

Alaska Flexible Pavement Design Manual



Alaska Department of Transportation
and Public Facilities

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Preface

This manual guides the designer through a comprehensive computerized method for the design of Alaskan pavement structures. The Alaska Flexible Pavement Design method version 2 (AKFPD-2), a computer program, replaces the original Alaska Flexible Pavement Design tool previously used by designers. While the process remains essentially unchanged, this manual reflects the changes to the software to operate on Windows 7.0 and above and to include the Life Cycle Cost procedure described in Chapter 8.

The purpose of this manual is to help the pavement designer produce cost effective pavement designs.

The manual

- Contains general engineering background to help the designer understand basic principles of both the Excess Fines and Mechanistic-Empirical design process and how the AKFPD-2 software works.
- Describes all functional capabilities of the AKFPD-2 software and provides step-by-step design examples.
- Provides specific guidance on input variables and other decision criteria required to run the AKFPD software.
- Describes the new Life Cycle Cost procedures which help the designer select the most cost-effective design.
- Presents some of the broader aspects and policies bearing on the pavement design process.

The Alaska Flexible Pavement Design is the accepted pavement structure design procedure for the Alaska Department of Transportation and Public Facilities (DOT&PF), as outlined in Chief Engineer's 2020 Directive (<http://www.dot.state.ak.us/stwddes/dcspubs/directives.shtml>). As such, the methods and policies provided in this manual should be followed unless appropriate approvals are obtained as provided in Chapter 2 of this manual.

1 Introduction

- 1.1 Pavement Design Focus
- 1.2 Background
- 1.3 The “Pavement Structure”
- 1.4 Excess Fines Design Method
- 1.5 Mechanistic Design Method
- 1.6 Pavement Drainage

1.1 Pavement Design Focus

The pavement structural design process consists of determining whether a proposed assemblage of layered materials will perform as expected when subjected to a specified number and intensity of vehicle load cycles. This process is often termed layer thickness design because the designer starts by defining each layer of a pavement structure using known material properties and assumed layer thicknesses. The designer then calculates whether the pavement structure will withstand the required vehicle loadings (design loadings).

This manual addresses the design of flexible pavements, i.e., those having an asphalt concrete surface. These methods cannot be used for designing pavement structures surfaced with rigid Portland cement concrete. In the simplest designs, asphalt concrete will serve only as the topmost layer of the pavement structure. In more complex designs, asphalt concrete will often be used as an overlay on existing pavements and/or in the form of an asphalt-treated base course.

Two methods for pavement structural design are presented here. The Excess Fines method and the Mechanistic-Empirical method. The Excess Fines method is useful for designing low volume roadways while the mechanistic method is appropriate for roadways with higher traffic volumes.

This manual and computer program combination will not turn the neophyte engineer into a pavement structural design expert. Other engineering skills are needed, many of which are covered minimally or not at all here. The expert pavement structure designer will have accumulated expertise in economic analysis; construction methods; materials science, including laboratory test procedures and test-data interpretation; hydrology; geological and engineering evaluation of aggregate sources; and asphalt concrete and asphalt cement technology.

The designer must realize that poor foundation conditions and other geotechnical or drainage problems profoundly affect the performance of pavement structure, regardless of the quality of the pavement design. The designer must therefore actively seek help from technical specialists. Design measures that tackle drainage problems, foundation problems, slope stability, and erosion usually require consultation with the Regional subject matter experts. Regional Materials personnel will also help with designs that must address special Alaska problems such as ice-rich permafrost and muskeg with the associated use of special materials such as insulation and geotextiles.

1.2 Background

The *Alaska Highway Preconstruction Manual*, Chapter 11⁽¹⁾, Section 1180 (Pavement Design) is found at:

<http://www.dot.state.ak.us/stwddes/dcsprecon/assets/pdf/preconhwy/chapters/chapter11.pdf>

This chapter refers designers to AKFPD as the Department’s pavement structure design procedure. AKFPD has its root base on the Excess Fines Method developed in the 1980’s and adopted by DOT&PF in 1983 and the Mechanistic Design method first proposed in the late 1980’s. Shortly, thereafter, the mechanistic design method became the method of choice for DOT&PF.

Although it is now considered to have limited usefulness, the excess fines method’s easily understood concepts and simple computations made it the favorite tool for designing highway pavements on low volume roads. Limitations are inherent because the excess fines method was empirically derived using only highway data.

Therefore, the excess fines method cannot be used for any pavement structures that will be subjected to anything other than normal highway vehicle loadings. Because of its empirical origin, the excess fines method cannot be applied to designing pavement structures that will contain unusual materials. This includes any materials other than the standard road-building types like those that characterized the original database.

On the other hand, the mechanistic method easily handles a variety of material types and vehicle load configurations. DOT&PF officially recognizes mechanistic design as not only the more comprehensively useful tool but as the more defensibly “correct” of the two analytical methods. The term *mechanistic design* is a generic one, implying that the pavement structure is objectively analyzed as a mechanical system of elastic layers. Be aware that the mechanistic pavement design process could be done in a variety of ways, only one of which has been developed for use in Alaska.

Chapters 3 and 4 cover, in detail, the basics of the excess fines method and mechanistic method, respectively.

Regardless of which design method you use, economics will remain a chief concern. As in any engineering discipline, the design engineer must design a pavement structure that cost-effectively meets the intended need. To do this, the designer must consider life-cycle costs. Life-cycle costs include all costs associated with constructing, maintaining, and rehabilitating the pavement structure through a defined period of service (the analysis period). Chapter 8 provides a complete description of the life-cycle cost analysis procedures preferred by DOT&PF.

1.3 The “Pavement Structure”

Vehicles are not supported by the hot asphalt concrete surfacing material alone. Much of the support comes from the bound layer (asphalt cemented) and unbound material below. This brings up a few questions: (1) What total thickness of material supports the load? (2) What quality of material is required within this thickness? (3) What happens if poor quality materials are used within this thickness?

The asphalt concrete layer is the top layer or wearing course of a pavement structure. Pavement structure is an important concept, defined for our purpose as the total thickness of material that “feels” significant compression stresses (and therefore strain) under the design vehicle’s wheel loading, i.e., the material that must support that load. Material at the surface (asphalt concrete surfacing) and material close to the surface (base course) will be subjected to relatively high compression stresses and therefore high levels of strain. Stresses and strains due to vehicle loadings are distributed laterally within the pavement structure and attenuate quickly with depth. The influence of a standard vehicle loading is attenuated to such a degree that at about 10 feet below the surface, stresses and strains are about zero. A good discussion of stress distribution through uniform and layered soil structures can be found in almost any soils engineering textbook.

The empirically derived rule-of-thumb adopted for use in Alaska is that normal highway loads are carried by the hot asphalt concrete plus an additional 3.5 feet of layered structural materials. Alaska’s excess fines design method specifically defines the pavement structure based on this rule-of-thumb.

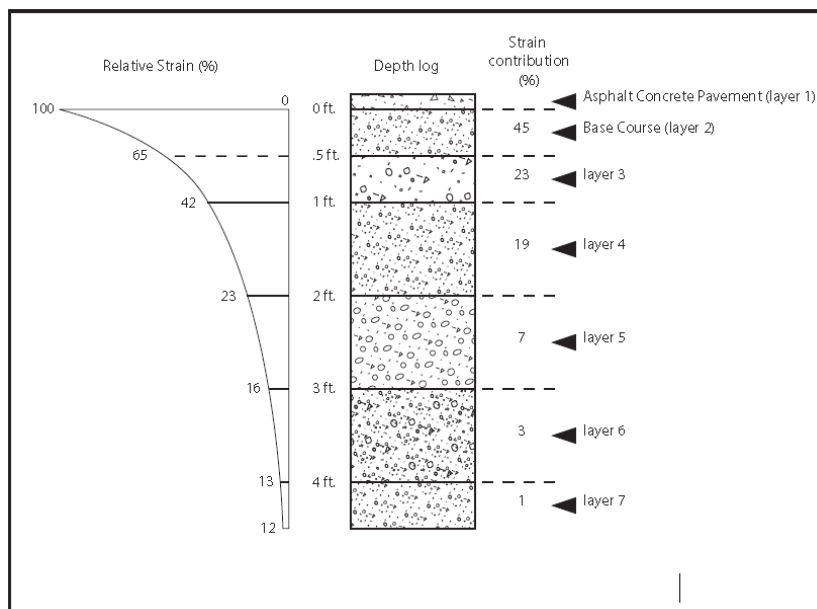


Figure 1-1 Strain Distribution with Depth

The excess fines method therefore requires that all material to a depth of 3.5 feet below the bottom of the asphalt concrete layer be accounted for in every pavement design analysis.

Figure 1-1 illustrates how strains are distributed within a typical pavement structure. The required load-carrying capacity of each layer is directly related to the strain contribution of that layer. Layers having the largest strain contributions must therefore be of highest quality (stiffest) to minimize pavement bending and resultant damage. Both the Excess Fines design method and the Mechanistic Design method recognize this, but they handle the integration of this relationship in different ways.

1.4 Excess Fines Design Method

Alaska research strongly suggests that the quality of unbound aggregate materials within the pavement structure is mostly controlled by the percentage of fines (weight percent of particles finer than the #200 sieve, also known as P_{200} , minus 75 micron, or $P_{0.075 \text{ mm}}$). The P_{200} content correlates with the aggregate's ability to support vehicular load, especially during the springtime thaw period. The general relationship is low P_{200} content = good support and high P_{200} content = poor support. The P_{200} content matters less as depth below the asphalt concrete pavement surface increases. At a depth greater than 3.5 feet, a high P_{200} content is acceptable (assuming standard highway-type loadings).

The Excess Fines design method combines the relationship of P_{200} to performance at depth and the strain distribution at depth to estimate the surface deflection under a standard 18kip single axle loading. Using the estimated deflection, the fatigue life of the asphalt concrete surface can be estimated. This procedure works well as long as the underlying assumptions of the procedure are met. The Excess Fines methodology is discussed in more detail in Chapter 3.

1.5 Mechanistic Design Method

The pavement structure is made of up elastic layers with a known thickness, stiffness or modulus and Poisons ratio. Knowing these three parameters, it is possible to compute the stress, strain and deflection at any point below and at any offset from vertical for any surface loading. Using the stresses and/or strain at strategic points in the pavement structure in conjunction with empirically derived equations, the number of cycles to failure can be estimated for each layer. If the estimated number of cycles to failure for each layer are greater than the number of cycles imposed on that layer, then the pavement structure can be expected to perform as designed. By varying the material and the thickness of those materials, the designer has an infinite number of possibilities to choose from. The goal is to select the most cost effective combination. It is important to realize that the most cost effective design may not be the design with the lowest first cost.

As the designer gains experience, finding the best combination becomes easier. The mechanistic procedures are fully described in Chapter 4. The AKFPD software makes complex computations simple. With a little practice, the designer will be able to find a cost effective solution in short order.

1.6 Pavement Drainage

It is often said that the first dollar spent in the construction of a roadway should be spent on drainage. Water has a greater impact on the performance than the materials selected or their thickness. Too much water reduces soil strength, increases the thaw rate of permafrost under the roadway and may increase the chances of spring thaw weakening. Even asphalt pavement itself will be damaged by standing water.

Avoid premature pavement failures by providing proper drainage. Refer to the *Alaska Highway Preconstruction Manual*, Chapter 11, Section 1120.5. Drainage, for basic guidelines:

<http://www.dot.state.ak.us/stwddes/dcsprecon/assets/pdf/preconhwy/chapters/chapter11.pdf>

DOT&PF's *Highway Drainage Manual*⁽²⁾ is also available at a

http://dot.alaska.gov/stwddes/desbridge/pop_hwydrnman.shtml

Pavement designs must provide for surface and subsurface drainage of moisture away from the pavement surface and supporting layers.

For driver safety, the less water on the road surface the better. The pavement design engineer must be able to identify any points on the pavement where standing water or sheet flow is enough to cause hydroplaning and skidding. This requires strategies for eliminating standing water and minimizing the film thickness of moving water on the pavement surface. Typical strategies include increasing cross slope, adding drainage inlets, adding culverts, increasing ditch depth, or grooving the pavement. Paved shoulders help move water away from the pavement structure.

For structural reasons, the less free water in the pavement structure the better. Drainage ditches must be large enough to store the annual snow accumulation as well as move the water away when the snow melts. Ditches must keep water moving away from the pavement structure during rainstorms. This requires careful attention to ditch grades and cross drainage, especially in areas of sag curves.

Additional pavement drainage design references and information can be found at:

- Subsurface Water and Drainage Requirements (Section 7.2 of “Geotechnical Aspects of Pavements” Reference Manual FHWA-NHI-05-037)⁽³⁾:
<https://www.fhwa.dot.gov/engineering/geotech/pubs/05037/05037.pdf>
- Improved Surface Drainage of Pavements, NCHRP Web Document 16⁽⁴⁾
<https://www.nap.edu/read/6357/chapter/1>
- Pavement Subsurface Drainage Systems, NCHRP Synthesis 239⁽⁵⁾
http://onlinepubs.trb.org/Onlinepubs/nchrp/nchrp_syn_239.pdf
- Drainage Requirements in Pavements⁽⁶⁾
http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part3_Chapter1_Subdrainage.pdf
http://onlinepubs.trb.org/onlinepubs/archive/mepdg/2appendices_TT.pdf
<https://www.fhwa.dot.gov/pavement/pubs/010942.pdf>
- Effects of Subsurface Drainage on Pavement Performance, NCHRP Report 583⁽⁷⁾
http://onlinepubs.trb.org/onlinepubs/nchrp/nchrp_rpt_583.pdf

2 Policies and Required Considerations in the Pavement Design Process

- 2.1 General Policy (GP) Statements
- 2.2 Policy on Selecting the Correct AKFPD Design Procedure
- 2.3 Policy on Base Course Stabilization
- 2.4 Policy on Life Cycle Cost Analysis

2.1 General Policy (GP) Statements

- GP-1* Prepare a pavement design analysis for all highway projects requiring pavement construction, reconstruction, rehabilitation or resurfacing on arterials and interstates. (See Section 2.2.4 for non-highway designs)
- GP-2* The pavement design method presented in this manual is the standard for flexible highway pavement designs for the Alaska DOT&PF.
- GP-3* The regional preconstruction engineer is responsible for the final pavement design.
- GP-4* If cost-effective, design pavement structures such that no seasonal load restrictions are needed.

GP-5, GP-6 and GP-7 refer to construction, reconstruction, and rehabilitation projects.

- GP-5* For projects with design average annual daily traffic (AADTs) volumes $\geq 10,000$ without curb and gutter:
 - Use Alaska Renewable Pavement (see Section 7.4.3).
 - Use a 15-year design life for both the fatigue failure criterion and the functional failure criterion (see Section 4.3.2).
- GP-6* For projects with AADTs $< 10,000$ without curb and gutter and for projects with AADTs $< 5,000$ with curb and gutter:
 - Use no less than one layer of binder course, asphalt-treated base, or other stabilized base (see Section 7.4).
 - Use a 15-year design life for both the fatigue failure criterion and the functional failure criterion (see Section 4.3.2).
- GP-7* For projects with AADTs $> 5,000$ with curb and gutter:
 - Use Alaska Renewable Pavement (see Section 7.4.3).
 - Use a 30-year design life for the fatigue failure, (TAI equation, see Section 4.3.2).
 - Use a 15-year analysis period for the functional failure analysis (Per Ullidtz equation, see Section 4.3.2).
- GP-8* The minimum design life of resurfacing projects using the Alaska Renewable Pavement will be not less than 15 years for both fatigue life and functional failure.
- GP-9* Surface treatments may be used if any of the following conditions are met:
 - AADT $< 1,000$.

- Life-cycle cost analysis supports their use.
- Unstable foundations underlie more than 60% of the project.
- Approved by the regional preconstruction engineer.

GP-10 On arterials and interstates, use 2.0 inches as the minimum thickness of asphalt concrete for new pavement designs or pavement designs that involve complete replacement of the old asphalt concrete layer.

GP-11 Use 1.5 inches as the minimum thickness of new asphalt concrete overlay placed on an existing layer of asphalt concrete, or two times the maximum aggregate size, whichever is greater.

GP-12 Designs utilizing the AKFPD software will be performed by personnel (DOT&PF staff or consultant) trained in its use.

GP-13 In case of reconstruction or resurfacing of a paved roadway, consider recycling or reusing the existing asphalt concrete material in the new structure.

GP-14 Use hard aggregate in the wearing surface of high-volume roadways (AADT per lane in the construction year $\geq 5,000$), exhibiting studded-tire wear. Hard aggregate is defined as an aggregate with a Nordic Abrasion Value (ATM 312) of 8.0% or less.

