DEVELOPMENT OF AN IMPROVED OVERLAY DESIGN PROCEDURE FOR THE STATE OF ALASKA
Volume I: Executive Summary

by

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FOR THE STATE OF ALASKA

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Development of an Improved Overlay Design Procedure for the State of Alaska: Volume I Executive Summary

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Conducted in cooperation with the U.S. Department of Transportation, Federal Highway Administration.

The state of Alaska is currently developing a pavement management system for their road system. An important input to the system is the determination of flexible overlays based on the structural adequacy of the existing pavement. The current overlay procedure (The Asphalt Institute method) does not always show a need for overlays, despite the poor surface condition of the pavement and high traffic volumes. Therefore, an improved procedure that would consider not only traffic but also surface distress, the structural properties of the pavement, and, most importantly, the effects of a freezing and thawing of the base, subbase, and subgrade layers are needed. This report recommends two such methods for further consideration: a simplified mechanistic method using equations developed in Pennsylvania, and a mechanistic procedure employing a backcalculation computer program known as ELSDEF. The results of these analyses are compared with those of the Asphalt Institute procedure. It was determined that the Asphalt Institute procedure was inappropriate and underdesigned the overlay for the road sections. The Pennsylvania equations tended to be slightly more conservative than the mechanistic method using ELSDEF except for cases where the pavement fatigue life had been utilized completely by past traffic loads. It is recommended that Alaska utilize the mechanistic approach to design overlays and ELSDEF as the backcalculation procedure. The Pennsylvania equations can be used in those cases not requiring considerable accuracy.
A NOTE FROM THE PROJECT MANAGER

During the past few years the Alaska Department of Transportation and Public Facilities' (DOT&PF) need for improved overlay design procedures has grown tremendously due to increasing traffic and reduction of available funds to maintain the highway network. Previous methods have had only limited success in adequately analyzing the alternatives available to the designer. This project does not and cannot stand alone. It is the combination of millions of dollars of research performed throughout the world. This report represents the state of the art in pavement design. Gary Hicks and Margot Yapp have done an excellent job of gleaning from the literature information which can be applied in Alaska.

The design approach recommended here allows the designer considerable flexibility in overlay design. The design approach may not be as rigid as some would prefer. It is hoped that by applying engineering principles, expertise, and judgement, more cost effective pavement overlay projects can be produced. As designers gain experience in the approach recommended here, they will find that they are no longer confined to the limits of past experience. This will allow more innovation in pavement design through the ability to analyze the performance of new materials and material response through the seasons. However, experience will continue to guide the designer in determining which design alternative is most cost effective for any project.

The principles presented in this paper will provide new dimensions to the ability of the pavement designer's ability to predict pavement performance thereby allowing the selection of the best alternative within political and budget constraints. Although the ideal would be an overlay design without
such constraints, we feel the resulting design process will offer significant benefits for Alaska DOT&PF.

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INTRODUCTION

At present, Alaska uses the Falling Weight Deflectometer (FWD) to measure pavement deflections and then determines overlay thickness using the Asphalt Institute procedure (2). The availability of a FWD for each region enables Alaska to obtain sufficient deflections at the critical time of the year for design. However, this procedure does not consider the remaining life of the pavement or the effects of freezing and thawing on subgrade materials. Therefore, the resulting design thicknesses have turned out to be inadequate and, on occasion, the procedure has shown no need for an overlay. An improved procedure using the current state-of-the-art would remedy this situation and may reduce costs. It would also ensure good performance in future construction and rehabilitation activities and provide a procedure that is applicable to all regions in Alaska.

Purpose

This paper describes the development of an improved framework for overlay thickness design. Data were collected from selected highway projects to evaluate the proposed methodology. The results are compared with those from the existing design procedure.

The study approach consisted of two phases as shown in Figure 1. Phase I evaluated existing methods of overlay thickness design and developed a framework for a new procedure to be used in Alaska. In Phase II, testing was performed to modify the design procedure as needed. Once tested and evaluated, the overlay design procedures are to be documented in a field manual.
BACKGROUND

Current Overlay Design Method

The Alaska Department of Transportation and Public Facilities' (ADOT&PF) official flexible overlay design method (11) is essentially that contained in the Asphalt Institute's MS-17 (2). Using this procedure, the Representative Rebound Deflection (RRD) at the center-of-load is measured with a Falling Weight Deflectometer (FWD) using an impulse load of 9000 lb (40 kN). The average of all deflection values plus twice the standard deviation together with the design traffic, in equivalent 18-kip axle loads (EALs), is used to obtain the required overlay thickness (Figure 2).

