mechanistic-empirical (1). Mechanistic approaches are preferred over others since they characterize the response of the pavement to a load based on

basic parameters such as strains or stresses. On the other hand, failure is normally defined by specific mechanisms such as fatigue cracking or rutting, or both. Currently, no completely mechanistically based overlay design method exists. All mechanistically based methods depend in part on empirical relations between pavement parameters and the number of load applications the pavement can support before failure.

A number of overlay design methods for flexible pavements are available (2-6). Although these methods cover a wide variety of design concepts and applications, they may not be completely applicable to the conditions in Arizona. The main purpose of this study was to develop a mechanistic-empirical method of overlay design for flexible pavements that could be used by the highway personnel of the Arizona Department of Transportation (ADOT) with a high degree of confidence. In this study, the available overlay design methods were reviewed including a method that was developed by ADOT (7). It was decided that sufficient new data were available and could be collected to develop a new procedure tailored to the conditions in Arizona.

The overlay design method presented in this study was developed by the Center for Advanced Research in Transportation (CART) at Arizona State University. The design method was named CODA, which stands for CART Overlay Design for Arizona.

## DATA COLLECTION

Currently, the existing highway network in Arizona is approximately 6,000 miles, including Interstate highways, U.S. routes, and state roads. The majority of this network is paved with asphalt. More than 50 percent of these roads are in excellent to good condition, whereas about 25 percent are in poor condition. In this study the data collection process can be divided into three groups: (a) data base search; (b) deflection measurement; and (c) sample collection and laboratory testing, as discussed in the following paragraphs.

### Data Base Search

The Arizona Department of Transportation has an efficient, computerized pavement management system (PMS) file that includes various pavement data dating back to 1972. For each milepost of the highway network in the state, and for each year, data such as Maysmeter roughness, cracking, and friction number are available. Rutting data are available for

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Interstate highways starting in 1986. Traffic data include average daily traffic, 18-kip equivalent single axle load (ESAL) in the current year, and the average annual traffic growth for the last 10 years. Other data are also available, such as the number of lanes, lane width, and shoulder width.

In addition to the PMS file, a pavement construction history file is available. The construction history file includes layer thicknesses, material types, and year of construction of each layer of the pavement network. These data are stored by construction project.

### Deflection Measurement

Although a deflection data file is available in the ADOT computer data base, the available data were incomplete and taken at various dates and for various conditions. Therefore, falling weight deflectometer (FWD) tests were performed at 20 selected sites distributed throughout the state with good historical records. The criteria for the site selection include

1. availability of traffic data,
2. availability of material properties,
3. overlay history of the site,
4. current pavement condition,
5. geographical location, and
6. materials in the pavement structure.

Because one objective of the evaluation of the pavement sites was to permit the evaluation or development of performance models for overlaid pavements, factor 3 was very important. The most desirable pavement site would be one that had been overlaid one time with an overlay near the end of its service life. Sites that meet these criteria would provide direct data on the service life of overlaid pavements in Arizona.

The search process showed that it was not possible to identify 20 sites that completely met the criteria. Therefore, the selection was aimed at satisfying most of the criteria. The selected sites were uniformly distributed throughout the state and covered various climatic zones, as shown in Figure 1. The sites also covered a wide range of traffic volume, roughness, cracking, rutting, and friction number. The material types and layer thicknesses at the test sites are given in Table 1.

For the purpose of this study, FWD testing was performed in the outside wheel track at each site. Ten stations at 10-ft intervals were tested at each site starting at the milepost and proceeding in the direction of traffic. The pavement surface temperature was measured during the test to allow for temperature corrections in the computed modulus values for the asphalt bound layers. The FWD was operated at three load levels (6, 9, and 12 kip) at stations 1, 5, and 10 at each site, whereas a 9-kip load was used at the other stations. In addition to FWD, some sites were also tested using the Dynaflect for comparison and for the possible correlation between FWD results and those from previously gathered Dynaflect data. As expected, the FWD and the Dynaflect results were not well correlated. For the remainder of the study the FWD data were used in the development of the overlay design procedure, because the FWD can represent truck loads more accurately than can the Dynaflect. FWD tests were also conducted at a few locations using 6-, 9-, 12-, 15-, 18-, and 21-kip load

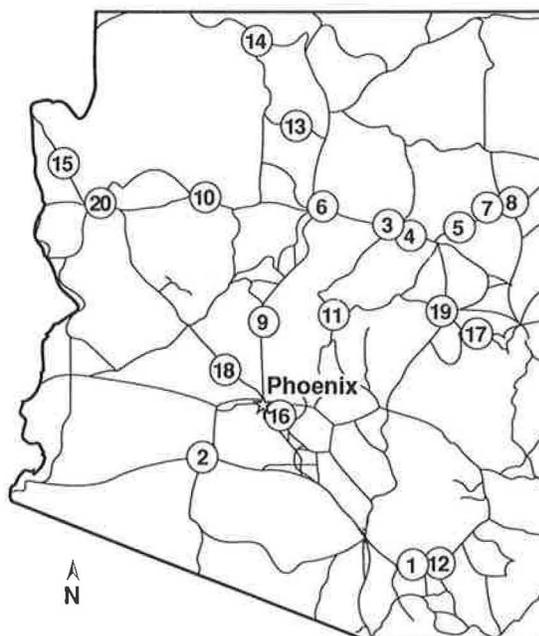


FIGURE 1 Selected test sites in Arizona.