Hard aggregate may also be used in the wearing course when the “AADT per lane” is lower than 5,000, if there is a history of pavement rutting due to studded tire wear or when repaving an existing wearing course that incorporates hard aggregate.

Hard aggregate is not required for: temporary paving, or if it is expected to be replaced within five years, or where the roadway is underlain with unstable foundation, or where it can be demonstrated that studded tire abrasion is not a significant cause of rutting.

2.2 Policy on Selecting the Correct AKFPD Design Procedure

2.2.1 For Designing New Highway Pavements with ESALs < 1.0 Million

The excess fines method **may be used** for designing flexible highway pavement structures if:

1. The flexible surfacing material is composed of a standard form of asphalt concrete (no inclusions of unusual aggregate types or modified asphalt cements), and
2. The P_{200} content of all non-surfacing materials within the pavement structure falls within limits allowable by the excess fines design method.

The mechanistic method **may be used** for design work or for checking excess fines designs if the project’s available materials meet criteria 1 and 2 listed above.

The mechanistic method **must be used** if the project’s available materials do not meet criteria 1 and 2 listed above, or if the pavement structure incorporates one or more stabilized base course layers.

2.2.2 For Designing New Highway Pavements with ESALs > 1.0 Million

Use the mechanistic method.

2.2.3 For Designing Overlays of Existing Highway Pavements

Use the mechanistic method.

1. Do not overlay existing pavements if more than 80% of fatigue life of the existing pavement is exhausted (the AKFPD program determines this mechanistically based on historical traffic).
2. Do not overlay extensively cracked pavements, typically 20% or more of the surface cracked. Assume that all cracks in the existing pavement will reappear in the overlay within two years after the overlay is placed.

2.2.4 For Non-Highway Pavement Designs

Either the excess fines method, mechanistic method or other method approved by the regional preconstruction engineer may be used for designing flexible non-highway pavement structures, regardless of design vehicle type and/or available materials. These types of pavement structures can include asphalt sidewalks, paths, and parking/staging areas. For aviation pavement design on federally funded projects, use FAA's latest version of FAARFIELD software and its accompanying Advisory Circular 150/5320-6 series.⁽⁸⁾

2.3 Policy on Base Course Stabilization

It is the Department's policy to use stabilized bases on roadway construction, reconstruction, and rehabilitation projects where cost-effective.

In developing flexible pavement designs incorporating stabilized bases, refer to policies GP5, 6 and 7 in Section 2.1. In addition, use the following:

1. *Alaska's Soil Stabilization Manual, 2014 Update*, Report No. 60392⁽⁹⁾, and
2. The definition of stabilized layers as found in Section 7.4 of this manual.

Exceptions to this policy are as follows:

Projects exempted in writing by the regional preconstruction engineer. Rationale for an exemption may include:

- Projects with a low AADT.
- Areas underlain by unstable foundations such as ice-rich permafrost, where settlement results in frequent maintenance.
- Projects for which a stabilized base will not provide a cost-effective improvement in the pavement performance, reduced maintenance, or reduced future rehabilitation costs through a comprehensive life-cycle cost analysis. The analysis period of the life-cycle cost analysis shall be 35 years.
- Projects where matching existing, adjacent pavement, such as adding passing or turning lanes, pullouts, or short stretches.
- Roadways designed on behalf of agencies other than DOT&PF.

2.4 Policy on Life-Cycle Cost Analysis

Life-cycle cost analysis (LCCA) is a method to compare costs of alternative design over time. Concepts of engineering economics are used to calculate the net present value of costs for each of the alternatives being considered, including not only first cost but also any preservation, rehabilitation, and reconstruction costs over

the analysis period. Accurate LCCA requires a good understanding of all significant cost differences between the alternatives being evaluated.

LCCA should be considered when evaluating alternative pavement designs, especially on exceptionally large projects or projects with significantly different pavement alternatives, in which the incremental costs between alternatives can be accurately forecast. LCCA of pavement design alternatives may be required on certain projects, as directed by the regional preconstruction engineer.

Many factors impact the selection of a design strategy, including safety, local needs, available materials, environmental concerns, and appearance. Economic considerations are but one of the factors in such a decision. The method outlined in Chapter 8 provides a rational, straightforward means for analyzing the initial and future costs of pavement construction. Some social, safety or political issues may not be quantifiable according to the standard economic analysis concepts presented in the LCCA chapter. Such issues may nevertheless weigh heavily – even critically – on selecting a pavement design alternative.

3 Excess Fines Design

- 3.1. Introduction
- 3.2. Summary of Excess Fines Design
- 3.3. Principal Concepts
- 3.4. Stepping Through the Design Process: An Example
- 3.5. Excess Fines Design Using the AKFPD Computer Program

3.1 Introduction

This chapter provides a complete discussion of the excess fines design method. The AKFPD excess fines method is approved only for the design of some new, highway pavements. Chapter 2 provides detailed information concerning appropriate applications for this design method.

3.2 Summary of Excess Fines Design

Research leading to development of the excess fines method started in 1976 with the study of 120 asphalt concrete paved road sections throughout Alaska. Results of this research effort are described in reports published between 1980 and 1983.⁽¹⁰⁻¹¹⁾ An empirically derived design method was generated from this research and officially adopted for use by the DOT&PF in 1982.⁽¹²⁾ The Excess Fines Module of AKFPD provides a simple and rapid implementation of the Excess Fines Design procedures. While the designer can simply fill in the data fields and successfully create an acceptable design, it is important that the designer understand the concepts behind the procedure and the limitations of the procedure. Since the Excess Fines method was empirically derived, it is imperative that the designer stay within the bounds of the data from which the procedure was derived. While the software limits the more obvious inputs, it may not eliminate all potential violations of these bounds.

3.3 Principal Concepts

Alaska's excess fines design method relies on the following two empirically derived concepts:

1. An empirically definable relationship exists between pavement surface deflection at the center of a standard wheel load and the service life of the pavement in terms of the number passes of that wheel load. In the case of highway pavement design, the design load has long been standardized in the form of the 18,000 pounds dual-wheeled axle (110 psi tire pressure). This standard axle load is also known as the Equivalent Single Axle Load (ESAL).

$$\text{Service Life} = f(\text{deflection})$$

2. An empirically definable relationship exists between pavement surface deflection at the center of a standard wheel load and the amount of P_{200} material contained within individual layers of the pavement structure.

$$\text{Deflection} = f(\text{amount of } P_{200} \text{ in aggregate layers})$$

Simple calculations predict deflection at the surface of the pavement structure based on the amount of P_{200} in each granular layer of the structure. Another calculation determines the thickness of asphalt concrete pavement needed to reduce the predicted deflection level to accommodate the design number of ESALs without pavement failure.

3.3.1 Relationship Between P_{200} Content and Pavement Surface Deflection (A Measure of Pavement Structural Strength)

Based on Alaska DOT&PF research cited previously,⁽¹³⁾ increasing the P_{200} increases the potential of springtime thaw weakening which results in higher deflections. Higher deflections translate to fewer loadings to failure. Consequently, the Excess Fines method uses the P_{200} to estimate the deflections during spring thaw.

As wintertime cooling continues, a freezing interface or “freezing front” moves downward through the granular layers of the pavement structure. As the freezing front advances downward, soil moisture continually migrates upward toward the interface between frozen and unfrozen material. The upward transport of moisture during soil freezing satisfies energy balance requirements within the soil mass. After soil moisture reaches the freezing front, it freezes and becomes incorporated as part of the newly frozen soil mass. The process of freezing front progression, upward moisture migration, and freezing continues throughout the winter if appropriate temperature gradients exist in the soil. DOT&PF research correlated the amount of frozen moisture accumulating within a granular layer to the amount of P_{200} contained in that layer, establishing a relationship between P_{200} and frozen moisture content. Figure 3-1 illustrates the condition of the pavement structure during the freezing process.

As common sense suggests, frozen moisture is of little concern. On the other hand, the combination of springtime thawing and high moisture content in pavement structural layers is very much a problem. DOT&PF research derived a useful functional relationship between thaw weakening of the pavement structure and the amount of P_{200} contained in various layers of the pavement structure. The indicator of thaw weakening employed during the research project was measurement of pavement surface deflection in response to a standard test load, thus establishing a relationship between P_{200} and deflection. The standard test load used during research was the one side of a standard ESAL. The dual wheel set weighed 9,000 pounds total with tire pressures set at 80 psi. Figure 3-2 shows the pavement structure weakened by thaw.

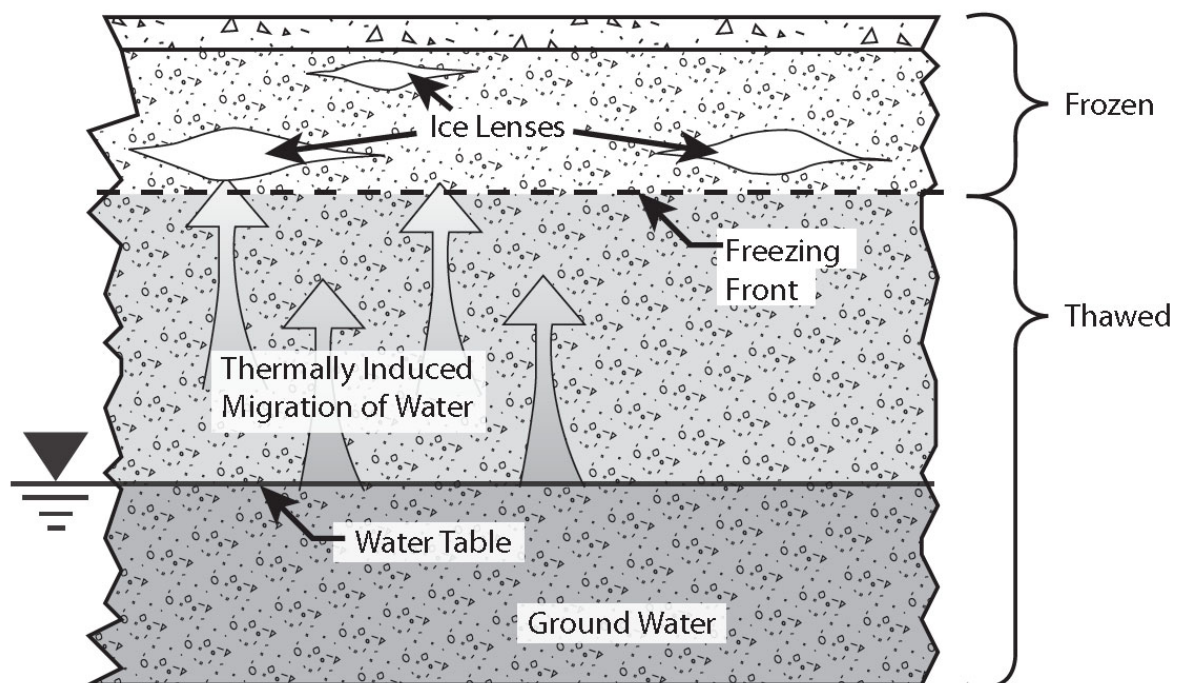


Figure 3-1 Progression of Freezing Front and Ice Formation

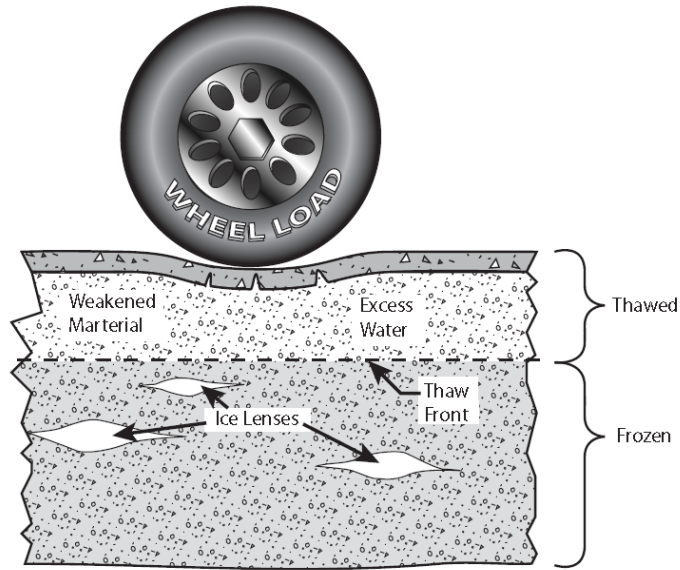


Figure 3-2 Soft Pavement Structure During Thawing

DOT&PF research refined the relationship between P_{200} and deflection by creating a function that attaches increasing importance to high P_{200} contents in upper layers of the pavement structure, which relates to the attenuating distribution of live-load stress with depth. Common sense and research provide the same conclusion: materials deep in the pavement structure “feel” almost none of the vehicle load and are therefore of less concern. Figure 3-3 illustrates the attenuation of strains (also stresses) with depth in the pavement structure.

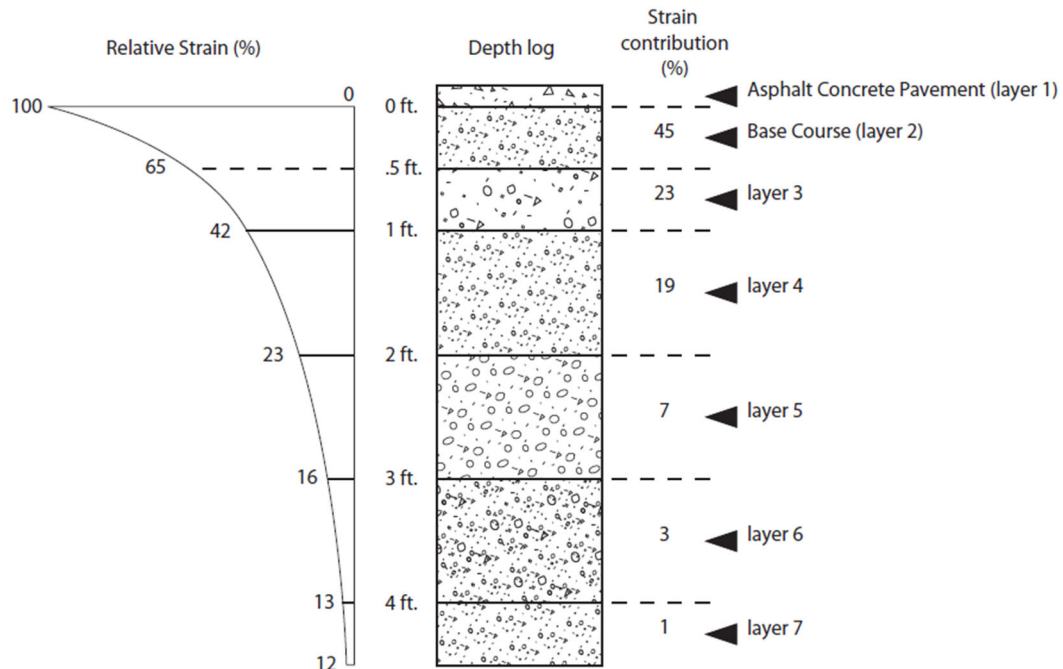


Figure 3-3 Attenuation of Vehicle Load with Depth

3.3.2 Relationship Between Pavement Surface Deflection and Pavement Service Life

As explained above, DOT&PF research derived a functional relationship between aggregate layer P_{200} contents and deflection. However, an additional design step is necessary. The design process requires a relationship between calculated deflection, design ESALs, and required asphalt concrete layer thickness. For this, DOT&PF modified and adopted The Asphalt Institute's TAI procedure from their 1977 MS-17 publication.⁽¹⁴⁾

3.3.3 Calculations Used in the Excess Fines Method

Separate each material layer into analysis layers based on the P_{200} content, with a maximum of 1-inch layer thickness. Analyze the pavement structure to a depth of 40 inches below the bottom of the asphalt concrete layer. This can include in-place material in the foundation.

Determine the critical fines content (P_{CR}) for each layer, using Figure 3-4. Enter the vertical axis with the depth from the bottom of the asphalt concrete layer to the top of each aggregate layer and move horizontally to intersect the P_{CR} line. Read the value of P_{CR} for that layer on the horizontal axis.