Although the official overlay design approach is currently the Asphalt Institute procedure, other methods have been used by ADOT&PF. For example, in the Central (Anchorage) and Southeastern (Juneau) Regions, the Asphalt Institute procedure indicates that no overlay is required despite the fact that roadways have high traffic volumes. Therefore, mechanistic procedures have been used. However, in the Interior Region (Fairbanks), only the Asphalt Institute procedure is used and, interestingly enough, deflections in this region are higher than in Anchorage despite a colder climate and lower traffic volumes. One possible explanation could be that Fairbanks generally uses thinner asphalt concrete pavements than Anchorage and Juneau.

Problems With Current Overlay Design Methods

The problems associated with the current ADOT&PF overlay design procedure are summarized in Table 1. The official procedure does not always show a need for overlays, particularly on roads with a higher traffic volume. Typically, higher traffic volumes would require thicker overlays, and a rational design
procedure should illustrate this. Further, surface distress (type, extent, or severity) is not considered in the design process. Although it is known that cracking reduces the remaining life of the pavement, most existing overlay procedures do not address this issue very well (6).

The data collected from current procedures are insufficient to provide accurate indications of the contributions of each structural layer, particularly if the pavement structure is partially frozen. When underlying layers are partially frozen, the surface deflections measured are smaller than when taken over thawed pavements. However, despite the smaller deflection values for a partially frozen pavement, the stresses and strains that the pavement endures may be much greater (15). The pavement stresses and strains are actual indicators of the structural strength, with high value indicating a weaker pavement. Therefore, it may be concluded that the center-of-load deflection alone would be a poor indicator of the potential for pavement distress.

It should also be noted that the maximum damage potential may occur long before peak deflection occurs. This is due to the fact that pavement strains may be highest when deflections have not yet reached their peak. This may be remedied if the depth of thaw is known, in which case an adjustment to the deflection value is made. However, the thaw depth generally varies greatly from point to point, depending on factors such as exposure to sunlight, types of materials present, and water content. Frost tube measurements used for approximate thaw depths may be in error by several feet (15) and the adjusted deflection values may err by a factor of two or more as well.

Further, the Asphalt Institute method as given in MS-17 (2) does not consider the remaining life of the pavement. Work completed by Kennedy and
Lister (9) indicates that, in many pavements, deflections remain relatively constant for most of the serviceable life of the pavement and only increase near the end of the pavement life. Therefore, a fatigue relationship that incorporates both past and future traffic should be used in the analysis. Also, the tolerable deflection used may not be adequate to quantify the desired deflection criteria properly (14). This tolerable deflection is a function of the materials in the pavement structure, subgrade support, and the layer thicknesses; however, the deflection method as used does not consider the contributions of the different layers.

In addition, the use of new additives such as rubber and other polymers in asphalt concrete have added new dimensions to the overlay problem which are not addressed in the current method.
DEVELOPMENT OF IMPROVED PROCEDURE

Design Approach

In July 1986, a planning meeting with ADOT&PF was held to discuss existing and future overlay design practices. The advantages and disadvantages of the current overlay procedures were debated, and outlines of an improved procedure were developed. Project sites and data needs were also identified.

A two-tiered concept for overlay design was also determined to be most desirable. The first tier would be a simpler and easier approach than the second. Therefore, it was proposed that the first tier be a simplified-mechanistic approach, such as that developed by Fernando et al. (7). This is a simpler and more straight-forward approach than the mechanistic approach using backcalculation analysis. However, the range of layer properties used in the study by Fernando et al. were not completely appropriate for Alaska's conditions. Therefore, it may prove necessary to develop strain versus deflection relationships specifically for Alaskan pavements. This step was outside the scope of the project.

The second tier is a fully mechanistic approach in which layer moduli are backcalculated from surface deflections. A number of automated backcalculation programs with iterative basin-fitting procedures, are available for microcomputers (13). Two of these procedures (BISDEF and ELSDEF) were evaluated as a part of this study.
**Simplified Mechanistic Procedure**

The simplified mechanistic method employs mechanistic principles to arrive at empirical relationships. Linear elastic theory and regression analysis techniques are used to develop equations that describe the relationships between surface deflections and pavement layer properties.

This strain-deflection approach is a simplified mechanistic procedure that was developed by Fernando et al. (7). Essentially, the linear elastic-layered theory was used to develop strain vs. deflection relationships for the direct calculation of pavement strains from measured FWD deflections rather than using a deflection basin-fitting procedure to backcalculate moduli values. The multilayer linear elastic program, BISAR, was used in a large factorial study to develop these relationships. Strain versus deflection relationships were then developed for the tensile strain ($\varepsilon_T$) at the bottom of the existing asphalt layer and the compressive strain ($\varepsilon_C$) at the top of the subgrade. These are:

\[
\log\frac{\varepsilon_T}{\log (H_1+1)} = -2.261 - 0.944 \log (\delta_1-\delta_2) \\
+ 1.947 \log \left[ (\delta_1-\delta_3)/\delta_2 \right] \\
+ 0.175 (\delta_1*H_2) \\
+ 0.926 \log (\delta_1*\delta_2) \\
\]