TABLE 1 MATERIAL TYPES AND LAYER THICKNESSES AT THE TEST SITES

Site	AC		Base		Subbase		AASHTO Subgrade Classification
	Thickness (in.)	Material	Thickness (in.)	Material	Thickness (in.)	Material	
1	8.5	AB	3	SM	12		A-2-4 (0)
2	6	AB	3	SM	6		A-4 (2)
3	12.5	BTB	3	SM	6		A-4 (3)
4	12.5	BTB	3	SM	6		A-2-4 (0)
5	12.5	BTB	3	SM	6		A-2-4 (0)
6	12	AB	6	SM	10		-
7	7.5	CTB	6	SM	6		A-2-4 (0)
8	11	CTB	6	SM	6		A-4 (5)
9	6	AB	2	SM	17		A-7-b (7)
10	6.5	AB	6	SM	22		-
11	3.5	BS	2	-	-		A-4 (1)
12	6.5	AB	6	SM	15		A-1-b (11)
13	4	BS	3	AB	3		-
14	9	BS	4	AB	4		A-7-b (11)
15	6.5	BS	3	AB	6		A-1-b (0)
16	4.5	BS	2	SM	9		A-2-4 (0)
17	3	BS	2	SM	6		-
18	4	AB	4	SM	15		A-1-b (0)
19	7	AB	3	SM	6		A-4 (2)
20	9.5	AB	4	SM	15		A-1-a (0)

Note AC = Asphalt Concrete  
 AB = Aggregate Base  
 BTB = Bituminous Treated Base  
 CTB = Cement Treated Base  
 BS = Bituminous Treated Surface  
 SM = Select Material

levels for the purpose of development of the plastic deformation models.

In addition to deflection measurements, cone penetration tests (CPT) were performed at all 20 sites to investigate the homogeneity of the subgrade material and to measure the depth to bedrock, if any. The CPT provided some understanding of the layering system within the subgrade. However, the test has proven to be impractical for design purposes, primarily because of the high cost and the lack of a precise relationship between CPT and modulus. Therefore, the CPT test results were not used in the development of the overlay design procedure. The analysis of the CPT data is discussed elsewhere (8).

### Sample Collection and Laboratory Testing

At each of the 20 sites, cores were taken from asphalt concrete layer and the stabilized base layer if available. Disturbed samples were also taken from untreated base and subbase materials as well as from subgrade materials. Undisturbed samples were also collected from subgrade materials using Shelby tubes. During the sampling process, material types and layer thicknesses were recorded, to a depth of about 25 ft, unless bedrock was encountered at a shallower depth.

Asphalt concrete cores were trimmed and tested in the laboratory to determine their diametral resilient modulus according to ASTM D4123 test procedure. The test was performed at temperatures of 41°F, 77°F, and 104°F. Several cores from each site were tested.

The undisturbed subgrade samples were tested in the laboratory using the triaxial resilient modulus test. The test followed the AASHTO T274 test procedure with some modifications (8). At least one sample was tested from each site. In addition to resilient modulus testing, sieve analysis and Atterberg limits were performed on subgrade materials. Sieve analysis tests were also conducted on untreated base and subbase materials.

### BACK CALCULATION OF MODULI AND COMPARISON WITH LABORATORY MODULI

For the development of the overlay design method, the layer moduli were back calculated using an operator-controlled trial-and-error computer procedure rather than one of the available computer back calculation programs. The reasons for this trial-and-error procedure were to allow for better estimation of moduli values and to allow for the use of five layers above the bedrock to model the pavement system. Thus, the pavement system was represented by asphalt concrete layer, base, subbase, 9-in. compacted subgrade layer, and uncompacted subgrade. In this procedure, a set of typical layer moduli and a specific depth to bedrock were assumed, and the Chevron program (9) was used to compute the surface deflections. The layer moduli, as well as the depth to bedrock, were then varied based on the fact that deflections remote from the loaded area are primarily governed by the stiffness of deeper layers. This process was repeated until the computed deflections were close to the measured deflections.

The trial-and-error back calculation procedure was used as a research tool only, in an effort to enhance understanding

of the problem and to aid in the subsequent simplification. For the purpose of the routine overlay design, the microcomputer program BKCHEVM was developed to provide an automated back calculation procedure (8). The BKCHEVM program is based on the CHEVDEF program (10), developed by the U.S. Army Corps of Engineers, with some modifications to simplify use and to improve convergence.

The back calculated moduli of both asphalt concrete surface and subgrade materials were compared with laboratory moduli. In general, back calculated and laboratory moduli were not well correlated. A number of factors that might contribute to these differences were considered and verified with the results obtained from cone penetration testing. The detailed results of this comparison are presented elsewhere (8). For the purpose of overlay design, it was concluded that the moduli back calculated from FWD testing are more appropriate for the design process than those obtained from laboratory testing (8).

### DEVELOPMENT OF OVERLAY DESIGN METHODOLOGY

The majority of roads in Arizona are built with asphalt. These roads vary from heavily traveled Interstate highways to sparsely traveled secondary roads. The state covers a large area from hot desert to snowy highland. Subgrade material type also varies from one part of the state to another, whereas moisture conditions vary depending on the location and season.

Based on the experience of ADOT personnel and the available pavement management data base, the primary mode of failure for flexible pavements in Arizona is roughness. In fact, about 80 percent of overlaid pavements in the state were overlaid because of excessive roughness, about 15 percent because of excessive cracking, and about 5 percent because of rutting. Therefore, three separate design criteria were developed in this study for roughness, fatigue cracking, and plastic deformation as a part of CODA method.

Figure 2 shows the flow of calculations required to meet the problem constraints. A general set of input data is required to define the parameters of the problem. The analyst can then select one or more of four options: overlay design, remaining life of an existing pavement, life of a user-specified overlay, or economic analysis. If the user selects the overlay design option, the roughness, fatigue, and plastic deformation models are used to obtain the required overlay thickness. If the user selects the remaining life option, the program uses both roughness and fatigue models to evaluate the remaining life. If the user selects a specific overlay thickness, the program determines its life based on both roughness and fatigue models. Finally, the user has the option of performing an economic analysis to determine the equivalent uniform annual cost of four rehabilitation strategies: overlay only, mill plus overlay, mill and recycle plus overlay, and reconstruction. The development of the roughness, fatigue, and plastic deformation models are described in the following sections.