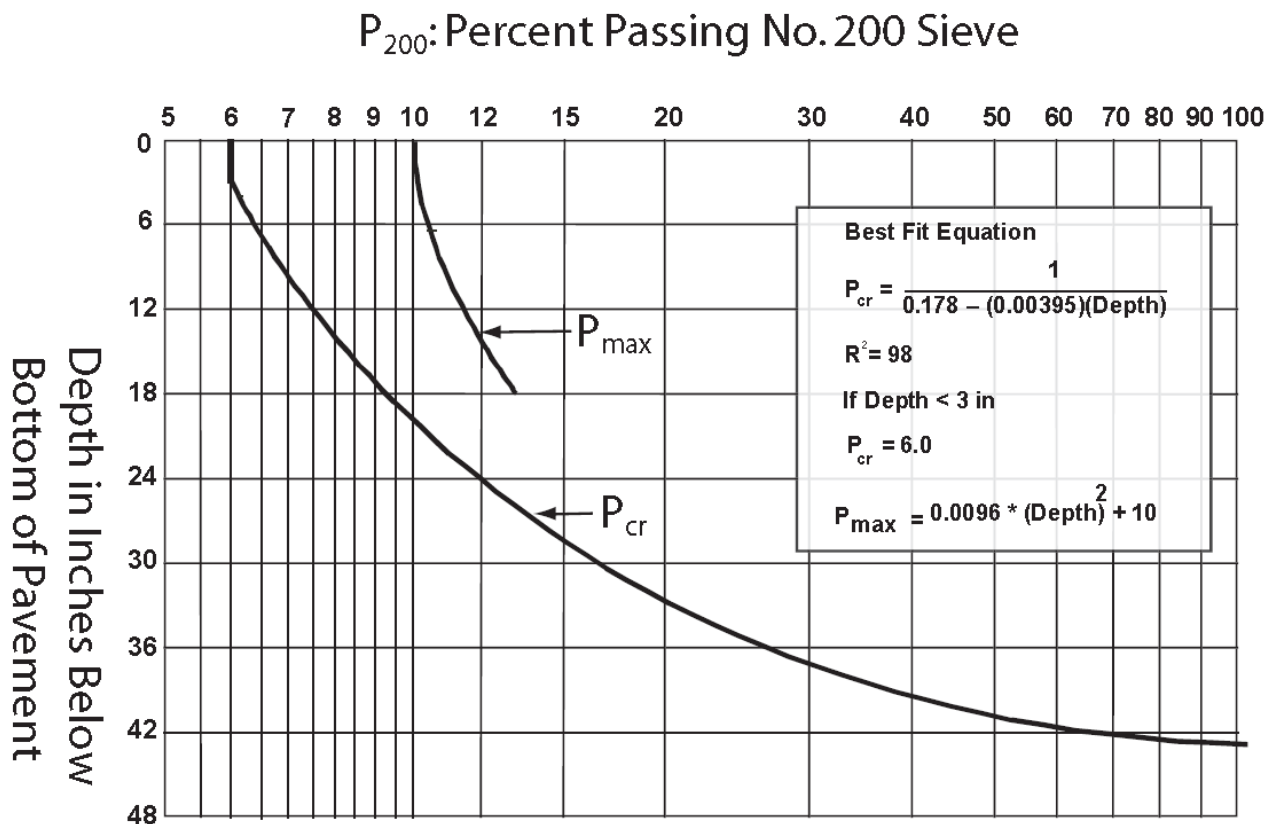


Figure 3-4 Critical and Maximum Fines Versus Depth

In no case shall the P_{200} exceed P_{max} for an analysis layer (i) with its top between 0 inches and 18 inches beneath the asphalt concrete layer (See P_{max} line on Figure 3-4).

Some materials degrade when crushed and handled. It is important that the P_{200} estimated for the base layer is the anticipated P_{200} of the crushed material after it has been placed and compacted. If the degradation value (Alaska

Test Method, ATM 313) of the material is less than 45, check with the regional materials engineer for guidance.

Calculate the excess fines for each layer (i) to the nearest tenth of a percent. If the calculated excess fines are less than or equal to zero, consider them equal to zero.

$$\text{Excess Fines } (i) = (P_{200} - P_{cr})i$$

For each such layer (i) find the change in stress reduction factor (ΔSRF). The ΔSRF is equal to the stress reduction factor (SRF) at the bottom of layer (i) minus the SRF at the top of layer (i). SRF is presented in graphical form in Figure 3-5.

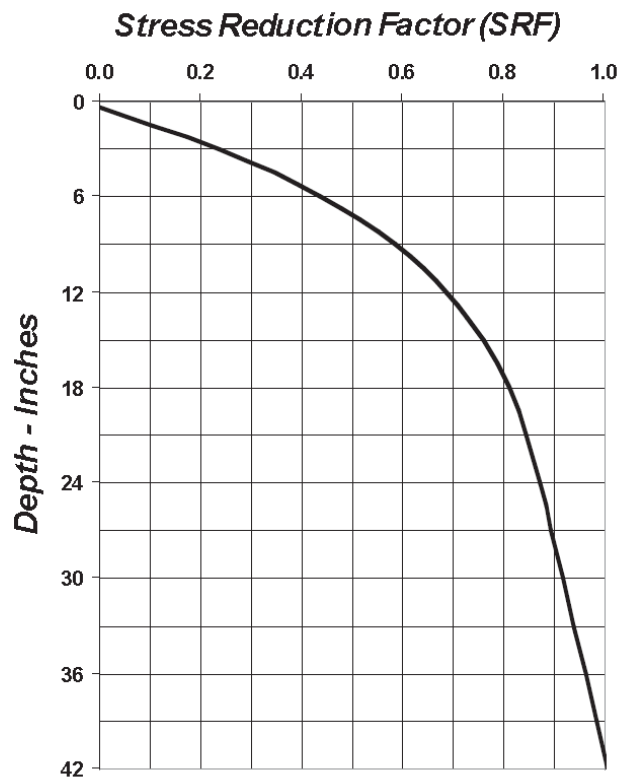


Figure 3-5 Stress Reduction Factor vs Depth Beneath a Thin Asphalt Concrete (assumes homogeneous elastic material and a standard ESAL loading)

$$SRF = -7.6477232 \times 10^{-7} y^4 + 9.7898212 \times 10^{-5} y^3 - 0.0046242158 y^2 + 0.10298199 y - 0.034613$$

where: y = depth in inches

(If SRF is negative, set SRF to zero)

Calculate the excess fines factor (EFF_i) for each layer, i.e.

$$EFF_i = (\Delta SRF_i) [(P_{200} - P_{cr})^{0.8}]$$

Add all the EFF_i values to get total EFF

$$EFF_t = \sum EFF_i$$

Calculate the predicted maximum deflection, D_p , according to the statistical relationship:

If $EFF_t = 0$, then $D_p = 0.034$

If $EFF_t > 0$, then $D_p = 0.056 + 0.0035(EFF_t)$

Determine the required pavement thickness. Enter Figure 3-6, with the predicted maximum deflection, D_p , on the horizontal axis, move vertically to intersect the appropriate equivalent axle loading (ESAL) curve, and read the required asphalt concrete pavement thickness from the vertical axis. The minimum pavement thickness is 2 inches. The thickness should be rounded up to the nearest 0.5 inch.

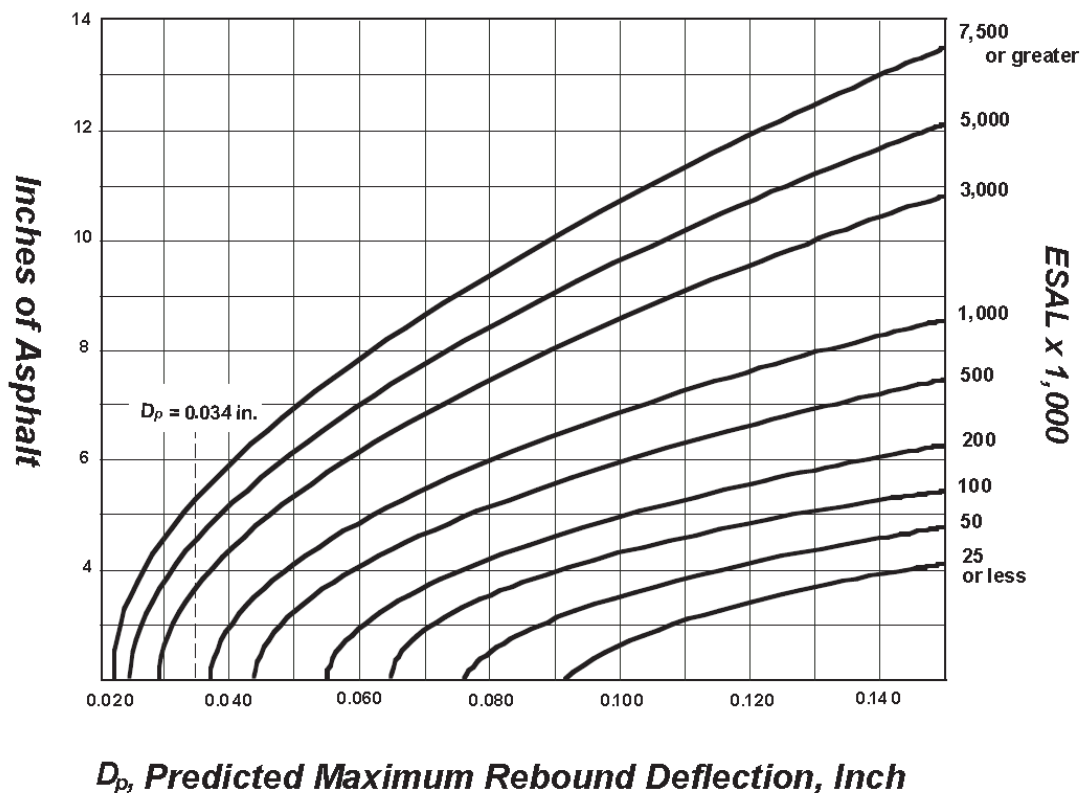


Figure 3-6 Pavement Design Chart

3.4 Stepping Through the Design Process: An Example

This example explains the excess fines design method as a series of simple computational steps. Each computational step in the example is aided by the tabular formatting shown in Table 3-1.

Step 1. Define design ESALs and a system of aggregate layers with P_{200} contents as shown below:

Table 3-1 Example Aggregate Layers

Layer Depth (inch)	P_{200} Content (%)
0 to 6	8
6 to 18	10
18 to 42	20

Design ESALs = 964,200

Step 2. Subdivide actual layering into layering shown in column 1 of Table 3-2.

Step 3. Enter applicable fines content (P_{200}) in column 2 of Table 3-2.

Step 4. Determine critical fines content (P_{cr}) for each layer from Figure 3-4. Place in column 3 of Table 3-2. Ensure that P_{200} in the upper 18 inches does not exceed P_{max} in Figure 3-4.

Step 5. Determine excess fines for each layer to the nearest tenth of a percent. Place in column 4 of Table 3-2.

$$Excess\ Fines = P_{200} - P_{cr}$$

If $P_{200} - P_{cr} < 0$, then excess fines = 0. Do not use negative values. (See layer 34 in Table 3-2.)

Step 6. Determine the change in stress reduction factor (SRF) for each layer from Figure 3-5. To do this, first locate the SRF at the top of the layer and the SRF at the bottom of the layer. Place these numbers in columns 5 and 6, respectively, of Table 3-2. Then subtract the smaller number (SRF at the top of the layer) from the larger number (SRF at the bottom of the layer). This is best done by calculating the SRFs from the equation shown in Figure 3-5. Place the result for each layer in column 7 of Table 3-2.

Step 7. Determine the excess fines factor (EFF) for each layer, and place in column 8 of Table 3-2.

$$EFF = (\Delta SRF) (P_{200} - P_{cr})^{0.8}$$

Step 8. Sum the EFFs for all layers at the bottom of column 8 of Table 3-2.

Step 9. Determine the predicted maximum deflection (D_p) $EFF > 0$, i.e., $EFF = 2.14$

Therefore:

$$D_p = 0.056 + 0.0035(EFF_t)$$

$$D_p = 0.056 + 0.0035(2.14)$$

$D_p = 0.063$ inch (bottom of Table 3.2)

Step 10. Determine the required asphalt concrete pavement thickness from Figure 3-6.

Entering Figure 3-6 with $D_p = 0.064$ and ESALs = 964,200

Figure 3-6 requires pavement thickness = 5.0 inches (rounded up to nearest 0.5 inch)

Table 3-2 Excess Fines Pavement Design Example

Column	1	2	3	4	5	6	7	8
Obtained From	Trial Dimensions	Specifications of Field Data	Figure 3.4	Column 2 minus Column 3	Figure 3.5	Figure 3.5	Column 6 minus Column 5	Column (7)*(4) ^{0.8}
Layer Number	Depth Interval (inches)	Fines Content (P ₂₀₀)	Critical Fines (P _{cr})	Excess Fines	SRF at Top of Layer	SRF at Bottom of Layer	Δ SRF	EFF
1	0 to 1	8	6.00	2.00	0.000	0.064	0.064	0.11
2	1 to 2	8	6.00	2.00	0.064	0.154	0.090	0.16
3	2 to 3	8	6.00	2.00	0.154	0.235	0.081	0.14
4	3 to 4	8	6.00	2.00	0.235	0.309	0.074	0.13
5	4 to 5	8	6.20	1.80	0.309	0.376	0.067	0.11
6	5 to 6	8	6.30	1.70	0.376	0.437	0.061	0.09
7	6 to 7	10	6.50	3.50	0.437	0.491	0.054	0.15
8	7 to 8	10	6.70	3.30	0.491	0.540	0.049	0.13
9	8 to 9	10	6.80	3.20	0.540	0.584	0.044	0.11
10	9 to 10	10	7.00	3.00	0.584	0.623	0.039	0.09
11	10 to 11	10	7.20	2.80	0.623	0.658	0.035	0.08
12	11 to 12	10	7.40	2.60	0.658	0.689	0.031	0.07
13	12 to 13	10	7.70	2.30	0.689	0.716	0.027	0.05
14	13 to 14	10	7.90	2.10	0.716	0.740	0.024	0.04
15	14 to 15	10	8.10	1.90	0.740	0.761	0.021	0.04
16	15 to 16	10	8.40	1.60	0.761	0.780	0.019	0.03
17	16 to 17	10	8.70	1.30	0.780	0.797	0.017	0.02
18	17 to 18	10	9.00	1.00	0.797	0.811	0.014	0.01
19	18 to 19	20	9.40	10.60	0.811	0.825	0.014	0.09
20	19 to 20	20	9.70	10.30	0.825	0.836	0.011	0.07
21	20 to 21	20	10.10	9.90	0.836	0.847	0.011	0.07
22	21 to 22	20	10.50	9.50	0.847	0.856	0.009	0.05
23	22 to 23	20	11.00	9.00	0.856	0.865	0.009	0.05
24	23 to 24	20	11.50	8.50	0.865	0.873	0.008	0.04
25	24 to 25	20	12.00	8.00	0.873	0.881	0.008	0.04
26	25 to 26	20	12.60	7.40	0.881	0.888	0.007	0.03
27	26 to 27	20	13.30	6.70	0.888	0.895	0.007	0.03
28	27 to 28	20	14.00	6.00	0.895	0.902	0.007	0.03
29	28 to 29	20	14.80	5.20	0.902	0.910	0.008	0.03
30	29 to 30	20	15.80	4.20	0.910	0.917	0.007	0.02
31	30 to 31	20	16.80	3.20	0.917	0.924	0.007	0.02
32	31 to 32	20	18.00	2.00	0.924	0.932	0.008	0.01
33	32 to 33	20	19.40	0.60	0.932	0.939	0.007	0.00
34	33 to 34	20	21.00	0.00	0.939	0.947	0.008	0.00
35	34 to 35	20	22.90	0.00	0.947	0.955	0.008	0.00
36	35 to 36	20	25.20	0.00	0.955	0.963	0.008	0.00
37	36 to 37	20	27.90	0.00	0.963	0.971	0.008	0.00
38	37 to 38	20	31.40	0.00	0.971	0.979	0.008	0.00
39	38 to 39	20	35.80	0.00	0.979	0.986	0.007	0.00
40	39 to 40	20	41.80	0.00	0.986	0.994	0.008	0.00
							EFF_t	2.14
Calculations: Predicted Deflection D _p = 0.064 If EFF _t = 0 then D _p = 0.034.. If EFF _t > 0 then D _p = 0.056 + 0.0035(EFF _t) Pavement thickness from Figure 3-6 (Enter with D _p on the horizontal Axis, rise vertically to curve for ESALs, then horizontally to read pavement thickness on vertical axis) = 5.0 inches for 964,200 ESALs.								

3.5 Excess Fines Design Using the AKFPD Computer Program

Perform excess fines designs for projects using the AKFPD program. The previous example “by hand” was meant to acquaint you with the computations used by AKFPD.