(1)

\[
\log \varepsilon_C = -0.054 + 1.941 \log (\delta_1-\delta_2) \\
- 2.004 \log \left[ (\delta_1-\delta_3)/\delta_2 \right] - 1.465 \log (H_1+H_2) \\
- 0.365 (H_2)^{0.5} + 0.725 \log (\delta_1*H_2) \\
+ 0.285 (\delta_1*H_1)^{0.5} - 0.910 \log (\delta_1*\delta_2) \\
\]

(2)
where:

\( \varepsilon_t \) - tensile strain at bottom of the existing AC layer,

\( \varepsilon_c \) - compressive strain at top of subgrade,

\( \delta_i \) - deflection at \( i^{th} \) sensor of the FWD (in.),

\( H_1 \) - thickness of existing AC layer (in.), and

\( H_2 \) - thickness of existing subbase (in.).

Using the calculated strain values, performance prediction estimates are possible. The tensile strains were used with the Austin Research Engineers fatigue equation (4), and the subgrade strains were used with the performance model developed by Luhr et al. (10) to obtain the equations relating expected life to allowable strain. However, it was found that the performance predictions from the deflection basin fitting (i.e., backcalculation) procedures match more closely the predictions generated from the theoretical strains than from this simplified strain-deflection procedure. The two performance equations are defined as follows:

\[
W_{18} = 9.33 \times 10^{-15} (\varepsilon_t)^{-5.16} \tag{3}
\]

\[
\log N_x = 2.15122 - 597.662\varepsilon_x - 1.32967 \log \varepsilon_x + \log \left[ \left( \text{PSI}_i - \text{TSI} \right) / (4.2 - 1.5) \right]^{0.5} \tag{4}
\]

where:

\( W_{18} \) - weighted 18-kip applications prior to Class 2 cracking,

\( \varepsilon_t \) - tensile strain at bottom of the AC layer,

\( N_x \) - allowable applications of axle load \( x \),

\( \varepsilon_x \) - subgrade compressive strain due to axle load \( x \),

\( \text{PSI}_i \) - initial PSI of pavement, and
TSI = terminal serviceability index.

Once the strains are known, it is possible to input various thicknesses of the overlay and then to recompute the strains. The required overlay thickness would be the one that reduces the strains to a specified tolerable level. The relationships developed for estimating the pavement's tensile and compressive strains are given by the following:

\[
\log (\epsilon_T)_{ov} = -0.689 + 0.793 \log \epsilon_t
\]

\[\quad -0.041 (H_{ov} + H_1)^0.5 - 0.057H_{ov}\]  

\[
\log (\epsilon_C)_{ov} = -0.359 + 0.870 \log \epsilon_c - 0.051H_{ov}
\]

\[\quad -0.109 [(H_{ov}+H_1)/H_1]^{0.5}\]  

(5)  

(6)  

where:

\((\epsilon_T)_{ov}\) = tensile strain at bottom of original AC layer after overlay,

\((\epsilon_C)_{ov}\) = compressive strain of subgrade after overlay,

\(\epsilon_T\) = tensile strain of AC layer before overlay,

\(\epsilon_C\) = compressive strain before overlay,

\(H_{ov}\) = overlay thickness (in.), and

\(H_1\) = thickness of original AC layer, in.

The relationships and equations were developed for conditions in Pennsylvania; therefore, they may not be valid for Alaska. Some experimentation on the validity of the procedure should be performed. However, it should be possible to derive similar relationships utilizing Alaskan conditions, if needed. Figure 3 provides a flowchart of this procedure.
Mechanistic Procedure

A fully mechanistic design procedure characterizes the response of the pavement to a load in terms of strains and/or stresses in various pavement layers. A fatigue relationship between that response and number of load repetitions to a designated failure criteria is used to determine pavement life. Mechanistic and deflection procedures are not mutually exclusive. Most procedures use a stress or strain level based on deflection testing as the pavement response that is related to performance. The difference between such a system and a "deflection" approach is that the deflection used to develop the performance relationship is based on a mechanistic model rather than an empirical one.

Figure 4 illustrates the flowchart of a fully mechanistic design procedure (8). As in the deflection-based procedures, nondestructive pavement tests, condition surveys, and traffic data are required as inputs. In addition, some knowledge of the stiffness properties and distress characteristics (such as fatigue cracking and plastic deformation) of the various materials comprising the pavement structure are needed. Stiffness characteristics of the various pavement components can either be determined by tests on undisturbed or representative specimens of the pavement components, or backcalculated from NDT measurements.