#### Roughness Model

Roughness is the single measure of pavement performance or condition that correlates with the highway user's opinion of

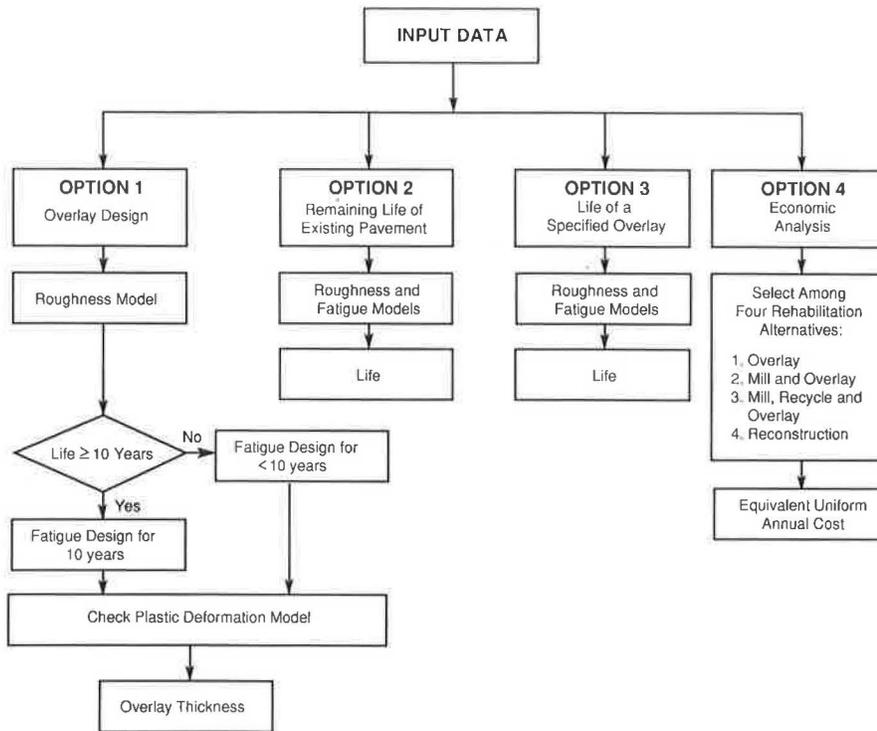


FIGURE 2 Flow chart of the overlay design procedure.

the quality of the pavement. Roughness criteria are a major factor in the project selection process used in ADOT's pavement management system. Thus, a roughness model is an important component in the overlay design process.

Roughness is generally defined as random variations in the longitudinal profile of the pavement surface. The development of random variations in the profile depends on such factors as traffic loads, environment, soil support, and construction variations. The number of variables, and their interactions, that contribute to the development of roughness are too numerous to permit rigorous mathematical modeling. Hence, empirical methods are used for the development of roughness models. Fortunately, the ADOT pavement management data base provides an extensive source of data for the development of roughness performance models. All ADOT roughness data are measured by using the Maysmeter device. A correlation model is available to convert from Maysmeter roughness reading to present serviceability rating (7).

In an overlay design method, two forms of roughness models are required: the change in roughness of the pavement caused by overlay and the rate of roughness development as a function of either time or traffic.

For the development of roughness performance models, the ADOT microcomputer PMS data base was queried to identify all overlay projects constructed since 1960. The projects selected for the analyses had "conventional" overlays, i.e., an asphalt concrete layer placed directly on an existing pavement surface. Other rehabilitation options such as milling, recycling, and asphalt-rubber membranes, were excluded from the data base analyzed during this project.

The data were separated into homogeneous overlay projects. A homogeneous overlay project is defined as having a constant project number, route number, direction, and overlay thickness. The data fields were then averaged across each milepost included in each project. Thus, the roughness data used for statistical analyses consisted of the average for the project for each year. Data for Interstate, state, and U.S. routes were analyzed separately.

#### *Change in Roughness Caused by Overlay*

Included in the analysis of the change of roughness are all overlay projects performed since 1972. The initial roughness after an overlay is modeled as the roughness before the overlay minus the reduction in roughness caused by the overlay. The data were analyzed by using various statistical models. The level of roughness after overlay and the change in roughness caused by the overlay were used as dependent variables. The roughness before overlay, thickness of overlay, and type of surfacing layer were used as independent variables.

A multiple regression analysis was performed to define equations for the change in roughness. Only routes with roughness of more than 100 in. per mile before overlay were included in the regression analysis. The smooth pavements were removed from the analysis to avoid placing an artificial limit on the change in roughness that could be obtained with an overlay. For each highway type, there was a strong relationship between the change in roughness and the roughness before overlay.

The reduction in roughness caused by overlays versus the roughness before overlay is shown in Figure 3 for Interstate highways. There is a definite trend between the change in roughness and the roughness before overlay. This trend is expected simply because there is a greater opportunity to improve the roughness of rough roads than to improve the roughness of smooth roads. It is interesting to note from this graph that several routes with relatively low roughness were overlaid. Similar trends were also obtained for U.S. and state routes.

An indicator variable was used to determine if the construction project included an asphalt concrete friction course, ACFC. All Interstate projects had ACFC layers, so this factor could not be evaluated for Interstates. ACFC layers were used on about one-third of both U.S. and state routes. The regression equations for change in roughness are

$$\Delta R = -61.76 + 0.940R_b \quad R^2 = 0.86 \quad (1)$$

$$\Delta R = -78.82 + 0.900R_b + 0.104KR_b \quad R^2 = 0.88 \quad (2)$$

$$\Delta R = -97.12 + 0.938R_b + 0.153KR_b \quad R^2 = 0.89 \quad (3)$$

for Interstates, U.S. routes, and state routes, respectively, where

$\Delta R$  = roughness before overlay – roughness after overlay (in./mi);

$R_b$  = roughness before overlay (in./mi);

$K$  = ACFC indicator;

$K = 1$  if ACFC was placed; and

$K = 0$  for no ACFC.

It is interesting to note that the slope of the relationship between the change in roughness and the roughness before overlay is almost identical across the three highway types.