3.5.1 Generalized Steps Through the Excess Fines Module

1. After opening the program select *Start a New Project* and insert the project data, designer, etc.
2. Select the Excess Fines Design Method.
3. Input design ESAL data supplied by Regional Traffic/Planning Section (see chapter 6).
4. Input P_{200} content and thickness of each layer of material that will be used to a depth of 40 inches below the asphalt concrete layer into the Excess Fines Design page and run the program.
5. AKFPD subdivides each aggregate layer into 1-inch-thick sublayers.
6. AKFPD assigns each sublayer the appropriate P_{200} content at that depth (using the P_{200} content entered as design input data).
7. AKFPD calculates the amount of pavement deflection that would occur if the pavement structure were to be subjected to the dual tire loading from one side of a standard ESAL axle.* Pavement deflection is a function dependent on (1) the percent P_{200} at each sublayer depth and (2) the amount of load support contributed by each sublayer (derived from Boussinesq stress distribution theory).

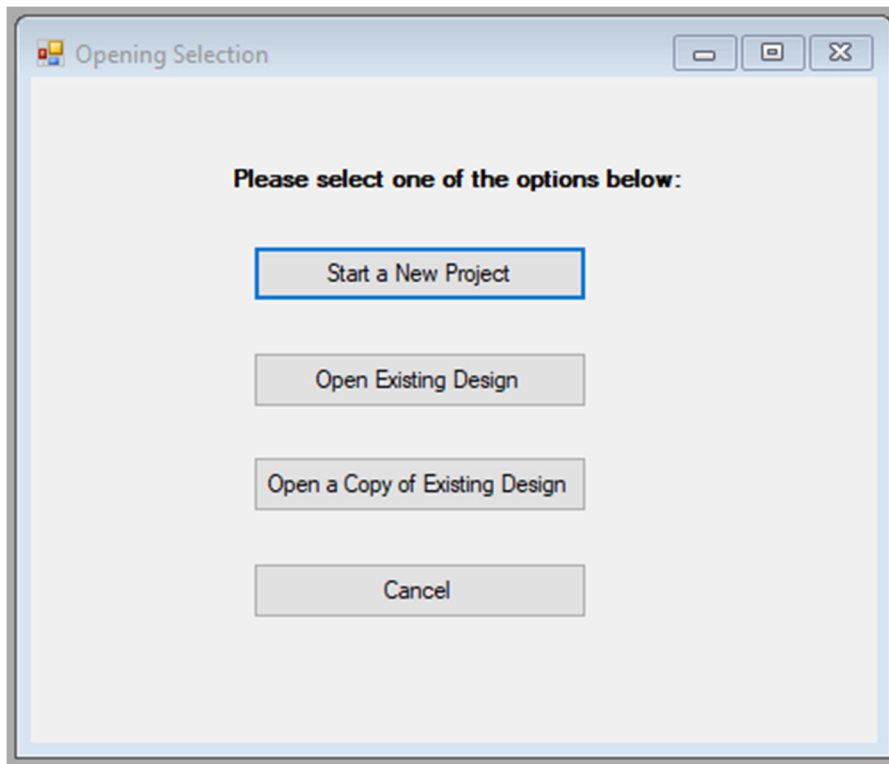
* The standard ESAL wheel load is now defined as two tires, each inflated to 110 psi, each loaded to 4,500 lbs., with a centerline-to-centerline separation of 13.5 inches.
8. AKFPD then calculates the required asphalt concrete thickness. Pavement thickness is a function dependent on (1) pavement deflection and (2) the total number of design ESALs.
9. If the calculated asphalt concrete thickness is not acceptable, the designer adjusts the input data and rerun the Excess Fines Design module.

3.5.2 Example 1—Getting Started and Performing a Simple Design

The following steps lead you through a simple example of AKFPD excess fines pavement design analysis and interpretation of the results. It is assumed that the software is installed on the computer and that the designer is familiar with the computer.

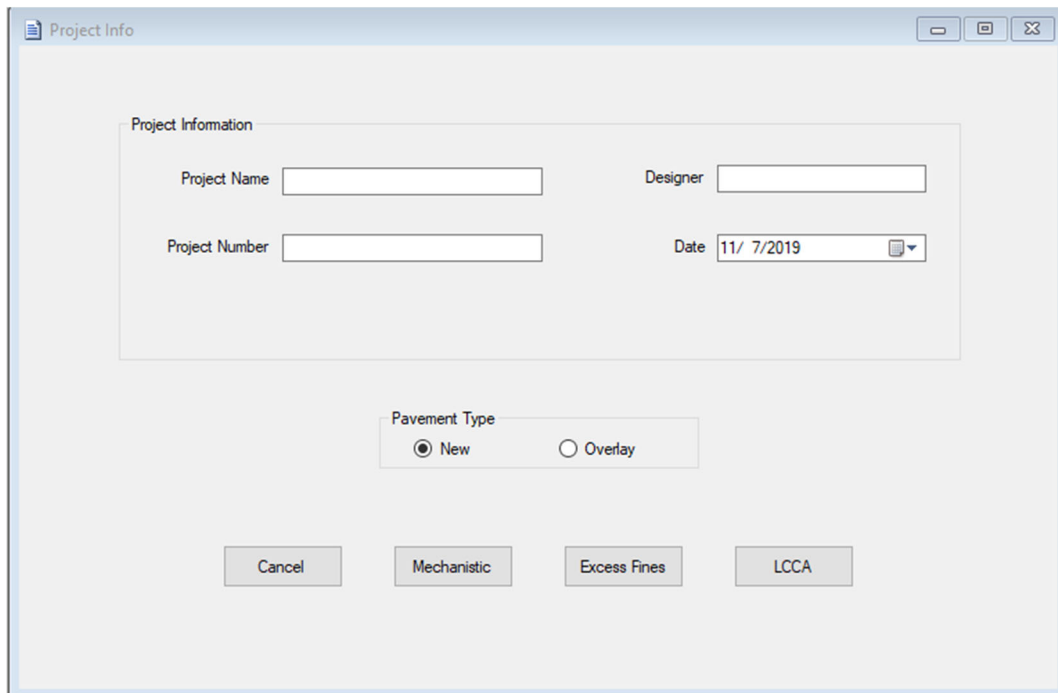
The first design example, which is explained in detail, does not use a previously saved input data file. You will use a previously saved input data file in the next example.

Step 1. Start the AKFPD software using the Start Menu. This varies slightly for each operating system. The AKFPD title screen will briefly appear followed by the landing page. (Screen Clip 3-1): To start a new project, click *Start a New Project*. This will open the project input screen with all fields blank. The *Open Existing Design* button will open an existing project in its current file location. Any changes to the data will be made in the current file and previous data will be lost. The *Open a Copy of Existing Design* button will create a new copy of an existing project so that the original data will be preserved. This is quite useful when exploring multiple options of a project design.



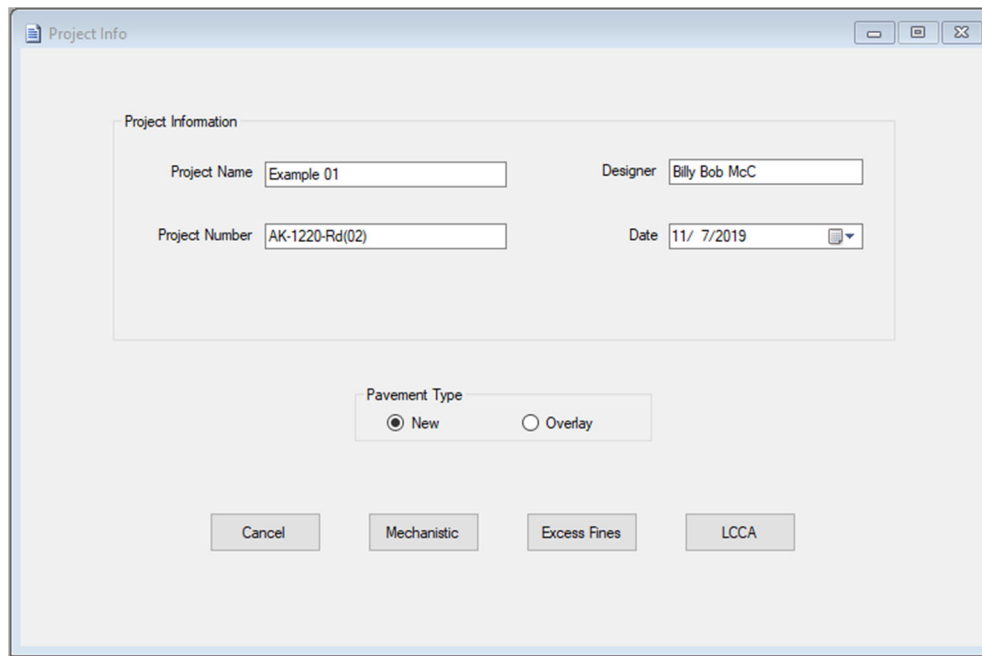
Screen Clip 3-1

Step 2. Start a new project for this example click on *Start a New Project*. The Project Info page will appear. (Screen Clip 3-2)



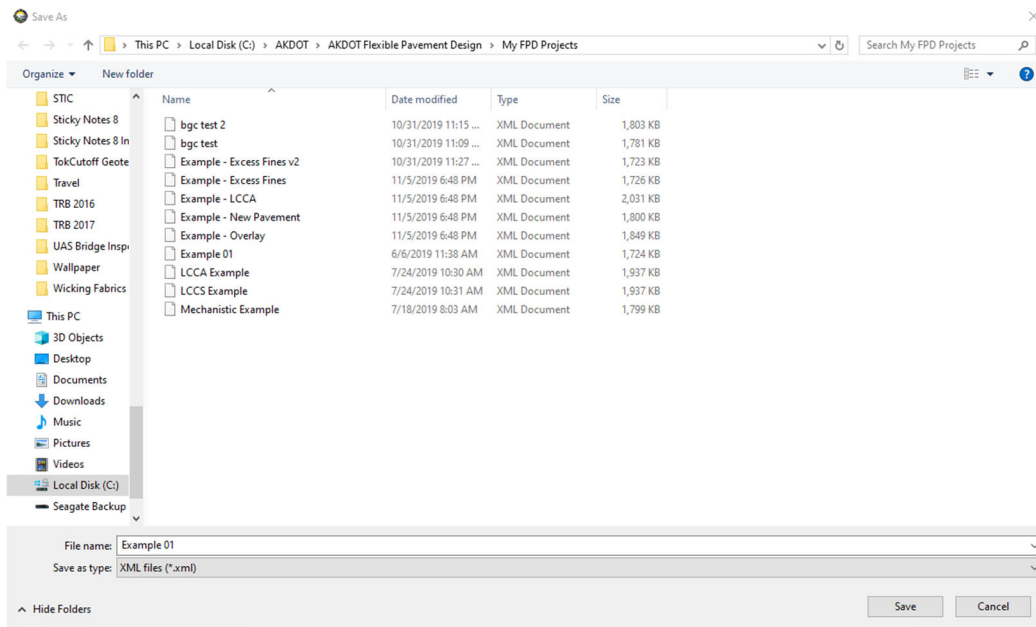
Screen Clip 3-2

Step 3. Fill in the appropriate data and click *New* then *Excess Fines*. (Screen Clip 3-3)



Screen Clip 3-3

Step 4. Enter a filename tied to the project. In this case “Example01.” Click *Save*. (Screen Clip 3-4)



Screen Clip 3-4

Step 5. There are two choices based on the information supplied by the Regional Traffic Section:

- a. Manually enter ESALs as provided by the Traffic Section. The design period, design construction year, base year, and base year total AADT must still be entered. After entering required data click **Manual ESALS**. (Screen Clip 3-5)

Or,

- b. Enter requested data and compute ESALs by clicking “Calculate.” If the Computed Design ESALs appears reasonable, click **Next**. Otherwise correct incorrect values and recalculate. (Screen Clip 3-6)

Screen Clip 3-5

Screen Clip 3-6

Step 6. This brings up the Excess Fines input screen (Screen Clip 3-7). This screen contains all input data for the analysis. All data items required for doing the complete excess fines analysis are typed as input onto the input screen (Screen Clip 3-8); click **Analyze**. Your completed screen should look like the one shown in Screen Clip 3-8. Note, the Total ESALs on the traffic page (Step 5 will be brought forward. The user is not allowed to enter values in the gray blocks. Consequently, if the ESALS require change, return to the traffic page.

Screen Clip 3-7

Layer No.	Depth Interval (in)	Thickness (in)	P200(%)
1	0 - 6	6	8
2	6 - 18	12	10
3	18 - 42	24	20

	Thickness (in)		
	Design Alternative	Asphalt Concrete	Stabilized Base
1	1.5	6	
2	2	5.5	
3	2.5	4.5	
4	3	3.5	
5	3.5	2.5	
6	4	2	
7	4.5	1	
8	5	0	

Screen Clip 3-8

The solution is shown in the Solution box (right-hand side of Screen Clip 3-8). The estimated deflection is 0.062 inches. Refer to Figure 3-6 with a traffic level of 964,200 ESALs. The required thickness of asphalt concrete is 5 inches which agrees with the AC thickness computed by the Excess Fines Design example. This example considers a case where the excess fines design method will require a pavement thickness of more than 3 inches. When the design process requires more than 3 inches of asphalt concrete, the software provides an alternative Stabilized Base Design. (Screen Clip 3-9) The alternative design assumes an asphalt stabilized base of with a Marshall Stability of 800 pounds. Note this value can be changed by the user based on values obtained from the laboratory. Contact the regional materials laboratory to obtain values for the material that will be used on the project. Any combination of Asphalt Concrete and Stabilized Base thickness is acceptable. It is the responsibility of the designer to select the most cost-effective combination.

Alternative Stabilized Base Design

Marshall Stability (lbs):

Design Alternative	Thickness (in)	
	Asphalt Concrete	Stabilized Base
1	<input type="text" value="1.5"/>	<input type="text" value="6"/>
2	<input type="text" value="2"/>	<input type="text" value="5.5"/>
3	<input type="text" value="2.5"/>	<input type="text" value="4.5"/>
4	<input type="text" value="3"/>	<input type="text" value="3.5"/>
5	<input type="text" value="3.5"/>	<input type="text" value="2.5"/>
6	<input type="text" value="4"/>	<input type="text" value="2"/>
7	<input type="text" value="4.5"/>	<input type="text" value="1"/>
8	<input type="text" value="5"/>	<input type="text" value="0"/>

Screen Clip 3-9

Step 7. If the design is not acceptable go back and modify the layer thicknesses and fines content and recalculate until an acceptable design is reached. Then click **Generate Report**. This report documents the design and contains the project information, the name of the designer, traffic information, design input, the solution, and the date of the analysis (Screen Clip 3-10). The report cannot be altered by the user. Any modifications must be made by returning to the Excess Fines Module.

Alaska DOT&PF Flexible Pavement Design Software, Version 2.0 - Project: Excess Fines Example (Filename: Example - Excess Fines.xml) - [Workbook: Excess Fines]

File Clipboard Modules View Help

	A	B	C	D	E	F
1	Project Name: Excess Fines Example			Designer: AKDOT&PF		
2	Project Number: EX03			Date: 1/27/2020		
3	Pavement Type: New Design			Unit: US Customary		
4						
5						
6	INPUT DATA			SOLUTION		
7						
8	TRAFFIC DATA			Predicted Deflection: 0.063 in		
9	ESALs: 964,200			AC Thickness: 5 in		
10	AADT: 5,000					
11				ALTERNATIVE STABILIZED BASE DESIGN		
12	UNBOUND LAYERS DATA			Marshall Stability (lbs): 800		
13						
14				Thickness (in)		
15				Asphalt Concrete		Stabilized Base
16	Layer	Thickness (in)	P200 (%)	1.5	6	
17	1	6	8	2	5.5	
18	2	12	10	2.5	4.5	
19	3	24	20	3	3.5	
20				3.5	2.5	
21				4	2	
22				4.5	1	
23				5	0	
24						
25						
26						

Screen Clip 3-10

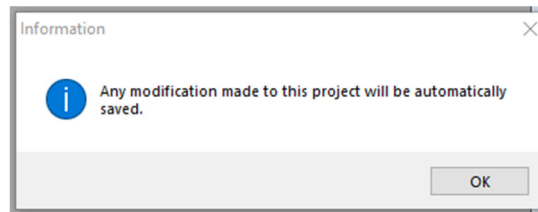
Step 8. Print the report selecting **File** then **Print** using the Ribbon at the top of the primary window.

Step 9. If you wish to perform another analysis, it is suggested that you select **File** then **Copy To** and give the file a new name such as “Example 01a.” The software will create another file that can be modified without altering the file you just completed. Make any desired changes and analyze as before.