The FWD, when used to measure the structural response, provides a measure of the surface deflection under an impulse load. Measurements should be obtained at reasonable intervals throughout the project. The condition of the pavement must be carefully ascertained. This is needed to determine the analysis sections and to help establish performance criteria for the observed distress types.
The stiffness characteristics of the various layers can be estimated from surface deflection measurements. The shape of the deflection basin is defined by deflections measured directly under a load and at a number of radii. Layer elastic computer programs are used to determine a set of modulus values that provides the best fit between the measured and computed deflection basins at the pavement surface. Normally, the procedure involves assuming a set of modulus values and then iterating with the computer until the measured and computed deflections are in "reasonable" agreement. Various programs are available to perform the backcalculation analysis. This paper analyzes two such programs, ELSDEF and BISDEF. With both programs, it is recommended that some laboratory testing be performed to check the backcalculated results (8). Figure 5 is a flowchart of the CHEVDEF/BISDEF program (5).

As in all design methods, traffic volumes using the facility should be known. The distribution of traffic across lanes and the concentration of truck traffic in the outer lane should be recognized. With the traffic information and layer stiffnesses, critical performance parameters can be determined using layered elastic analysis. The parameters can be related to "acceptable" and "not acceptable" performances observed in the condition survey as well as to laboratory-defined distress criteria.

Since Alaska has not developed its own design criteria at present, those developed by the Asphalt Institute (3) may be used. This was selected because of its widespread use. For control of fatigue in the asphalt layer, Eq. (7) is used:

\[ N = 18.4C \times 0.004325 \times \varepsilon_{\text{t}}^{-3.291} \times E^{-0.854} \]  

(7)
where:

\[ N = \text{number of 18-kip (80 kN) equivalent single axle loads}, \]

\[ \varepsilon_t = \text{horizontal tensile strain on underside of existing AC layer}, \]

\[ E = \text{modulus of AC layer, psi, and} \]

\[ C = \text{a function of voids and volumes of asphalt in the mix design, and} \]

can be calculated:

\[ C = 10^M \] (8)

and \[ M = 4.84 \times \frac{V_v}{(V_v + V_b)} - 0.69 \] (9)

where:

\[ V_b = \text{volume of asphalt, \%}, \]

\[ V_v = \text{volume of air voids, \%}. \]

Similarly, the vertical compression strain criterion is used to control permanent deformation:

\[ N = 1.365 \times 10^{-9} \varepsilon_c^{-4.477} \] (10)

where:

\[ \varepsilon_c = \text{vertical compression strain at the subgrade surface}. \]

If the future anticipated traffic for the life of the overlay were known, it is possible to rearrange Eqs. (7) and (10) to obtain the tolerable strains.

The remaining life of the pavement can be determined using Miner's Hypothesis. A simple form of the expression is:

\[ \frac{N_T}{N_{D1}} = 1 - \frac{N_{A1}}{N_{D1}} \] (11)
where:

\[ N_r/N_{D1} = \text{remaining life}, \]
\[ N_{A1} = \text{number of applications of EALs to date}, \]
\[ N_{D1} = \text{allowable number of applications of EALs according to fatigue relationships, and} \]
\[ N_r = \text{additional number of applications of EALs that can be applied to the existing pavement.} \]

If it is determined that an overlay is needed, it must then be designed to resist fatigue and rutting. For a specific thickness of overlay to minimize fatigue, the tensile strain on the underside of the existing layer must be determined. The allowable number of applications may be estimated using some form of fatigue relationship and modified by the remaining life ratio. It is possible to define the relationship between overlay thickness and additional load applications. Similarly, relationships exist between the compressive subgrade strain and the pavement life to preclude rutting. At the present time, the Asphalt Institute criteria (3) are recommended for use in Alaska.

If other distress modes are considered, similar relationships between thickness and load applications can be developed. The design overlay is the maximum thickness value required to satisfy both fatigue and rutting criteria (8).
EVALUATION OF IMPROVED PROCEDURE

Projects Sites

Three sites were evaluated, two in Anchorage and one in Fairbanks. Figure 6 shows the location of the three project sites. Sterling and Seward Highways are located in the Anchorage region and Parks Highway is located near Fairbanks. Table 2 summarizes the parameters of all three projects. Generally, Sterling and Parks Highways are rural highways, with low traffic volumes and thin AC layers ranging from 1.5 to 2 in. (3.8 to 5 cm). The Seward Highway project site, however, is located in downtown Anchorage with thicker AC layers ranging from 2.25 to 5 in. (5.7 to 12.7 cm). All projects, therefore, consist of an asphalt concrete surface over untreated aggregate base and subbase.