The change in roughness versus overlay thickness is shown in Figure 4 for Interstate highways. This figure does not show any identifiable correlation between overlay thickness and the change in roughness caused by the overlay. Again, similar trends were obtained for U.S. and state routes.

#### Rate of Change in Roughness

For each project a regression analysis was performed between roughness and time to find the rate of increase of roughness (inches per year) after an overlay. For projects overlaid before 1972, the rate was calculated using the data between 1972 and 1987 because no roughness data before 1972 were available. A linear model was used for the change of roughness versus time. Nonlinear models were tested, but they led to improved correlation. The slope of the relationship between roughness and time was used to define the rate of change in roughness. Figure 5 shows a typical relationship between roughness and years since overlay for Interstate highways. U.S. and state routes provided similar trends.

For most projects, there was a strong correlation between roughness and time. The average changes in roughness per year in inches per mile, as measured with the Maysmeter, are 6.7 for Interstates, 5.1 for U.S. routes, and 5.8 for state highways.

The independent variables available in the data base were the regional factor, ESAL, structural number, and the overlay thickness. Graphs of the rate of change of roughness (inches per year) and the above variables for Interstates, U.S. routes, and state routes were analyzed but did not show any specific trends. A multiple regression analysis was performed for each class of highway taking the rate of increase of roughness as the dependent variable and the regional factor, ESAL, struc-

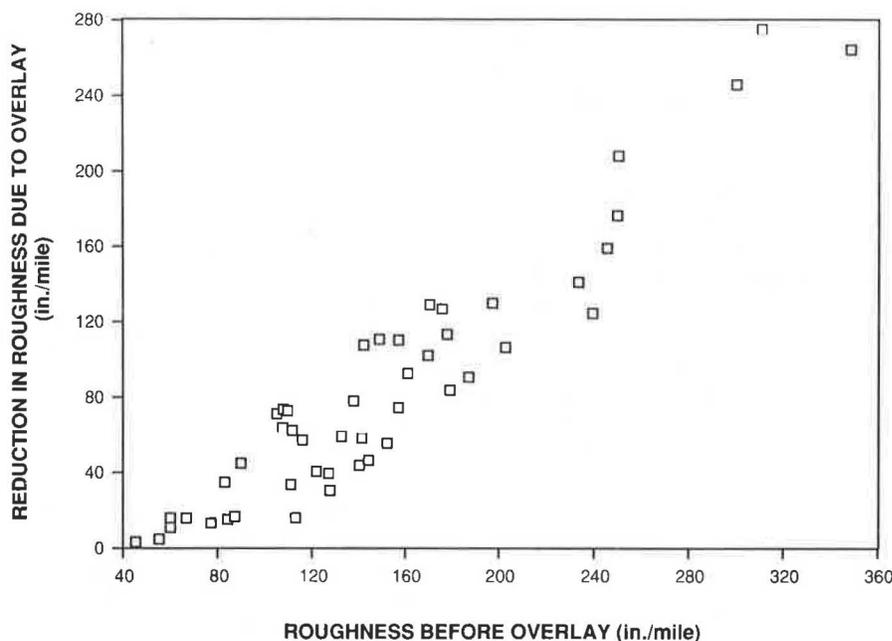
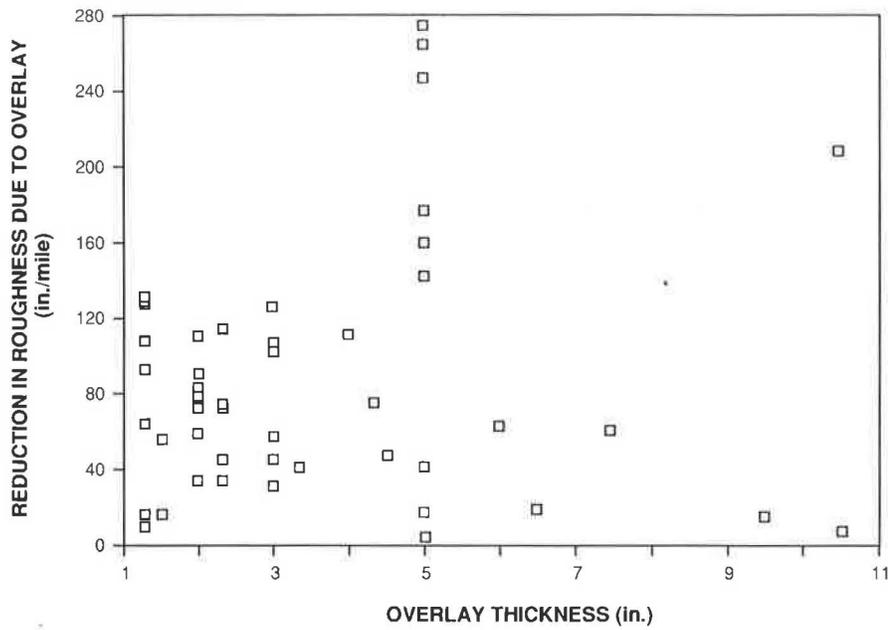


FIGURE 3 Reduction in roughness caused by overlay versus roughness before for Interstate highways.