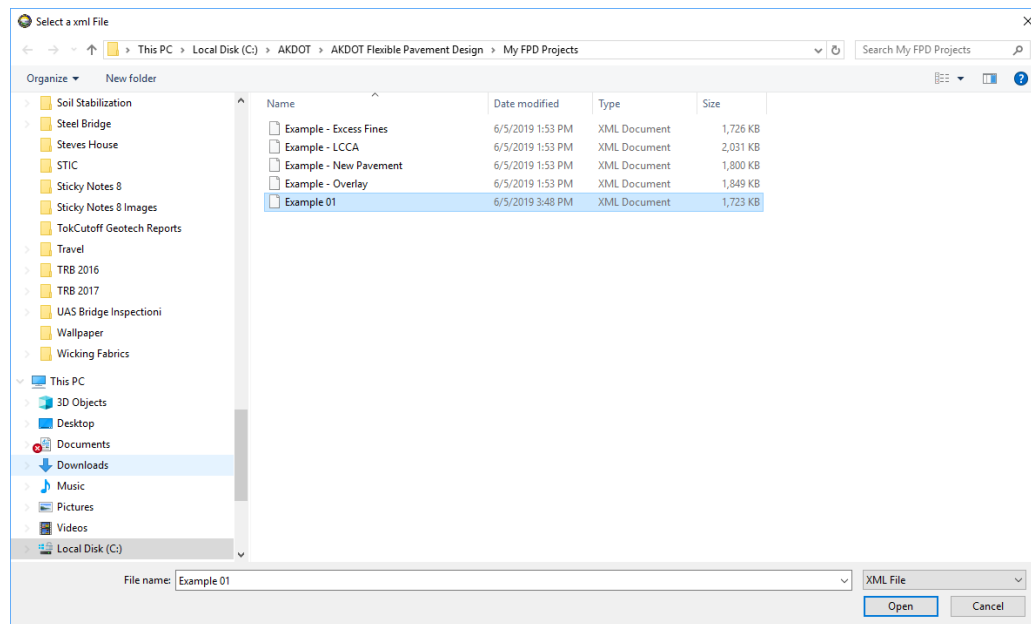
If you are finished, you may exit the program by clicking **File** then **Exit** in the left corner of ribbon across the top of the Excess Fines windows. The data has been saved for future reference.

3.5.3 Recalling Files and Modifying Files

There are two ways of recalling an existing file. By clicking the **Open Existing Design** button on the landing page shown in Screen Clip 3-1, Screen Clip 3-11 will appear warning the user that modifications will be saved. Saved data will overwrite the existing data. The warning screen will be replaced with a list of available files (Screen Clip 3-12).

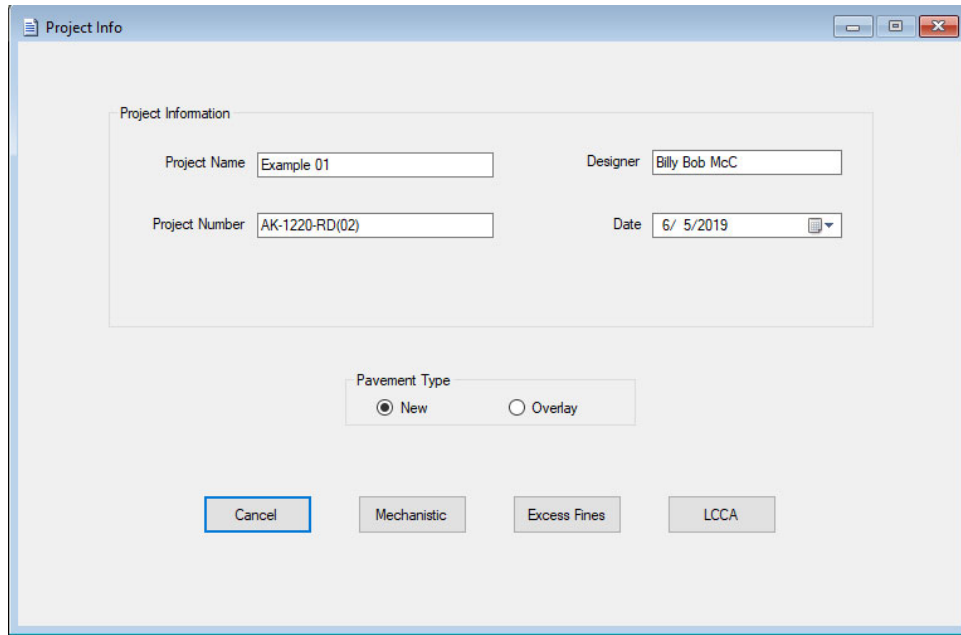


Screen Clip 3-11



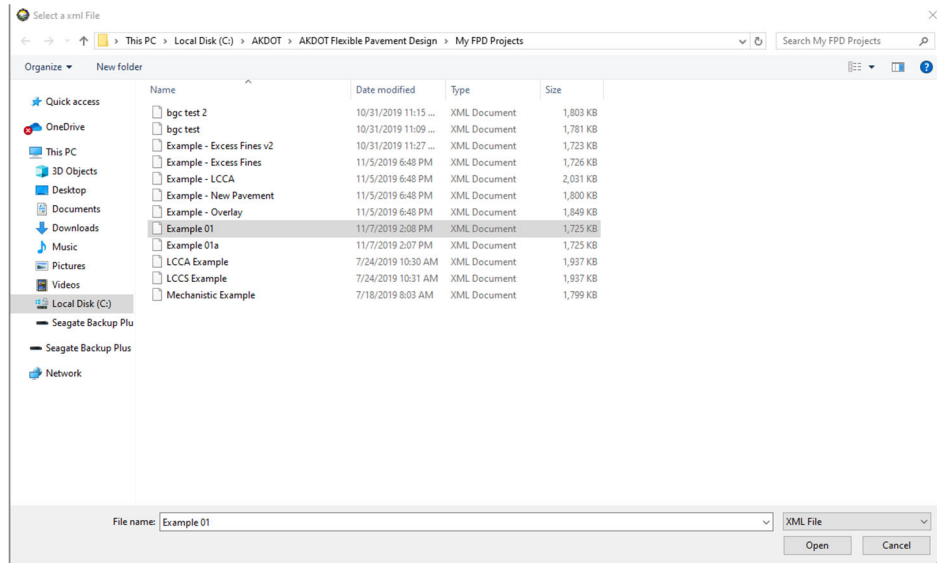
Screen Clip 3-12

Type in or select “Example01” and then click **Open**. The Project Info page will open. (Screen Clip 3-13)



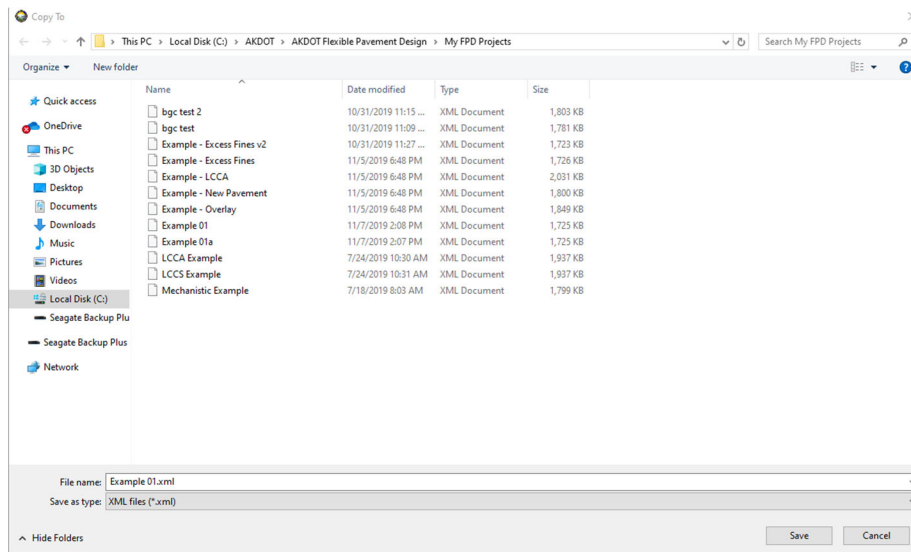
Screen Clip 3-13

At this point, any changes to the data will overwrite the data in that file causing the loss of previously stored data. If the user wishes to save the existing data, select ***Open a Copy of Existing Design*** from the opening screen (Clip 3-1). Available files will be displayed (Screen Clip 3-14).



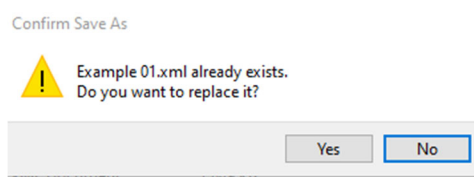
Screen Clip 3-14

Select the desired file and click ***Open***. In this case Example 01. The screen shown in Screen Clip 3-15 will appear.



Screen Clip 3-15

Note that this screen looks much like the previous screen except the button in the lower right has been changed from “Open” to “Save”. Give the filename a unique name such as “Example 01a” indicating this is a variation of Example 01. If a unique name is not selected or the user forgets to change the name a message will appear as shown in Screen Clip 3-16.



Screen Clip 3-16

Select **No** to go back to the file list to revise the name. Select **Yes** to accept the name and the data in that file will be replaced with the new data. From this point forward, any changes to the file will overwrite existing data. If the user does not wish to make changes to the file either click **File**, then **Exit** and restart the program or click **File** then **Close All**.

4 Mechanistic Design

- 4.1. Introduction
- 4.2. Summary of Mechanistic Design
- 4.3. Design Principles
- 4.4. Stepping Through the Design Process - An Example
- 4.5. Mechanistic Design Using the AKFPD Computer Program
- 4.6. Design Strategy
- 4.7. Example 2 – An Overlay Design
- 4.8. Saving, Recalling and Modifying Files
- 4.9. Advanced Users

4.1 Introduction

This chapter provides a complete discussion of the mechanistic design method. The AKFPD mechanistic method is approved for designing all types of asphalt concrete pavement structures. Chapter 2 provides detailed information concerning appropriate applications and policies for this design method.

There are many systems for mechanistically designing a flexible pavement structure. Although some Alaskan designers may have at least passing familiarity with a wide range of mechanistic design technologies, most of these methods have never been used for designing pavements in Alaska. The following presentation covers only the mechanistic methods the DOT&PF has adopted for use and associated underlying principles.

4.2 Summary of Mechanistic Design

DOT&PF adopted AKFPD which incorporated mechanistic design in 2004. The heart of the mechanistic design method is the difficult calculation of stresses and strains, i.e., structural response, at selected locations within the pavement structural layers. The computational engine, ELSYM5, had been used by DOT&PF since the 1980's. Unfortunately, the adoption of Windows 7 and above did not allow the continued use of ELSYM5.

The new software, AKFPD v 2.0, replaced ELSYM5 with WESLEA as the computational engine. The two engines yield the same results so the designer should see no differences in the computed stresses and strains nor a difference in predicted pavement life. However, the software has been rewritten to improve workflow and add life cycle costing. The principles employed have not changed.

If the designer requires a more in depth understanding of mechanistic design, textbooks by Yoder and Witczak,⁽¹⁵⁾ Ullidtz,^(16,17) and Huang⁽¹⁸⁾ provide an excellent broad base of information.

4.3 Design Principles

Alaska's mechanistic design method relies on the following three principles:

1. The pavement structure is amenable to structural analysis as a basic mechanical system of elastic layers, i.e., the structural response of the system can be calculated if the loads and the physical properties of the system's layers are known. In the Alaska mechanistic method, structural response is calculated in terms of stresses and strains at specific critical locations within the layered pavement structure (the computer program module WESLEA is used for this purpose).
2. Structural response at critical locations in the pavement structure is functionally related to pavement performance. Using this principle, it is possible to plug stress and strain values (calculated by WESLEA) into simple, empirical equations and thereby estimate the number of design load repetitions (N_r) that will cause the structure to fail (requires application of empirically derived transfer functions).

$$N_f = f(\sigma, \epsilon, \text{loads})$$

- Pavement failure is the result of a linear, incremental mechanical process. Pavement structural failure can therefore be modeled using Miner's law—a method of predicting failure by summing up fractional increments of damage.

$$\sum_{i=1}^{i=\text{total}} \left(N_a / N_f \right) \geq 1 \text{ (a definition of failure)}$$

4.3.1 Calculating Stresses and Strains

The first computational step in Alaska's mechanistic design process is estimating stresses and strains, i.e., structural response, at selected locations within the pavement structural layers using layered system analysis.

In his 1983 publication,⁽¹⁹⁾ Hicks discussed several programs useful for analyzing elastic layered systems, and he selected one of these, ELSYM5, as the stress/strain computational subroutine for use in Alaska's mechanistic design procedures. After 37 years, WESLEA replaces ELSYM5 as the computational routine used in AKFPD v 2.0 to perform this same important computational function. Since both use the same underlying principles, they yield the same results.

In simplest terms, the designer supplies WESLEA with input values of thickness and stiffness (modulus and Poisson's ratio) for each layer. Input also includes vehicle load configuration and magnitude. WESLEA calculates stresses and strains at any location within the pavement structure selected by the designer. Figure 4-1 shows a pavement structure defined in terms of individual elastic layers.

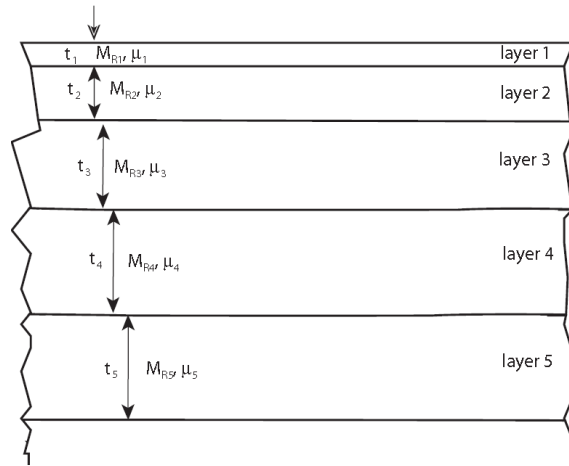


Figure 4-1 Typical Pavement Structure Showing Elastic Layers

The mathematical and programming details of how WESLEA performs these calculations are far beyond the scope of this manual. However, it is important to understand the general nature of the function performed by WESLEA as well as the principles and assumptions underlying WESLEA's stress and strain calculations.

Hicks covered this subject quite well in Chapter 2 of his 1983 publication.⁽¹⁹⁾ For analyzing layered pavement systems, he wrote: "Procedures for prediction of traffic induced deflections, stresses and strains in pavement systems are based on the principle of continuum mechanics. The essential factors that must be considered in predicting the response of layered pavement systems are: (1) the stress-strain behavior of the materials; (2) the initial and boundary conditions of the problem; and (3) the partial differential equations which govern the problem. The highway engineer, however, need only understand the stress-strain behavior of the material, the physical configuration of the problem, and the general assumptions that have been made or implied in developing

solutions to the layered system problem.” In the Alaska mechanistic method, the “solution to the layered system problem” is WESLEA.

WESLEA was selected for use in the AKFPD program because its theoretical basis and operational characteristics (within the personal computer environment) are suited to handling Alaska pavement designs. With WESLEA, or any other layered system solution, you must use realistic input values and must understand the assumptions and limitations used in developing the solution.

Hicks identified assumptions used in developing elastic layered system solutions such as ELSYM5 and WESLEA. Assumptions applicable to all elastic layer solutions:

- Each layer is infinite in horizontal extent and is composed of isotropic, homogeneous, linearly elastic material.
- Surface loadings can be represented as circular areas of uniform stress.
- Interface conditions between layers can be designated as either perfectly rough (called the “full friction” condition) or perfectly smooth (called the “no friction” or “slippery” condition).
- The underlying layer continuously supports the layer above.
- Inertial forces and vibrations are considered small in the elastic system and can be disregarded. Vibrations can damage the pavement by densifying granular materials and causing rutting, but this effect is not accounted for in mechanistic design.
- Deformations in the elastic system are small and can be disregarded.
- All loads are identical, uniform, and circular.
- All loads are placed at the surface of the elastic system and oriented normal to that surface.
- The surface of the top layer is free of shear stresses.
- Interfaces between layers are continuous, i.e., full friction.
- Nonlinear elastic behavior of materials—stress sensitivity—cannot be accommodated in WESLEA (see discussion below).
- The pavement structure modeled by WESLEA is an axisymmetric solid, which means that both load and pavement geometrics are symmetrical about a common centerline. Because of this axisymmetry, WESLEA cannot be used to analyze the effects of loads applied near the pavement edge, near cracks, or other edge-type boundaries.