Results of Analysis

Four overlay design procedures are evaluated in this section. The first is the current Asphalt Institute MS-17 (2) procedure. The second design procedure was developed by Newcomb (12) for the state of Washington. The procedure utilizes regression equations to calculate the layer moduli and was added only for comparison purposes. These values were used in the mechanistic design procedure to calculate overlay thickness. The last two are the simplified mechanistic procedure developed by Fernando et al. (7) and the fully mechanistic procedure which employs the backcalculation programs (ELSDEF and BISDEF) to determine the layer moduli of the pavements. Comparison of layer moduli are given in Table 3 while the overlay thickness resulting from the four procedures are summarized in Table 4.
Table 3 summarizes sample layer moduli that were obtained from BISDEF, ELSDEF, and Newcomb's equations for the three sites. Because of the presence of frozen subsurface layers and thin surface layers, it was difficult to close in on the layer moduli with BISDEF. It is readily obvious that the layer moduli obtained are at the limits of the input ranges. Final moduli values were also output despite the fact that error messages indicating computations had been suspended were present. This occurred in more than half of the deflection basins that were analyzed.

The layer moduli calculated with ELSDEF appear more reasonable when comparing these results with those obtained from laboratory tests. In approximately half of the pavements analyzed, the subgrade values show substantial stiffnesses of 150 to 650 ksi (1030 to 4480 kPa) indicating that this layer is frozen or partially frozen. Also, about half of the sections show asphalt concrete moduli in the 400 to 1200 ksi range (2750 to 8270 kPa), which would be in accordance with pavements tested during March when pavement temperatures were 37°F (2.8°C). For the remaining sections, temperatures averaged 60°F (15.6°C) and the moduli are correspondingly lower. The base moduli average from 30 to 40 ksi (206 to 275 kPa).

The layer moduli from Newcomb's equations are significantly different from those obtained using ELSDEF. As Table 3 indicates, the surface moduli are much lower while the base and subgrade moduli are higher than expected. It should be emphasized that these equations were not developed for frozen soil applications. These equations were included in the study only because initially it was thought that the thawed pavement would be the critical case. Also, the models were built with the assumption that the underlying moduli are smaller than the overlying layers, and this is not true for our sites. For
the Parks Highway, the surface moduli were so low that adding an overlay did not decrease the tensile strains as expected. Instead, there was an INCREASE in strains. This is apparently due to the weak AC layer moduli computed. Fernando et al.'s equations and the Asphalt Institute procedure do not yield layer moduli results and are therefore not included in Table 3.

Table 4 summarizes the overlay thicknesses for the projects. Two reliability levels as defined by AASHTO (1), 50% and 90%, are included in the analysis. The reliability level concept is further discussed in the technical appendices of the AASHTO Guide (2). Briefly, a 50% level would imply that a pavement will have a probability of 0.5 that it will perform well over the design period. Therefore, different levels may be used to underscore the relative importance of a road structure failing. For example, it would be desirable for a high volume highway to have a high probability of achieving its design life and performance, and so a higher reliability level would be selected.

As shown in Table 4, the mechanistic procedure indicates overlays ranging from 0 to 7 in. (0 to 17 cm) are needed for 50% reliability. For Seward Highway, reconstruction is recommended because of the high traffic volumes. The increase in the reliability level from 50 to 90% generally increases overlays by 2 in. (5 cm). The current procedure does not directly incorporate reliability levels.

Fernando et al.'s equations tend to be more conservative and produce overlays that are approximately 2 in. (5 cm) thicker than the mechanistic procedure. However, for the Seward Highway project, it recommends a 2 to 6 in. (5 to 15 cm) overlay rather than reconstruction.
The results of the analyses using the layer moduli from Newcomb’s regression equations indicate that no overlays are required. However, it agrees with the mechanistic procedure for Seward Highway and recommends reconstruction. The Asphalt Institute procedure is confirmed as producing the thinnest pavement sections. Overlay thicknesses range from 0 to 3 in. (0 to 7.5 cm) and are typically lower than those determined with the mechanistic method by 2 to 3 in. (5 to 7.5 cm) or more.

Discussion of Results

From the preceding sections, some clear trends or conclusions may be drawn. Overall, use of the program BISDEF was not very successful. This may be due to the presence of frozen layers, the presence of thin surface layers, or a combination of both. ELSDEF appears to give more reasonable values. Still, there are some anomalies. In the case of Seward Highway, more sections should have been analyzed to obtain a better sample of the population. The base moduli seem to be on the low side, but the program seems to handle the presence of a frozen layer well. To ensure that the backcalculated moduli are accurate, particularly for thin surface layers (< 2 inches), cores should be obtained from the project sites and tested in the laboratory.

Newcomb’s equations for layer moduli are not recommended for Alaskan conditions. It should be noted that they were specifically developed for non-frozen pavements and probably should not have been included in this paper. Also, the presence of a frozen layer and a very thick base and subbase result in shallower deflection basins, and the outermost sensors should probably be extended further than 48 in. (122 cm). Figure 7 shows an example of a deflection basin. If equations were developed for deflections beyond this point, the layer moduli may be more reasonable.
Fernando et al.'s method did not always appear to yield reasonable results. For example, it was determined that an overlay is sufficient for Seward Highway even though the pavement fatigue life is used up. It generally appears that this procedure produces overlay thicknesses at least equal to or greater than those determined by the mechanistic method in the majority of cases. The presence of the frozen layers generally does not appear to distort the results from the regression equations. However, the conservative tone of the overlay designs should prove sufficient for Alaska to develop their own equations.