**FIGURE 4** Reduction in roughness caused by overlay versus overlay thickness for Interstate highways.

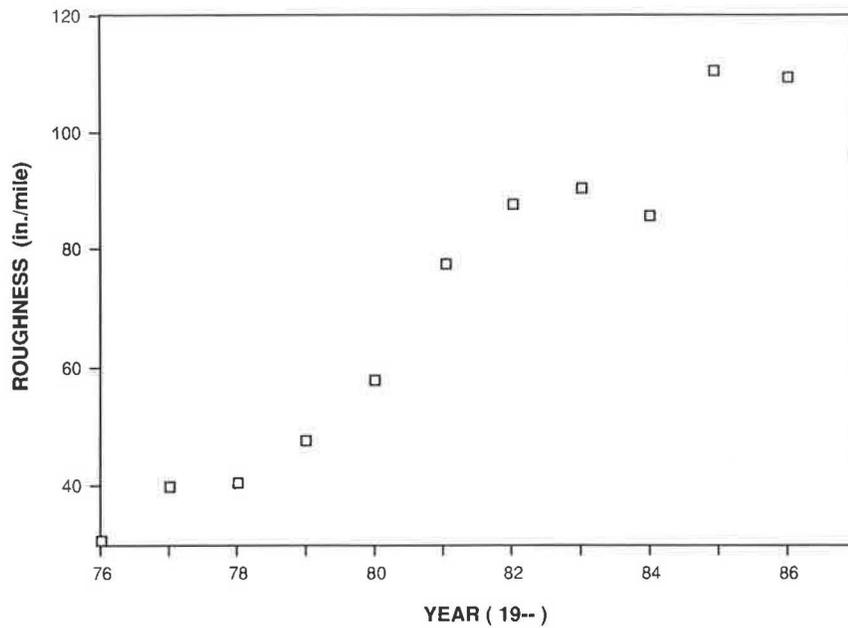
tural number, and overlay thickness as independent variables. Forward stepwise regression analyses were performed but no correlations were detected.

Since none of the independent variables was correlated with the rate of change in roughness, models could not be developed for predicting the increase in roughness as a function of pavement design variables. Hence, the average rate of change in roughness for each highway type can be used for estimating the performance of overlays.

*Overlay Life*

The conceptual model used to develop the roughness model is shown in Figure 6. The life of an overlay may be estimated by

$$N = (R_L - R_b + \Delta R)/C \tag{4}$$



**FIGURE 5** Typical rate of change of roughness versus time for an Interstate overlay project.

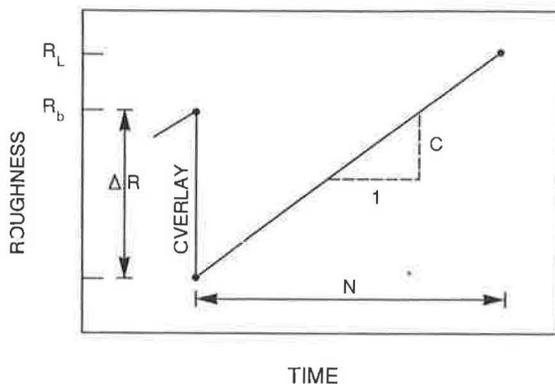


FIGURE 6 Conceptual model for roughness.

where

- $N$  = life of overlay until roughness failure in years;
- $R_L$  = limiting criteria for roughness (in./mi);
- $R_b$  = roughness before overlay (in./mi);
- $\Delta R$  = predicted change in roughness caused by overlay (in./mi); and
- $C$  = rate of change in roughness per year.

The roughness of the section to be overlaid ( $R_b$ ) is determined by direct measurement with the Maysmeter. Alternatively,  $R_b$  may be determined from the ADOT PMS data base. The change in roughness ( $\Delta R$ ) is estimated with Equation 1, 2, or 3 for Interstate, U.S., or state routes, respectively. The roughness level corresponding to the failure of the overlay is designated by  $R_L$ . Using relationships previously developed by ADOT,  $R_L$  would equal 260 for a present serviceability rating (PSR) of 2.5, and 190 for a PSR of 3.0 (7). The selection of an  $R_L$  value is a policy decision by ADOT and is in line with the procedures used in the department's pavement management system. The value of  $R_L$  can be selected as a function of highway type.

Since Equation 4 is not a function of overlay thickness, it cannot be used to determine the thickness requirements directly. In other words, previous experience with Arizona highways indicates that any practical overlay thickness will support approximately the same number of load applications before reaching the roughness failure condition. Since the normal overlay design life in Arizona is 10 years, Equation 4 can be used to check if a 10-year life is feasible. If the roughness equation results in an overlay life of 10 years or more, the roughness model is satisfied and the overlay is later designed for a 10-year life using the fatigue model. On the other hand, if the roughness equation results in a predicted overlay life of less than 10 years, this shorter life is used as the fatigue life for the fatigue deformation model unless milling or another special treatment is used.

#### Fatigue Model

The evaluation of fatigue life for asphalt concrete pavements is complex and has been the subject of study by a number of researchers for many years (11). The form of the fatigue relations in common use is derived from a logarithmic relation

between strain and the number of load cycles to fatigue failure. The relations between the logarithm of strain and the logarithm of load cycles are considered to be linear for asphalt concrete, which results in the following general equation:

$$N_i = K_1 \left( \frac{1}{\varepsilon_i} \right)^{K_2} \quad (5)$$

where

- $N_i$  = number of load cycles at strain level  $i$  until fatigue failure,
- $\varepsilon_i$  = calculated tensile strain at the bottom of asphalt layer, and
- $K_1, K_2$  = empirical material constants.

A very important problem with this form of fatigue life characterization is the extreme sensitivity of the equation to small variations in  $K_2$ . A number of fatigue models have been developed in previous studies, as shown in Figure 7 (11). The slopes ( $K_2$ ) of these logarithmic functions shown in Figure 7 range between 2.70 and 5.51, with an average value of 3.84.

#### Model Development

The fatigue model developed in this study is based on data obtained from the 20 selected sites and the fatigue models previously developed by other researchers (Figure 7). Using the 20 Arizona sites, the first step was to compute the cumulative ESALs in the design lane that were applied on the first AC layer, the first overlay (if any), the second overlay (if any), and so on.

Because each site has environmental conditions different from those of other sites, the ESAL data had to be normalized. The regional factor that is currently used by ADOT was selected as an adjustment factor to account for the difference in environmental conditions among various sites (12). It was further assumed that the regional factor changes the effect of traffic loads in a linear manner.