WESLEA input requirements:

- One or more wheel loads must be specified at designer-selected locations at the surface of the pavement structure. The solution uses the principles of superposition to solve for stresses and strains due to application of multiple wheel loads. This means that WESLEA first calculates the stresses and strains caused by each load independently. Then, by applying the principles of superposition, total stresses, and strains at any point in the elastic layer system are determined as the sum of stress and strain contributions from each load for that point.

Chapter 6 contains information about vehicle loadings that the engineer can use for designing pavements.

- The thickness must be defined for each layer of the pavement structure. Each layer except the bottom one is assigned a finite thickness. The bottom elastic layer is defined as having semi-infinite thickness (a “bottomless” layer). As a result, the thickness must be left blank or a value of “0” entered.

- Each layer of the pavement must be assigned two elastic properties:

Resilient modulus M_R (sometimes called the repeated-load or elastic modulus).

$$M_R = \sigma_d / \epsilon_r$$

Where:

σ_d = repeated axial stress (psi)

ϵ_r = recoverable elastic (resilient) strain

The repeated axial stress (σ_d), or deviatoric stress, is defined as a repeated series of pulse loadings, where each load pulse is followed by a short rest period. One cycle of the pulsed load/rest series usually consists of a load pulse lasting 0.1 second followed by a rest period of 0.9 second. This approximates traffic wheel loadings.

The recoverable elastic strain (ϵ_r) is defined as that portion of strain, due to σ_d , that is completely recovered when the load is released. For all materials that are not perfectly elastic, a portion of the load-induced strain will not be recovered. This nonrecoverable phenomenon is due to plastic deformation or some other form of permanent displacement.

Poisson's ratio (μ)

$$\mu = \epsilon_{lateral} / \epsilon_{load\ axis}$$

Where:

$\epsilon_{lateral}$ = lateral strain (normal to the axial load direction) caused by application of the axial/vertical load

ϵ_{axial} = axial strain (parallel to the axial load direction) caused by application of the axial/vertical load

Chapter 5 provides specific information about appropriate modulus (M_R) and Poisson's ratio values that the engineer can use for designing pavements. It is important that the M_R value used as design input truly represents the modulus of the material after it has been placed and compacted.

You must define the locations where WESLEA will calculate stresses and strains within the layered elastic system.

WESLEA will determine the stress/strain response at any location within a specified elastic layered system due to a specified load. In fact, WESLEA produces more output values than are used in Alaska's mechanistic design method. You must know which WESLEA output values to use and where (within the layered structure) WESLEA must calculate these values. Only two of values produced by WESLEA are used in Alaska's method, and the locations that must be selected for analysis are specific.

The two output values of interest are:

1. Maximum horizontal tensile strain, ϵ_t , at the bottom of bound layers
2. Maximum vertical compressive stress, σ_v , at the top of each unbound layer

The following section describes, in detail, which layers are specified for evaluation according to strain and which are evaluated according to stress, and why.

A few more words are necessary about selecting WESLEA analysis locations. As has been stated above, specify calculation of stresses and strains either at the top or the bottom of specified layers. But where (in a horizontal sense) along the top or the bottom of a layer will the **maximum** stress or strain value be found? The tire configuration of the design load (wheel locations) determines where maximum stresses or strains will be found. In

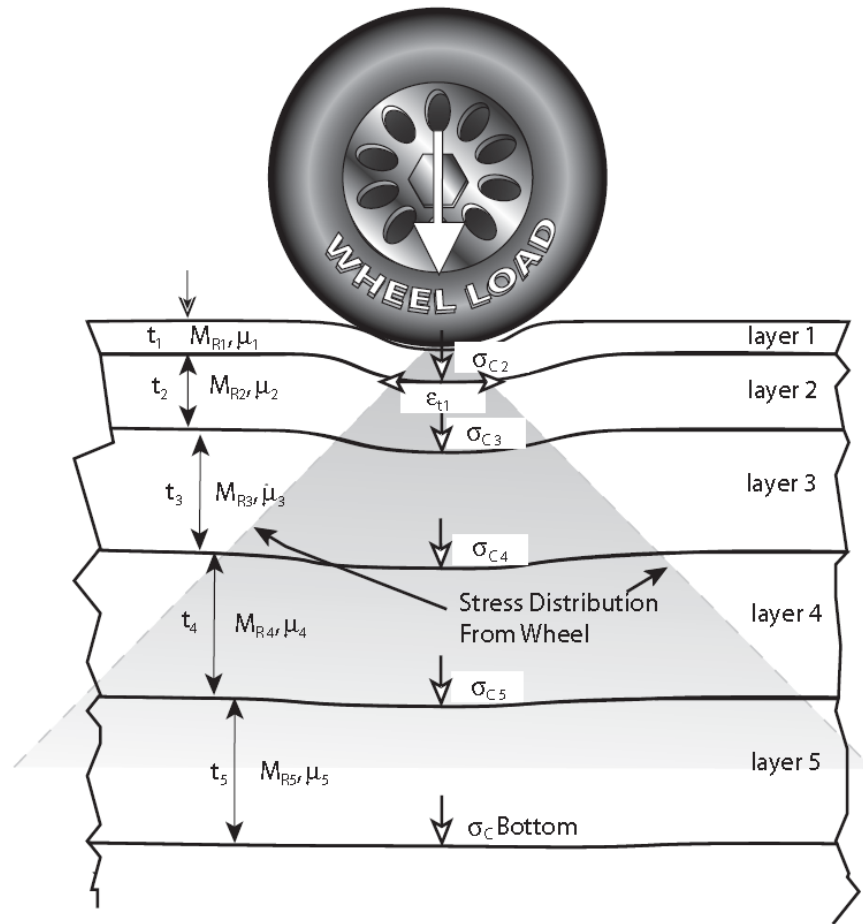


Figure 4-2 Analysis Data and Critical Load Analysis Points Used by WESLEA

the simplest case of a single tire design load, the maximum value will be found directly under the center of the load. For design load configurations having two or more tires, various locations along the bottom of the layer must be searched to find the maximum value. It is important to realize that, because of superposition effects, the horizontal location where the maximum value will be found will change as the depth of the analysis increases. Comparison of Figure 4-2 and 4-3 provides a visual, conceptual example of how superposition applies to a layered pavement structure. Figure 4-2 shows how the load of single tire is distributed with depth. For example, the load-induced vertical compression stress “felt” by the soil at 36 inches depth would be much less than the stress at the pavement surface directly beneath the tire. Figure 4-3 shows how the depth-distributed loads from two tires superimpose (and add together) at some depth. In Figure 4-3, see how the load distributions of the two tires overlap between the tires. When multi-wheel design loads are involved, it is often possible to simplify the search for the maximum horizontal strain value at the bottom of heavily bound layers. Figure 4-4 indicates how analysis locations are selected, and how taking advantage of the symmetry of multi-wheel configurations can minimize the number of search locations.

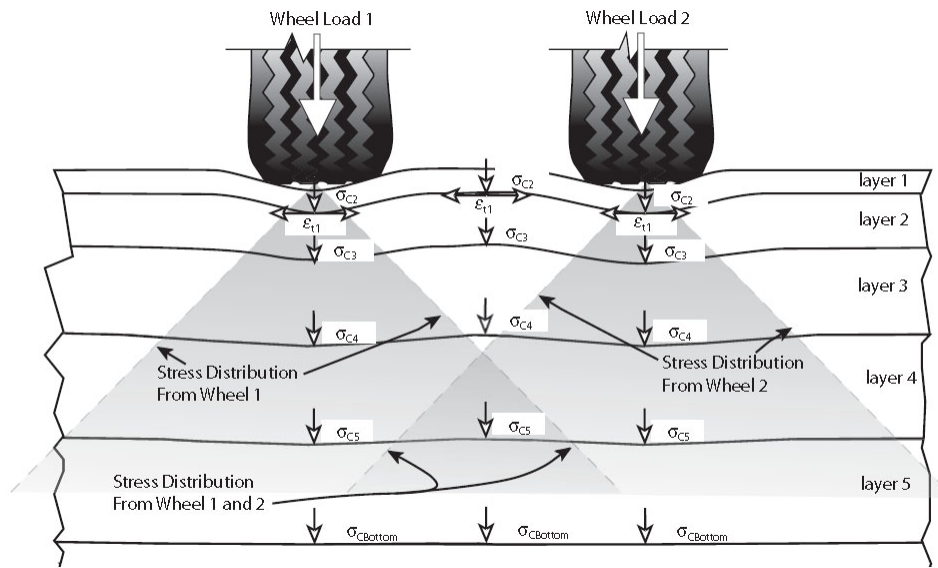


Figure 4-3 Elastic Pavement Layers Illustrating Superposition

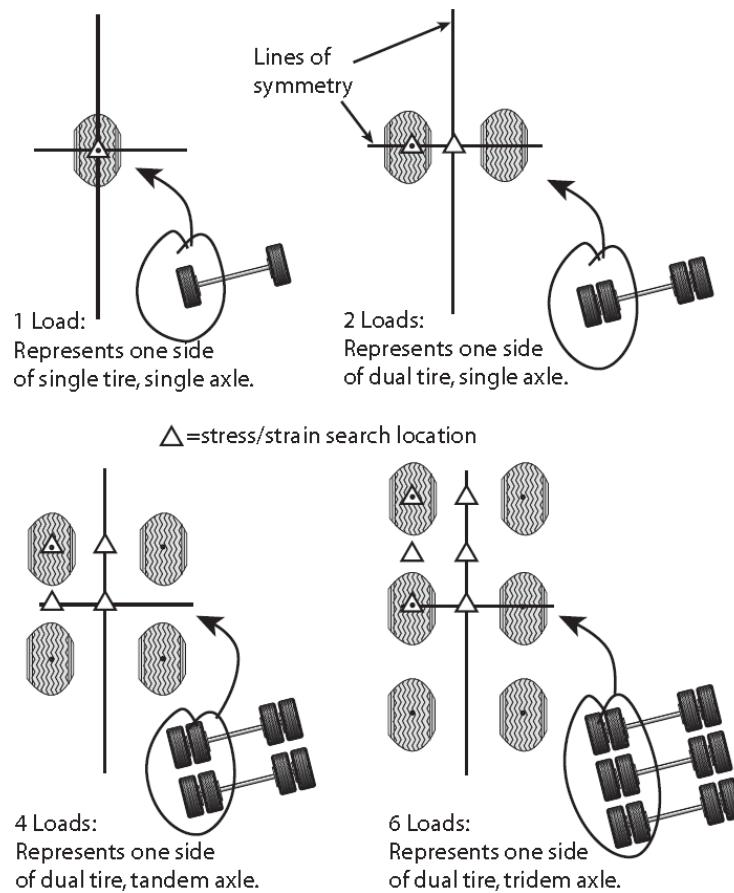


Figure 4-4 Plan View of Design Loads with Structural Response Search Locations

4.3.2 Relating Structural Response to Performance (Estimating the Number of Load Repetitions to Failure)

Empirical equations have been developed that relate stresses and strains at specific locations within the pavement structure to the number of load repetitions that will cause the structure to fail. These equations are variously known as “damage equations” or “transfer functions.” DOT&PF has selected two equations for application in the mechanistic design method. These equations have been incorporated into the AKFPD program and are discussed below.

Why two damage equations used instead of just one? The answer is that the mechanistic design method defines two distinctly different modes of pavement structural failure. Each mode of failure is controlled by a different structural response parameter.

Fatigue Failure: This type of failure exhibits itself as fatigue cracks (alligator cracks) that are seen at the pavement surface. Only heavily bound layers such as the asphalt concrete surfacing and heavily bound bases are susceptible to this failure mode. Fatigue cracking originates at the bottom of the bound layer and propagates upward to the surface. All heavily bound layers will become fatigue cracked after they are subjected to enough load repetitions. Fatigue failure of heavily bound layers is analogous to a paper clip failing after it is bent many times. Figure 4-5 is a photograph showing advanced alligator cracking.



Figure 4-5. Advanced Fatigue Cracking of Highway Pavement

The Asphalt Institute (TAI) developed an equation that predicts, for each heavily asphalt-bound layer, the number of load repetitions until fatigue failure (bottom-up cracking failure) occurs. The TAI equation applies only to heavily bound layers where asphalt cement has been used as the binder. The response parameter used in the TAI equation for each layer is the maximum horizontal tensile strain (ϵ_n) at the bottom of that layer.

The TAI equation—for fatigue failure (applicable to asphalt concrete and other heavily bound layers) is:

$$N_f = C \times 0.07958 \times \varepsilon_h^{-3.291} \times |E^*|^{-0.854}$$

(for fatigue cracking over 45% of the wheel path area, equivalent to about 20% of the total area)

$$C = 10^M$$

$$M = 4.84 \times \left(\frac{V_b}{V_v + V_b} - 0.69 \right)$$

where:

N_f = fatigue life (number of load repetitions to fatigue failure)

ε_h = maximum horizontal tensile strain at the bottom of the bound layer, in/in

$|E^*|$ = modulus of the asphalt concrete material, psi

V_v = percent air voids volume in total mix

V_b = percent binder volume in total mix

$$V_b = \frac{(\gamma_{mix}) \times (\%AC)}{G_b \times \gamma_w}$$

where:

γ_{mix} = mix density, pcf

% AC = binder content, weight %

G_b = binder specific gravity

γ_w = water density, pcf (62.4 pcf)

Functional Failure: This mode of failure appears as a combination of roughness and rutting (sometimes called functional distress). Failure occurs after the pavement structure is subjected to enough load repetitions to cause permanent deformations of unbound or lightly bound lower layers. All layers that are not heavily bound and susceptible to fatigue failure are susceptible to functional failure.

The Per Ullidtz equation predicts, for each unbound or lightly bound layer, the number of load repetitions until functional failure of that layer occurs. The response parameter used in the Per Ullidtz equation is the vertical compressive stress (σ_v) at the top of unbound or very lightly bound layers.

The Per Ullidtz equation—for functional failure⁽¹⁷⁾ (applicable to unbound or lightly bound layers) is:

$$N_f = \frac{1}{R} \times 3.069 \times 10^{10} \times \left(\frac{E}{E_0} \right)^{3.26b} \times \sigma_v^{-3.26} \quad (\text{for about 1-inch rut depth})$$

where:

N_f = number of load repetitions to functional failure

R = regional factor = 2.75 for Alaska conditions

E = modulus of the unbound or lightly bound material, psi

$E_0 = 23,000$ psi

$b = 1.16$ if $E < E_0$; otherwise $b = 1$

σ_v = maximum vertical compressive stress at the top of the layer, psi

Keep in mind where the horizontal tensile strain ϵ_h and vertical compressive stress σ_v values used in the above equations come from; they are calculated by the WESLEA subroutine of the AKFPD program.

N_f values calculated using the above equations define the maximum number of load repetitions (the allowable repetitions) that can be applied to the pavement structure before it fails. In other words, these equations define the potential “life” of the pavement structure in terms of load repetitions to failure (N_f). It should be obvious that any number of actual load repetitions (N_a) less than N_f ($N_a < N_f$) will consume a fraction of that life. Similarly, load repetitions $\geq N_f$ will fail the pavement structure. Conceptually, the fractional portion of the pavement structure’s life consumed by a total number of applied loads (N_a) can be calculated simply by dividing the number of applied loads by the allowable repetitions to failure (N_a/N_f). Failure is said to occur when $N_a/N_f \geq 1$. This line of reasoning leads to discussion of the next principle.

4.3.3 Predicting Structural Failure by Summing up Damage Increments

Mechanistic design applies the incremental damage concept using Miner’s law.

The $N_a/N_f \geq 1$ equation introduced above is conceptual. The equation is used in a modified form in the actual pavement design process. The modified equation (known as Miner’s law) expresses failure as an incremental process that is calculated using simple summation. In Miner’s law, a failed condition is approached as fractional increments of damage are added together. Each increment can be thought of as a fraction of total failure caused by design load repetitions applied when a specific combination load and/or materials conditions exist (such as during different seasons of the year).

The Miner’s law expression presented below shows that a condition of failure exists when the sum of damage increments exceeds 1.

$$\sum_{i=1}^{i=total} \left(\frac{N_a}{N_f} \right)_i \geq 1$$

where:

N_a is the actual number of design vehicle loads applied during the i^{th} set of conditions

N_f is the number of design loads that would cause failure during the i^{th} set of conditions

The $\left(\frac{N_a}{N_f}\right)_i$ term represents the fractional increment of damage occurring during the i^{th} set of loads and materials conditions. The Miner's law concept can be explained easily by an example. The following example examines an asphalt concrete pavement layer and the fractional portions of fatigue life consumed during various seasons of the year.