The results from the Asphalt Institute procedure confirms that this procedure may not always be appropriate for use in Alaska. The lack of consideration of surface distress or the presence of frozen layers is obvious in the results. The overlay thicknesses obtained are either significantly lower than the other procedures or indicate no need for an overlay.

Before concluding, the authors would like to comment on the present procedure of using center deflections to determine the critical section for analysis. Figure 8 illustrates both the overlay thicknesses (for the 50% reliability level) and center deflections at the same location for Sterling Highway. They show little, if any, coherent relationship between the center deflection (corrected to 70°F and normalized to a 9000 lb load) and the overlay thickness (from the mechanistic method). On the other hand, Figure 9 indicates a clearer relationship between the overlays and the tensile strains. This again emphasizes the point that center deflections should not be used as a criterion for selection of an analysis section.

As a final note, the lack of accurate data on the historical traffic EALs and the pavement layer thicknesses can be a major obstacle in the analyses.
Efforts are needed to improve the quality of traffic and thickness data. It cannot be emphasized strongly enough that an accurate knowledge of the traffic and layer thicknesses is very important. The thickness of the asphalt concrete layer, in particular, markedly affects the backcalculated results (13), and inaccurate traffic counts serve to disguise the actual remaining life of the pavement.
CONCLUSIONS AND RECOMMENDATIONS

Conclusions

Based on the findings of this study, the following conclusions appear warranted:

1. The present overlay design procedure used by the state of Alaska does not acknowledge the special problems that the climate presents in that region. It also does not utilize the concept of remaining life and tends to underdesign the pavement by specifying a thinner overlay than needed.

2. Four methods of overlay design were analyzed in this report. They include the Asphalt Institute procedure; Fernando et al.'s strain-deflection relationships developed in Pennsylvania; and a mechanistic method employing two backcalculation programs, BISDEF and ELSDEF and one set of regression equations developed in Washington State.

3. The tensile strain at the base of the asphalt concrete layer was the criterion used in determining the remaining life, as fatigue appeared to be the predominant failure mode. The fatigue equation developed by the Asphalt Institute was also used. However, other criteria such as the compressive subgrade strain and corresponding equations are available.

4. There were insufficient traffic and pavement structural data for the projects. This led to assumptions used in the analysis that may be inaccurate and, as a result, the overlay thicknesses could be misleading. However, the data are representative of what is
actually available for most design situations. Often, more accurate data are simply not available.

5. The BISDEF program did not always close; therefore, it was determined that the backcalculated moduli were not always reliable.

6. The ELSDEF program resulted in layer moduli which compared favorably with prior laboratory tests. However, additional work is needed to verify these backcalculated values.

7. The Asphalt Institute procedure appears inappropriate for use in Alaska in that the resulting overlay thicknesses are very thin. This method indicates that overlays are not required for several sections despite contrary results from the other methods.

8. Fernando et al.'s equations appear to perform reasonably well compared with the mechanistic method: if anything, they are more conservative. However, it appears that when a pavement is badly fatigued, the procedure can give misleading results. On the other hand, there probably should be more sections analyzed where the remaining life is close to zero before a definitive conclusion can be reached.

9. The basis for selection of the critical section should not be the center deflection alone because the overlay thicknesses obtained are not related to the magnitude of these deflections. Instead, tensile strain is a better indicator, and the procedure discussed in this report is recommended.

10. For the projects evaluated, an increase in the AASHTO reliability levels from 50% to 90% generally increases the overlay thickness by approximately 2 in.
Recommendations for Implementation

From the conclusions discussed above, the following are recommended for implementation:

1. The Asphalt Institute procedure should not be used for overlay design in Alaska.

2. The proposed mechanistic procedure using ELSDEF should be considered as a replacement for the Asphalt Institute procedure in the design of flexible overlays. For thin asphalt layers (< 2 inches), it is recommended that asphalt cores and aggregate samples be obtained and tested.

3. The tensile strain criterion and the Asphalt Institute fatigue relationships appear adequate at present.

4. For a simpler overlay design procedure, Fernando et al.'s equations may be used. However, the results are more conservative than those using the mechanistic procedure.

5. More accurate traffic data (EALs) are needed, particularly historical data. Efforts should also be concentrated on collecting traffic data for future use and making growth projections. It is recommended that Weigh-in-Motion and Automatic Vehicle Classification units be left in the field for several weeks at different times of the year to obtain a better representation of the traffic profile.