Another adjustment that had to be considered was because of the tensile strain at the bottom of the AC layer caused by standard wheel load changes when the pavement is overlaid. In other words, changing the pavement cross-section changes the strain level that the pavement is exposed to when the same load is applied at the pavement surface. Therefore, the ESAL applications had to be adjusted to a single strain level for each site.

For the original pavement, the critical strain for fatigue is at the bottom of the AC layer and when an overlay is placed, the critical strain is still at the bottom of the original AC layer (as long as there is no cracking); however the strain is reduced because of the overlay. In this case Equation 5 becomes

$$N_1 = K_1 \left( \frac{1}{\varepsilon_1} \right)^{K_2} \quad (6)$$

$$N_2 = K_1 \left( \frac{1}{\varepsilon_2} \right)^{K_2} \quad (7)$$

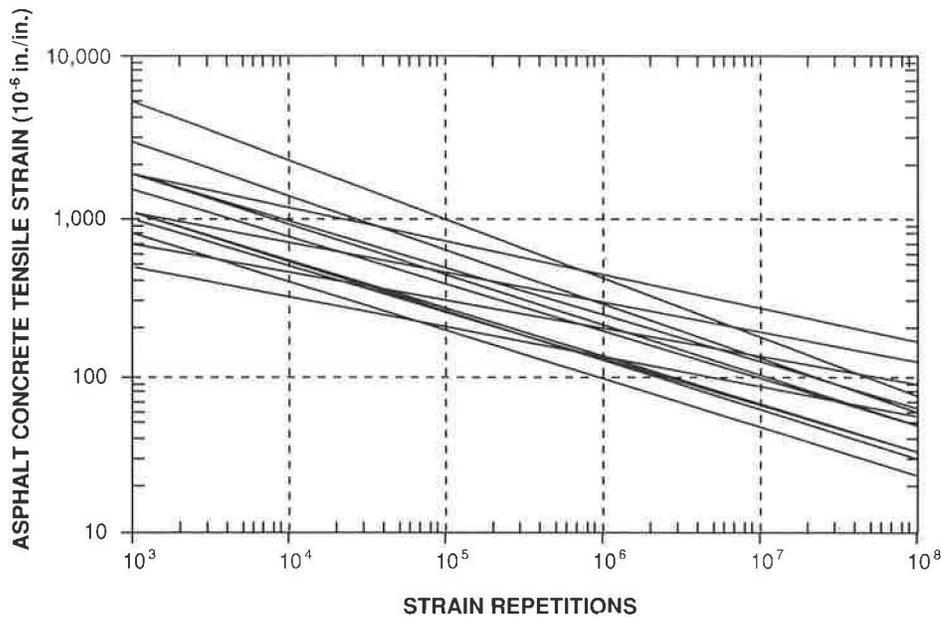


FIGURE 7 Some fatigue relations obtained by other researchers (11).

From Equations 6 and 7,

$$\frac{N_1}{N_2} = \left( \frac{\epsilon_2}{\epsilon_1} \right)^{K_2} \quad (8)$$

where  $\epsilon_1$  and  $\epsilon_2$  are the strains at the bottom of the original AC before and after overlay, respectively. Thus, the ESAL after overlay has to be adjusted according to Equation 8, then added to the ESAL before overlay to calculate the total ESAL that is matching the tensile strain before overlay (reference strain). The  $K_2$  value was taken as 3.84, which is the average of  $K_2$  values of existing fatigue functions (11). If the old AC layer is cracked, the ESAL before overlay cannot be added to the ESAL after overlay and the periods before and after overlay have to be treated separately. In this study the AC layer was considered to be cracked if cracking was 10 percent or more. It was also assumed that cracking is caused only by fatigue.

The tensile strain was calculated by using the Chevron computer program (9) for a standard dual-tire load of 9,000 lb and a tire pressure of 100 psi. The back calculated layer moduli previously obtained and adjusted to a standard temperature of 70°F were used. No adjustment was used for tire pressure since the tire pressure in Arizona has not changed significantly in recent years, with an average value in the high 90s and an 80th percentile of slightly more than 100 psi (13). In addition, the FHWA study (14) showed that the strain at the bottom of the AC layer is not largely affected by minor changes in tire pressure.

The total adjusted cumulative ESAL was plotted versus the reference tensile strain using a log-log scale as shown in Figure 8. A straight line (fatigue function) was selected close to the upper boundary of the band of previous fatigue functions (11) with a slope of 3.84. This selected fatigue line lies above most of the uncracked pavement sections, which indicates that

uncracked sections have some remaining life. A few uncracked sections lie above the fatigue line, and several cracked sections lie below the line, which indicate some discrepancies. These discrepancies are, however, considered acceptable. The equation of the selected fatigue function is

$$N = 9.33 \times 10^{-7} \left( \frac{1}{\epsilon_{AC}} \right)^{3.84} \quad (9)$$

where

- $N$  = theoretical number of ESAL repetitions until fatigue failure, and
- $\epsilon_{AC}$  = tensile strain at the bottom of the AC layer.

Equation 9 is valid for various conditions such as various regions and temperatures because these conditions were normalized to standard conditions. Therefore, if the tensile strain at the bottom of the AC layer is known under these standard conditions, the number of ESAL repetitions until fatigue failure can be computed.

#### Use of the Fatigue Model

To use the fatigue model, a multilayer elastic program such as Chevron can be used to compute the tensile strain at the bottom of the AC layer caused by a 9,000-lb dual-tire load. The layer moduli can be obtained from the FWD back calculation as discussed earlier.

In many cases the pavement is overlaid before it is significantly cracked. Therefore, the remaining fatigue life of the existing pavement has to be considered and added to the overlay life. This calculation can be accomplished by using the concept of cumulative damage (Minor's law) (15). Thus, the optimum overlay thickness is the one that satisfies the following equation.