4.3.4 A Simple Application of Miner's Law

In this example, first analyze the asphalt concrete pavement layer of a pavement structure using WESLEA and the TAI damage equation previously discussed (TAI applies to heavily bound layers). The pavement is analyzed for three sets of conditions ($i = 1$ through 3). The three sets of conditions are: spring, summer, and fall. WESLEA will be used to calculate the maximum tensile strain at the bottom of the asphalt concrete layer for each season, based on the properties of the materials (materials properties will be different for each season) and the design load. Using the maximum tensile strain calculated by WESLEA for each season, the TAI equation will be used to calculate the number of loads to fatigue failure (N_f) for each season. The actual number of load repetitions expected during each season (N_a) is known based on traffic forecasting, e.g., ESALs. The application of Miner's law is laid out in tabular form below.

Table 4-1 Miner's Law Example

Season	N_a	N_f	N_a/N_f
Spring	300,000	600,000	0.50
Summer	1,000,000	5,000,000	0.20
Fall	900,000	7,000,000	0.13
Miner's Law $\sum N_a/N_f$:			0.83

Miner's law states that the failure will not occur unless:

$$\sum_{i=1}^{i=\text{total}} \left(\frac{N_a}{N_f}\right)_i \geq 1$$

Therefore, the asphalt concrete pavement should not fail in fatigue with the expected number of load repetitions. Furthermore, the results indicate that no more than about 83% of the fatigue life of the asphalt concrete pavement will be consumed by the expected load repetitions.

4.4 Stepping Through the Design Process—An Example

1. The designer assembles design input data.
 - a. Wheel configuration, tire pressure, and intensity of design load Dual tire load of 4,500 lbs./tire, with 110 psi tire pressure
Tires separated 13.5 inches center-to-center

- b. Number of applied design load cycles expected during the pavement's design life (this total number is subdivided according to the percentages of load applications during spring, summer, fall, and winter)
1,000,000 load repetitions total, subdivided as:
- 30% in spring = 300,000 load repetitions = $N_{a, \text{Spring}}$
 - 50% in summer = 500,000 load repetitions = $N_{a, \text{Summer}}$
 - 20% in fall = 200,000 load repetitions = $N_{a, \text{Fall}}$
- c. M_R and μ of each layer in the proposed pavement structure (one set of these materials properties must be defined for each season of the year, i.e., spring, summer, fall, and winter)

Table 4-2 Example Materials Properties

Material Type	Thickness (inches)	Spring		Summer		Fall	
		M_R (ksi)	μ	M_R (ksi)	μ	M_R (ksi)	μ
Asphalt Concrete	3.5	754	0.30	508	0.30	508	0.30
Base Course	6	44	0.35	51	0.35	51	0.35
Subbase	36	26	0.40	36	0.40	36	0.40
Subgrade	Semi-I*	44	0.35	10	0.45	10	0.45

(* semi-infinite thickness)

- d. Proposed thickness of each layer in the proposed pavement structure. Layer thicknesses are included in the Table 4-2.
- e. Asphalt concrete mix properties:
- i. density of asphalt concrete = 150 pcf;
 - ii. % asphalt cement by total weight of mix = 5.5
 - iii. % air voids = 4

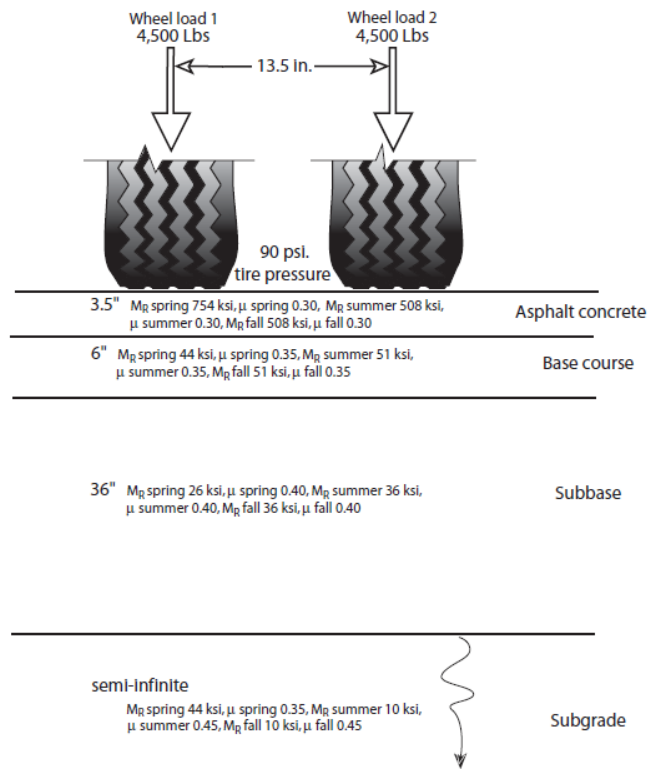


Figure 4-6 Pavement Structure Example Problem

- The designer loads data to AKFPD input screen and runs program.
- AKFPD calculates response stresses and strains at critical locations within the pavement structure due to application of the design load. A separate set of response stresses and strains is calculated for each critical location and for each season, based on materials properties for that season.

Table 4-3 Calculated Stresses and Strains

Season	Tensile Strain (micro-strain) at Critical Location	Compressive Stress (psi) at Critical Locations		
	Bottom of Asphalt Concrete (depth = 3.5")	Top of Base (depth = 3.5")	Top of Subbase (depth = 9.5")	Top of Subgrade (depth = 45.5")
Spring	192	26.4	11.6	1.9
Summer	202	33.5	13.6	1.0
Fall	202	33.5	13.6	1.0

- AKFPD then calculates the number of times the design load can be applied before all the pavement's life is expended and pavement failure occurs. The number of allowable loads is separately calculated for each critical location and for each season, using the previously calculated stresses and strains as input values to empirical damage equations (transfer functions).

Table 4-4 Calculated Loads to Failure

Season	Loads to Failure, N_f , Based on Analyses at Critical Locations			
	For Asphalt Concrete	For Base Course	For Subbase	For Subgrade
Spring	3,030,000	2,090,000	5,480,000	11,094,000,000
Summer	3,590,000	1,550,000	9,430,000	467,510,000
Fall	3,590,000	1,550,000	9,430,000	467,510,000

- AKFPD then calculates seasonal **fractional amounts** of pavement life expended (seasonal damage fractions) by dividing the number of design loads for each season by the number of allowable loads for that season.

Table 4-5 Calculate Fractions of Pavement Life Expended During Each Season

Season	N_a/N_f Based on Analyses at Critical Locations			
	For Asphalt Concrete	For Base Course	For Subbase	For Subgrade
Spring	$3.00e5 / 3.03e6$ = 0.099	$3.00e5 / 2.09e6$ = 0.144	$3.00e5 / 5.48e6$ = 0.055	$3.00e5 / 1.11e10$
Summer	$5.00e5 / 3.59e6$ = 0.139	$5.00e5 / 1.55e6$ = 0.323	$5.00e5 / 9.43e6$ = 0.053	$5.00e5 / 4.68e8$ = 0.00107
Fall	$2.00e5 / 3.59e6$ = 0.056	$2.00e5 / 1.55e6$ = 0.129	$2.00e5 / 9.43e6$ = 0.021	$2.00e5 / 4.68e8$ = 0.00043

- AKFPD next applies Miner’s law to determine **total amount** of pavement life expended by adding together the seasonal fractions. According to Miner’s law, the pavement has failed if this total damage summation for any layer of material is ≥ 1 .

Table 4-6 Using Miner's Law to Sum Damage

Season	Sum Damage: $(N_a/N_f)_{spring} + (N_a/N_f)_{summer} + (N_a/N_f)_{fall}$ Based on Analyses at Critical Locations			
	Asphalt Concrete	Base Course	Subbase	Subgrade
Spring	0.099	0.144	0.055	0.00003
Summer	0.139	0.323	0.053	0.00107
Fall	0.056	0.129	0.021	0.00043
Miner’s Law Damage	0.294	0.596	0.129	0.00153

Interpreting Miner’s law for this example: Miner’s law states that the pavement structure will fail if the damage summation for any critical location exceeds 1, i.e. $\sum_{i=1}^{i=total} \left(\frac{N_a}{N_f} \right)_i \geq 1$.

In this example, damage sums do not exceed 1 for any critical location. The proposed design is therefore structurally acceptable. Select the most economical design (using life-cycle cost analysis) from several different

designs that are found to be structurally acceptable (using mechanistic design).

In addition to determining acceptability or unacceptability of the proposed pavement structure, Miner's law provides some useful insight into the structure's behavior. Referring to the previous table, one can determine which critical locations (and therefore which materials) are controlling acceptability of the proposed design. In this case, the damage summation assessed at the top of the subgrade is near zero at less than 0.2% (table sum = 0.00153), showing that the subgrade is essentially completely protected from load effects by overlying structural layers. We can see that the asphalt concrete pavement has received enough load repetitions to use up about 30% (table sum = 0.294) of its available life, and that 60% (table sum = 0.596) of the base course's life has been exhausted. Such information can help you predict which failure modes are most probable in the future.

If the total damage summation had been ≥ 1 for any layer of material, the pavement structure (as a whole) cannot withstand the required number of cycles of the design load. In that case the designer would rerun the program using different sets of input variables, such as different aggregate layer thicknesses, higher quality aggregate materials, thicker asphalt concrete pavement, etc., until the total damage summation for each layer of material is less than 1.

4.4.1 Overlaying an Existing Asphalt Concrete Layer

Pavement overlay involves placing an additional (new) asphalt concrete layer on top of an existing asphalt concrete layer. The new total thickness is designed to withstand a specified number of future design load repetitions. The method of designing the required thickness for the new layer accounts, mechanistically, for fatigue damage done to the old asphalt concrete layer by past load repetitions (before the overlay). You can choose to operate AKFPD in an overlay design mode. If operated in this mode, AKFPD will automatically calculate the minimum required overlay thickness.

The process of determining an overlay thickness for an existing paved structure uses essentially the same series of steps shown above. Conceptually, the old asphalt concrete layer simply becomes redefined as the second layer of a "new" pavement structure. AKFPD then determines the thickness of new asphalt concrete layer required to satisfy the structural requirements of future traffic. The minimum overlay thickness is 1.5 inch.

Refer to Chapter 2's GP-11 and Section 2.2.3 for overlay design guidelines.

4.5 Mechanistic Design Using the AKFPD Computer Program

4.5.1 Generalized Steps Through the Program for Designing a New Pavement Structure

1. The designer assembles design input data:
 - a. Wheel configuration, tire pressure, and load intensity of design load
 - b. Number of design load repetitions expected during the pavement's design life (this total number is subdivided according to the percentages of load applications during spring, summer, fall, and winter)
 - c. M_R and μ of each layer in the proposed pavement structure (one set of these materials properties must be defined for each season of the year, i.e., spring, summer, fall, and winter)
 - d. Asphalt concrete mix properties
 - e. Proposed thickness of each layer in the pavement structure
2. The designer loads data to AKFPD input screen and runs program.
3. AKFPD calculates response stresses and strains at critical locations within the pavement structure due to application of the design load. A separate set of response stresses and strains is calculated for each season based on materials properties for that season.

- AKFPD calculates allowable loads, i.e., the number of times the design load can be applied before the pavement's life is 100% expended and pavement failure occurs. A separate set of allowable loads is calculated for each season using the previously calculated stresses and strains as input values to empirical damage equations (sometimes called transfer functions).
- AKFPD calculates seasonal **fractional amounts** of pavement life expended (seasonal damage fractions) by dividing the number of design loads for each season by the number of allowable loads for that season.
- AKFPD applies Miner's law to each layer to determine **total amount** of layer life expended by adding together the seasonal fractions. According to Miner's law, the layer (and pavement) has failed if this "total damage summation" is ≥ 1 .
- If the total damage summation is ≥ 1 , the pavement structure is not adequate to withstand the required number of cycles of the design load. Rerun the program using different sets of input variables, e.g., different aggregate layer thicknesses, higher quality asphalt or aggregate materials, thicker asphalt concrete layer, etc., until the total damage summation is less than 1.

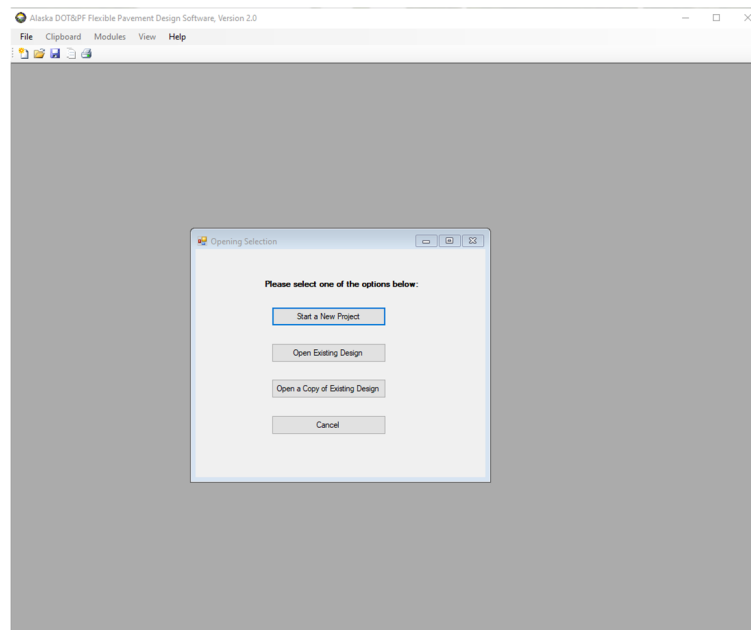
4.5.2 Example 1—Getting Started and Performing a Simple Design

The following steps lead you through a simple example of AKFPD-2 mechanistic pavement design analysis and interpretation of the results.

This design example does not use a previously saved input data file. In other examples, the use and modification of previously saved input data files are explored.

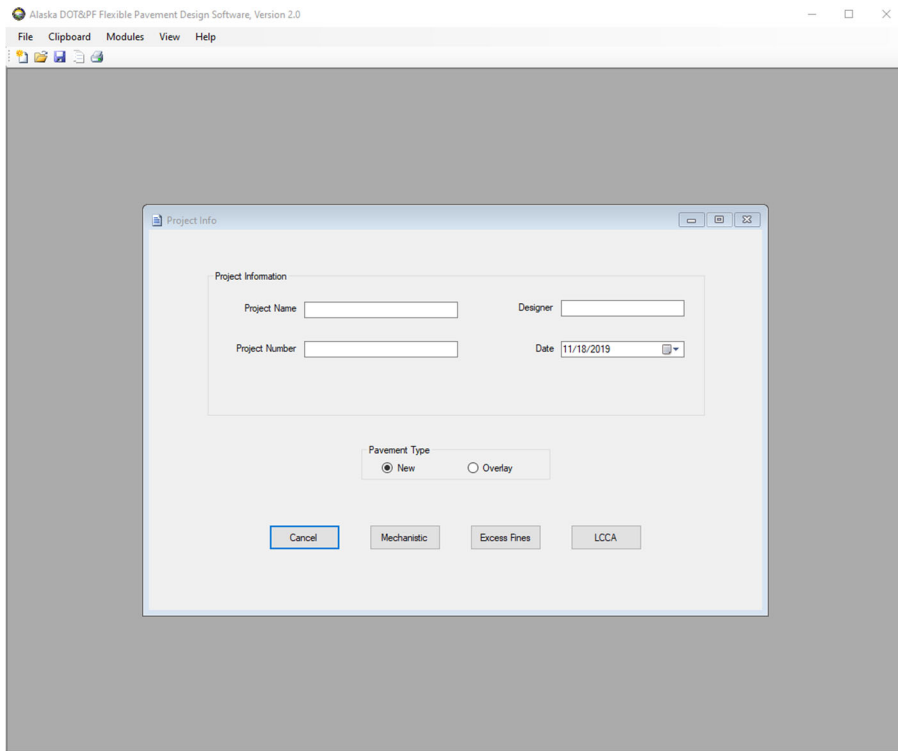
You will gain cumulative experience by going through each design example in turn because each successive example builds on information and tips contained in the previous one.