6. Similarly, accurate layer thicknesses are needed, as this strongly influences the backcalculated moduli. If construction records have not been kept, then cores should be taken, approximately at the rate of 5 per mile.
7. The 50% reliability level is recommended for design on roads with low volumes. However, for roads with significant traffic volumes or those which may expect a substantial increase in traffic, a 90% level may be more appropriate.
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LITERATURE REVIEW

TASK B
DEVELOP IMPROVED METHOD

PHASE II
TESTING

TASK C
COMPUTERIZE METHOD

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TASK E
FIELD MANUAL

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Figure 8. Comparison Between Overlay Thickness and Center Deflections for Parks Highway.
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Table 1. Problems with the Current Overlay Design Method

1. Current procedures do not always show a need for overlays despite high traffic volumes.

2. Pavement surface condition is not considered, especially cracking.

3. Effects of frozen subgrade and base are not considered.

4. Remaining life is not considered.

5. Pavement layer dimensions are not used.

6. Use of new additives in asphalt concrete is not considered.

7. Peak center deflection is not a good indicator of distress.
### Table 2. Summary of Parameters for Project Sites

<table>
<thead>
<tr>
<th>Project Location</th>
<th>Pavement Structure</th>
<th>Traffic To Date</th>
<th>20 yr. EAL</th>
<th>Pavement Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>STERLING HIGHWAY</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MP 117-130</td>
<td>AC - 1.5 in. Base - 4.0 Subbase - 6.0 Borrow - 24.0</td>
<td>130,000&lt;sup&gt;2&lt;/sup&gt;</td>
<td>1,800,000</td>
<td>Good</td>
</tr>
<tr>
<td>MP 130-157</td>
<td>Same as above</td>
<td>130,000&lt;sup&gt;2&lt;/sup&gt;</td>
<td>1,800,000</td>
<td>Extensive alligator cracking and rutting.</td>
</tr>
<tr>
<td>MP 157-162</td>
<td>AC - 1.5 Base - 6.0 Borrow - 0 to 36.0</td>
<td>130,000&lt;sup&gt;2&lt;/sup&gt;</td>
<td>1,800,000</td>
<td>No fatigue cracking.</td>
</tr>
<tr>
<td>MP 162-166</td>
<td>Same as above</td>
<td>Unknown</td>
<td>2,770,000</td>
<td>5-26% cracking.</td>
</tr>
<tr>
<td>MP 166-171</td>
<td>Same as above</td>
<td>Unknown</td>
<td>7,980,000</td>
<td>100% cracked.</td>
</tr>
<tr>
<td><strong>SEWARD HIGHWAY</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>36th to 4th Ave</td>
<td>AC - 2 to 5.25 Base - 6.0 Subbase - 18.0</td>
<td>4,400,000</td>
<td>5,083,000</td>
<td>Extensive cracking and rutting. (10 years)</td>
</tr>
<tr>
<td><strong>PARKS HIGHWAY</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>North Section</td>
<td>AC - 2.0 Base - 4.0 to 6.0 Subbase - 6.0 to 12.0 Borrow - 24.0 to 36.0</td>
<td>78,723&lt;sup&gt;2&lt;/sup&gt;</td>
<td>390,521</td>
<td>Severe rutting and alligator cracking.</td>
</tr>
<tr>
<td>South Section</td>
<td>Same as above</td>
<td>76,069&lt;sup&gt;2&lt;/sup&gt;</td>
<td>345,526</td>
<td>Severe rutting and alligator cracking.</td>
</tr>
</tbody>
</table>

<sup>1</sup>For Parks Highway, these are assumed dimensions.
<sup>2</sup>Traffic data are assumed.
1 in. = 2.54 cm
Table 3. Comparison of Layer Moduli