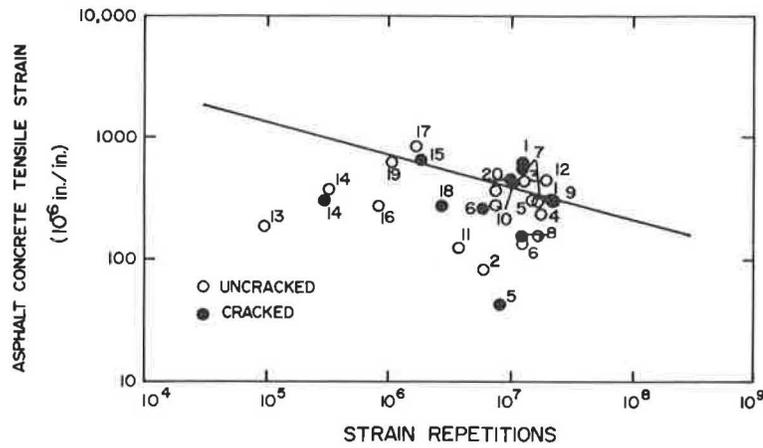


FIGURE 8 Asphalt concrete tensile strain versus strain repetitions for the 20 test sites.

$$\frac{n_1}{N_1} + \frac{n_2}{N_2} = 1$$

$$n_2 = N_2 \left( 1 - \frac{n_1}{N_1} \right) \quad (10)$$

where

- $n_1$  = actual cumulative ESAL in the design lane before overlay,
- $N_1$  = allowable cumulative ESAL in the design lane before overlay,
- $n_2$  = actual cumulative ESAL in the design lane after overlay, and
- $N_2$  = allowable cumulative ESAL in the design lane after overlay.

To reach this optimum overlay thickness, trial thicknesses can be used and the corresponding cumulative ESALs for each overlay can be computed. A polynomial equation is then fitted to compute the optimum overlay thickness that matches the expected ESAL.

If the existing pavement has been previously overlaid more than once and cracking has not reached 10 percent, the cumulative fatigue is added for each AC thickness. In all cases, the tensile strain is computed at the bottom of the original AC layer.

On the other hand, if the old pavement (before overlay) is cracked (10 percent or more), it is assumed that the old AC layer will not contribute to the cumulative fatigue life after overlay. Therefore, the tensile strain is computed at the bottom of the overlay, and the modulus of the original asphalt surface is assumed to be 40,000 psi.

### Plastic Deformation Model

Damage to a pavement structure can arise through a variety of mechanisms, including plastic deformations in the layers of the structure. The ultimate result of these deformations may be cracking, rutting, or simply the development of exces-

sive roughness. However, the initial cause being considered in this part of the design process is plastic or permanent deformations.

This design model deals only with plastic deformations in layers below the AC surface layer. It is assumed that plastic deformations in the overlay layer will be reduced or prevented by improvements in mix design and construction techniques.

In the plastic deformation model, attention is devoted to the plastic strains that are permanent, i.e., not recoverable. However, permanent deformations are much more troublesome to measure in the field than total deformations (elastic and plastic). It has therefore been assumed that the onset of significant plastic deformation corresponds to the onset of nonlinear load deflection response. In other words, as long as the load deflection curve is linear, it is assumed that the plastic deformations are negligible.

Two types of nonlinearity may arise: strain hardening or strain softening. Strain-hardening nonlinearity is believed not to represent a problem to plastic deformation. Therefore, only strain-softening nonlinearity is considered in this model.

Based on data examined to date, it is believed that most pavement sections under consideration for an overlay have sufficient factor of safety in the structural design that plastic deformations are unlikely to be a source of problem. This part of the design procedure is intended to detect and correct those few cases in which vulnerability to excessive plastic deformations is indicated by the FWD test results. Thus, the objective of this design procedure may be stated as follows. When the pavement gets older, the ability of the pavement surface to spread the load to the underlying layers may be decreased because of cracking or other surface failures. In addition, the pavement might be subjected to heavier traffic loads in the future that have not been applied before. Therefore, the objective of plastic deformation design is to provide sufficient overlay thickness that expected traffic loads will not induce significant plastic deformations in the underlying layers.

The design procedure is based on the assumption that the onset of significant nonlinear response can be measured directly with the FWD test. The load corresponding to the onset of significant nonlinear response is designated  $F_{NL}$ . The exact

point at which deviation from linear behavior occurs is difficult to select. Therefore,  $F_{NL}$  has been defined as the load at which the load deformation curve deviates 10 percent, measured horizontally, from the straight-line extension of the early portion of the curve, as shown in Figure 9.

The detailed step-by-step procedure for the plastic deformation overlay design is as follows.

1. Input existing pavement structure geometry, moduli from back calculation analyses, and highway type (Interstate, U. S., or state).

2. Input FWD data for loads from 6 kip to 15 kip or more. Use 6- 12-kip data to establish the straight line (least-squares fit) and check the higher loads for deviation from the straight line at each sensor.

3. If none of the loads shows 10 percent or more deviation, report that  $F_{NL}$  is greater than the maximum FWD applied load and bypass plastic deformation design. If one or more loads show deviation  $\geq 10$  percent, proceed to step 4.

4. Use linear interpolation to find the load corresponding to 10 percent deviation ( $F_{NL}$ ).

5. In comparing the stress states, several different stresses at various points in the underlying layers were considered. The maximum octahedral shear stress,  $(\tau_{oct})_{max}$ , in the subgrade was chosen because it reflects all the components of the stress tensor (16). Therefore, use Chevron program with  $F_{NL}$  applied to the FWD plate to calculate the maximum  $\tau_{oct}$  that occurs in the subgrade.

6. Determine the design load ( $F_{des}$ ), which is the load on a dual wheel that can be used in checking the adequacy of a trial overlay design thickness. The value of  $F_{des}$  is chosen so that the probability of an actual dual wheel load exceeding this value is very small. The degree of conservatism in this part of the design can be controlled through the choice of this probability. A sample of axle loads obtained from Arizona highways showed that the selected probabilities were 0.00001, 0.00008, and 0.00008 for Interstate, U.S., and state highways, respectively. These probabilities resulted in an expected number of dual wheel load values exceeding  $F_{des}$  of about seven in 1 year.