<table>
<thead>
<tr>
<th>Location</th>
<th>BISDEF&lt;sup&gt;2&lt;/sup&gt;</th>
<th>ELSDEF</th>
<th>NEWCOMB&lt;sup&gt;3&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>E1</td>
<td>E2</td>
<td>E3</td>
</tr>
<tr>
<td>Sterling Highway</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MP 117.5</td>
<td>1688</td>
<td>93</td>
<td>125</td>
</tr>
<tr>
<td>118.0</td>
<td>1719</td>
<td>36</td>
<td>103</td>
</tr>
<tr>
<td>118.5</td>
<td>657</td>
<td>51</td>
<td>93</td>
</tr>
<tr>
<td>119.0</td>
<td>2000</td>
<td>34</td>
<td>346</td>
</tr>
<tr>
<td>119.5</td>
<td>398</td>
<td>42</td>
<td>189</td>
</tr>
<tr>
<td>Seward Highway</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TH 1</td>
<td>2000</td>
<td>46</td>
<td>29</td>
</tr>
<tr>
<td>TH 2</td>
<td>2000</td>
<td>52</td>
<td>16</td>
</tr>
<tr>
<td>TH 3</td>
<td>2000</td>
<td>16</td>
<td>47</td>
</tr>
<tr>
<td>TH 4</td>
<td>1488</td>
<td>37</td>
<td>14</td>
</tr>
<tr>
<td>TH 45</td>
<td>570</td>
<td>26</td>
<td>12</td>
</tr>
<tr>
<td>TH 34</td>
<td>2000</td>
<td>32</td>
<td>14</td>
</tr>
<tr>
<td>TH 35</td>
<td>1522</td>
<td>42</td>
<td>24</td>
</tr>
<tr>
<td>Parks Highway</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CDS 206</td>
<td>2000</td>
<td>18</td>
<td>1000</td>
</tr>
<tr>
<td>206.2</td>
<td>286</td>
<td>25</td>
<td>1000</td>
</tr>
<tr>
<td>206.4</td>
<td>1035</td>
<td>24</td>
<td>171</td>
</tr>
<tr>
<td>206.6</td>
<td>1194</td>
<td>21</td>
<td>552</td>
</tr>
<tr>
<td>206.8</td>
<td>554</td>
<td>25</td>
<td>89</td>
</tr>
</tbody>
</table>

<sup>1</sup>All moduli values are in ksi (1 ksi = 6.89 MPa)
<sup>2</sup>BISDEF values did not converge due to difficulties within the program. The results included here are not reliable, and are only for general comparison purposes. Most of the moduli are at the extremes of the ranges specified.
<sup>3</sup>Newcomb’s regression equations specifically do not consider the effects of a frozen base and subbase, whereas these sections are partially frozen.
<sup>4</sup>Moduli values were not calculated due to a zero deflection reading.
Table 4. Comparison of Overlay Thicknesses, Inches

<table>
<thead>
<tr>
<th>Location</th>
<th>The Asphalt Institute</th>
<th>Newcomb</th>
<th>Fernando</th>
<th>Mechanistic*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>50%</td>
</tr>
<tr>
<td>Sterling Highway</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MP 117.5</td>
<td>The overlay</td>
<td>0</td>
<td>2.5</td>
<td>0.0</td>
</tr>
<tr>
<td>118.0</td>
<td>thickness</td>
<td>0</td>
<td>5.0</td>
<td>4.0</td>
</tr>
<tr>
<td>118.5</td>
<td>required</td>
<td>0</td>
<td>5.5</td>
<td>5.0</td>
</tr>
<tr>
<td>119.0</td>
<td>is 2.0 in.</td>
<td>**</td>
<td>5.5</td>
<td>**</td>
</tr>
<tr>
<td>119.5</td>
<td></td>
<td>0</td>
<td>5.5</td>
<td>5.5</td>
</tr>
<tr>
<td>Seward Highway</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TH 1</td>
<td>RRD = 26</td>
<td>0</td>
<td>3.5</td>
<td></td>
</tr>
<tr>
<td>TH 2</td>
<td>Overlay is</td>
<td>6</td>
<td>2.0</td>
<td></td>
</tr>
<tr>
<td>TH 3</td>
<td>2 in. (50%)</td>
<td>Recon.</td>
<td>4.0</td>
<td></td>
</tr>
<tr>
<td>TH 4</td>
<td>&amp; 3 in. (90%)</td>
<td>Recon.</td>
<td>5.0</td>
<td></td>
</tr>
<tr>
<td>TH 45</td>
<td></td>
<td>Recon.</td>
<td>6.0</td>
<td></td>
</tr>
<tr>
<td>TH 34</td>
<td></td>
<td>Recon.</td>
<td>4.5</td>
<td></td>
</tr>
<tr>
<td>TH 35</td>
<td></td>
<td>6</td>
<td>4.0</td>
<td></td>
</tr>
<tr>
<td>Parks Highway</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CDS 206</td>
<td>RRD = 26</td>
<td>**</td>
<td>6.5</td>
<td>4.0</td>
</tr>
<tr>
<td>206.2</td>
<td>Overlay</td>
<td>**</td>
<td>6.5</td>
<td>4.5</td>
</tr>
<tr>
<td>206.4</td>
<td>is 0.</td>
<td>**</td>
<td>6.5</td>
<td>3.5</td>
</tr>
<tr>
<td>206.6</td>
<td></td>
<td>**</td>
<td>7.0</td>
<td>4.5</td>
</tr>
<tr>
<td>206.8</td>
<td></td>
<td>**</td>
<td>5.0</td>
<td>7.0</td>
</tr>
</tbody>
</table>

NB: The minimum overlay thickness is 1 inch, and values are rounded up to the nearest 0.5 inch. (1 inch = 2.54 cm)
*These are the 50% and 90% reliability levels as defined by AASHTO.
**Overlay thicknesses could not be computed.