7. Use Chevron program with  $F_{des}$  applied to a dual wheel and calculate the maximum  $\tau_{oct}$  in the subgrade.

8. If  $\tau_{oct}$  caused by  $F_{des}$  is less than or equal to  $\tau_{oct}$  caused by  $F_{NL}$ , report this fact and bypass plastic deformation design. If not, proceed to step 9.

9. With the overlay obtained from fatigue analysis in place, use Chevron program with  $F_{des}$  applied to a dual wheel and calculate  $\tau_{oct}$  in the subgrade.

10. If  $\tau_{oct}$  in step 9 is less than or equal to  $\tau_{oct}$  because of  $F_{NL}$ , the pavement is safe against plastic deformation with the overlay. If  $\tau_{oct}$  in step 9 is more than  $\tau_{oct}$  caused by  $F_{NL}$ , proceed to step 11.

11. Increment the thickness of the overlay in steps until  $\tau_{oct}$  caused by  $F_{des}$  is equal to  $\tau_{oct}$  caused by  $F_{NL}$ .

The preceding step-by-step procedure, including computation of  $\tau_{oct}$ , has been programmed in the CODA program. The needed parameters have been fixed and the entire plastic deformation design procedure has been automated. The user is required only to input the data indicated in steps 1 and 2. Of course, modifications to the "fixed values" can be made readily to the program if desired. Users who desire to use the plastic deformation model independent of the fatigue analysis should proceed to step 11 after step 8.

It should be noted that the plastic deformation model deserves to be called mechanistic because it addresses permanent deformations, a particular mechanism of damage. Even though some plastic deformation occurs with every load application, the intent of this procedure is to keep the plastic deformations small. This intention corresponds to an attempt to maintain the deformations almost entirely within the elastic range. An extension of this method to include an estimate of the "expected life" of a pavement structure, with respect to plastic deformation, is theoretically possible. However, the art and science of predicting plastic deformations is not sufficiently well developed to justify this extension at the present time.

### Complete Overlay Design

The three models—roughness, fatigue, and plastic deformation—were incorporated in the overlay microcomputer program CODA. The traffic load requirement in this method of overlay design is the cumulative 18-kip ESAL in the design lane during the design period. Some existing pavement condition parameters are also needed, such as Maysmeter roughness reading and cracking. Pavement layer thicknesses, material type, previous overlays, if any, and years of construction are also needed.

Since both fatigue and plastic deformation models require the knowledge of pavement and subgrade layer moduli, the first step in the procedure is to run the FWD test on the pavement section under consideration. The fatigue model requires performing the FWD test at a load level of 9,000 lb only, whereas the plastic deformation model (if used) requires running the FWD test at several load levels as discussed earlier.

The FWD deflection data at a 9,000-lb load level are further used to back calculate the pavement and subgrade layer moduli. Since the modulus of the AC layer is significantly affected by temperature, a subroutine was developed in the CODA program to adjust the AC modulus to a standard temperature of 70°F based on the AASHTO guide (6).

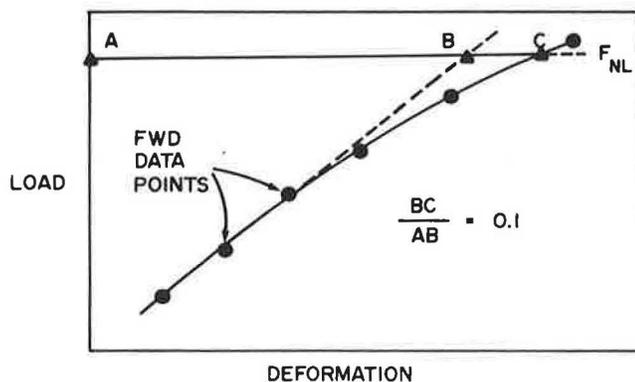


FIGURE 9 Nonlinearity of underlying layers and definition of onset of significant nonlinear response.

In addition to overlay design, other features have been incorporated in the CODA program, such as the ability to compute the remaining life of an existing pavement, the life of a user-specified overlay, and economic analysis.

The CODA method of design has been verified with typical pavement sections in Arizona. Although the long-term field performance has not been evaluated, preliminary evaluations indicate that the results are reasonable. A computer workstation has been developed to automate the design process, extract the required data from the data base files directly, and allow a user-friendly procedure.

Although the CODA procedure in its current form applies to Arizona conditions only, the design concepts can be used with other conditions. For example, the rate of change of roughness, the rate of crack development, and axle load distributions can be changed to match the conditions in other states.

### SUMMARY AND CONCLUSIONS

In this study a rational overlay design method for flexible pavements in Arizona has been developed. The method combines mechanistic approaches and historical performance of pavements in the state. Three models were developed for design: roughness, fatigue, and plastic deformation. The roughness model is based on the trend of the Maysmeter roughness data of overlaid pavements in the state. The fatigue model considers the fatigue experience of pavements in the state as well as fatigue criteria developed by others. The plastic deformation model ensures that the expected traffic loads will not induce significant plastic deformations in the underlying layers.

The development of the method involved coring and sample collection, laboratory testing, statistical analyses, and computer programming. Twenty in-service pavement sites were selected from Arizona highways covering various geographical and environmental regions, soil types, pavement conditions, and traffic volumes. FWD and Dynaflect tests were performed and the layer moduli were back calculated. In addition, resilient moduli were also evaluated in the laboratory. The back calculated and laboratory moduli were compared and found to be not well correlated. It was concluded that the back calculated moduli from the FWD are more representative of field conditions than of laboratory moduli. Application of the design method CODA requires FWD data but no sample collection or laboratory testing.

The overlay design method developed in this study was incorporated in a user-friendly microcomputer program. The method was verified on typical pavement sections and found to provide reasonable results. Although the method is based on data obtained from Arizona, the design concepts can be applied to conditions in other states.

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