Step 1. Start the AKFPD software using the Start Menu. This varies slightly for each operating system. The AKFPD title screen will briefly appear and the landing page will then appear. (Screen Clip 4-1)



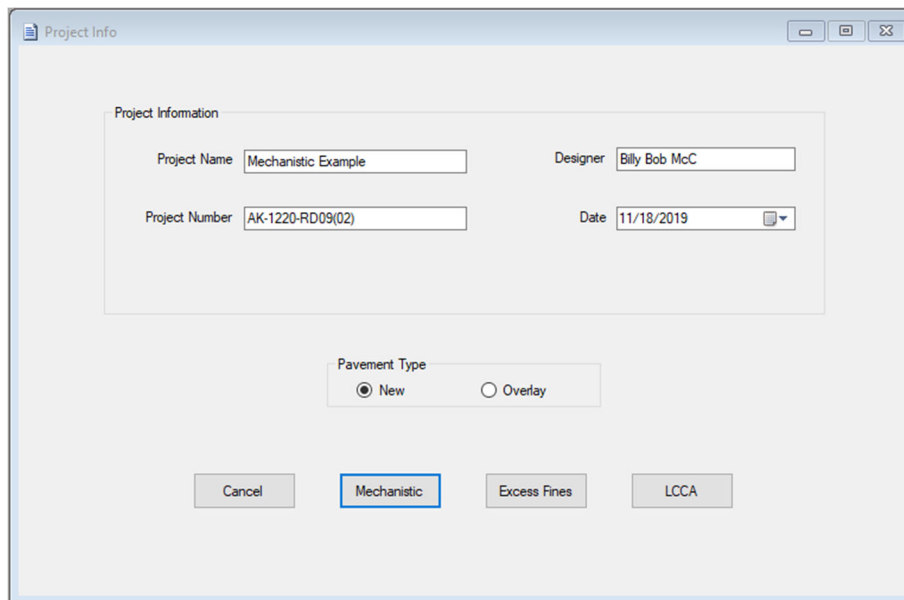
Screen Clip 4-1

Step 2. Start a new project for this example click on *Start a New Project*. The Project Info page will appear. (Screen Clip 4-2)



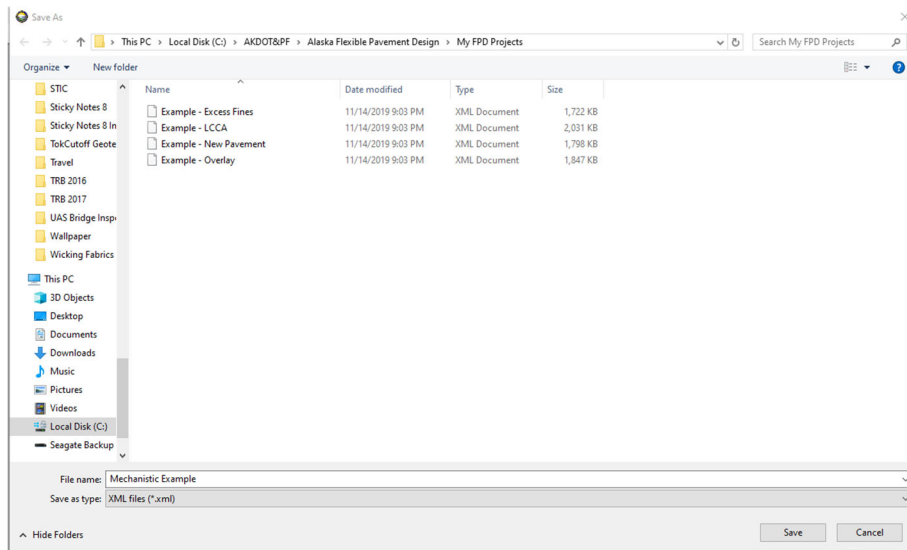
Screen Clip 4-2

Step 3. Fill in the appropriate data and click *New* which will tell the software that this pavement structure is new. Then click *Mechanistic* to save the project information. (Screen Clip 4-3)



Screen Clip 4-3

Step 4. Enter a filename tied to the project. In this case “Mechanistic Example”. Click **Save**. (Screen Clip 4-4)



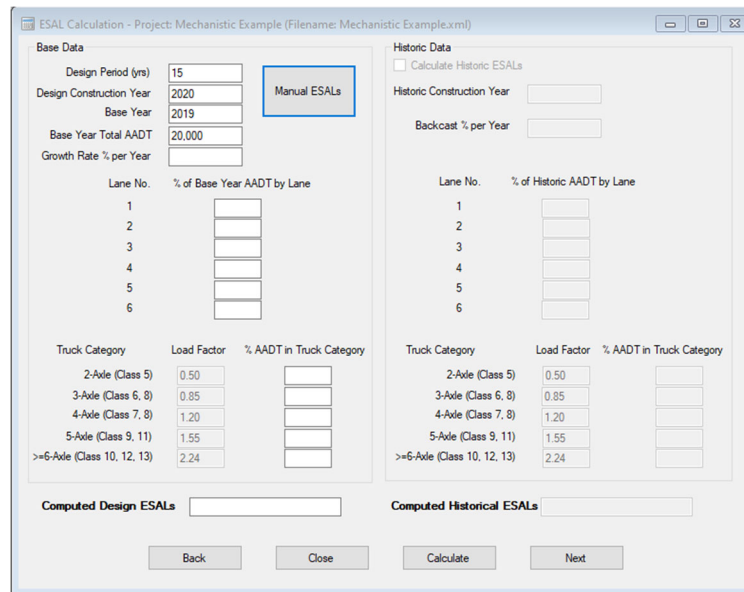
Screen Clip 4-4

Step 5. You have two choices based on the information supplied by the Regional Traffic/Planning Section:

- Manually enter ESALs as provided by the Traffic Section. The design period, design construction year, base year, and base year total AADT must still be entered (Screen Clip 4-5). After entering required data click **Manual ESALS**. Note the **Manual ESALS** button changes to **Computed ESALS** and the **Computed Design ESALS** box at the bottom left of the data input form changes to **Manual Design ESALS** (Screen Clip 4-6). Click “**Next**”. Go to Step 6.

Or

- Enter requested data and compute ESALs by clicking **Calculate**. A detailed discussion of the input data is provided in Chapter 6. If the **Computed Design ESALS** appears reasonable, click “**Next**.” Otherwise correct incorrect values and recalculate. (Screen Clip 4-7)

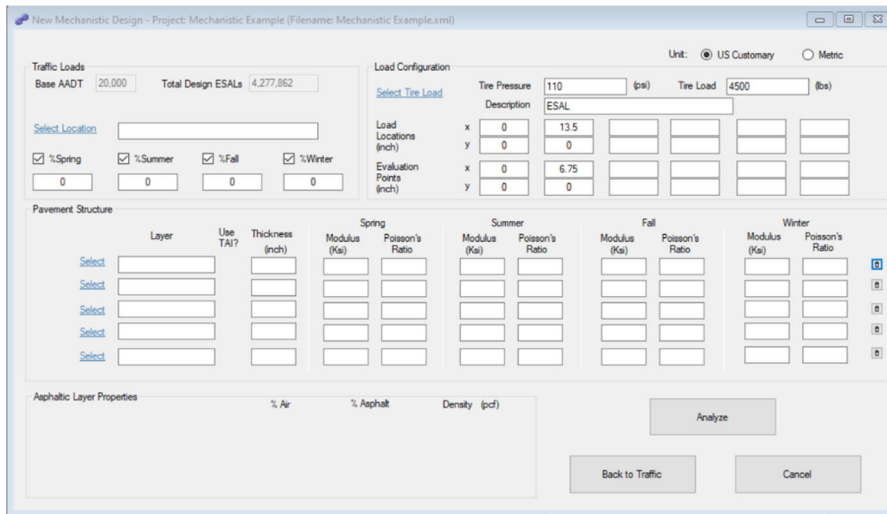


Screen Clip 4-5

Screen Clip 4-6

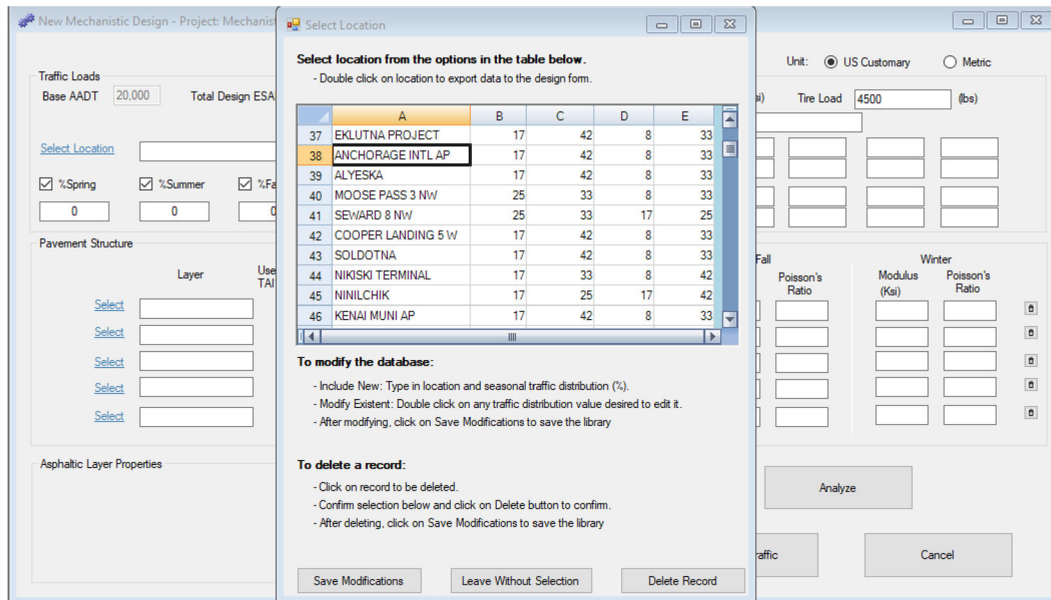
Screen Clip 4-7

Step 6. The Mechanistic design input screen will appear. Note the traffic inputs are brought forward. If the traffic data require changes, the user must go back to the traffic module by clicking on the *Back to Traffic* button. (Screen Clip 4-8)



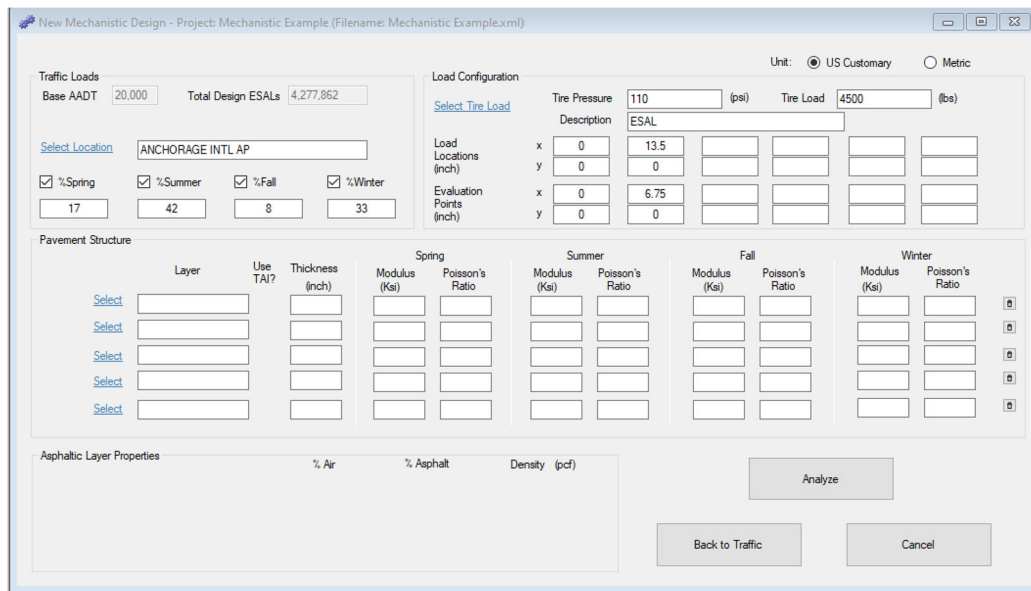
Screen Clip 4-8

Step 7. The traffic data and calculated ESALs (Step 5-a/b) has been brought forward. Click on *Select Location*.



Screen Clip 4-9

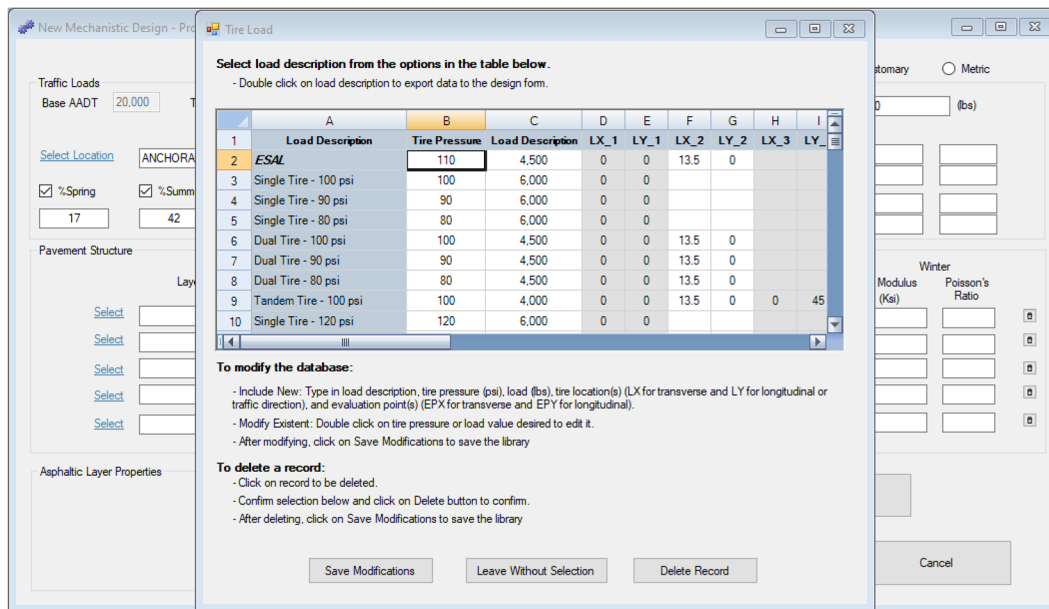
Step 8. Screen Clip 4-9 allows the designer to input the percentage of the year in each season. The stiffness of the pavement layers varies with the seasons. Therefore, the amount of damage caused to the pavement structure by a given number of vehicle loads varies depending on the season the load is applied. The user may select one of the existing areas, modify one of the existing areas, create new areas or delete an area. Instructions are provided on the screen. For this example, double click on the *Anchorage Area*. The user will be returned to the Mechanistic Design screen.



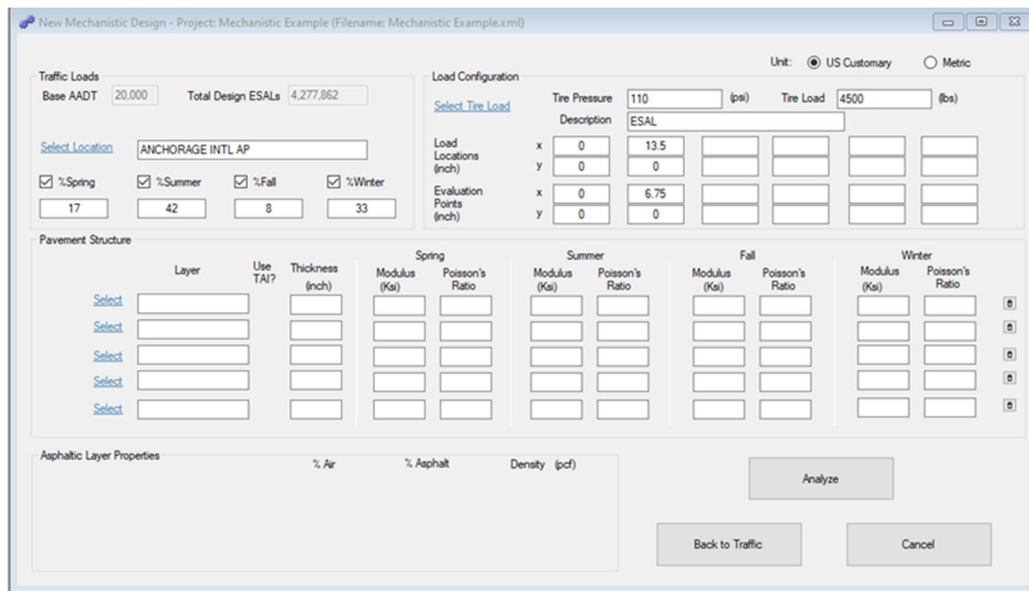
Screen Clip 4-10

Note the values for the seasons are inserted. (Screen Clip 4-10)

Step 9. The **Load Configuration** defaults to **ESAL** since this is the load configuration for which the traffic loadings were developed. However, there may be instances where you need to design for a specific load configuration. In this case select the appropriate load configuration by clicking on **Select Tire Load** and then select the desired load configuration (Screen Clip 11). Follow the instructions provided on the screen, to add or delete load configurations. Be sure to save modifications if appropriate.



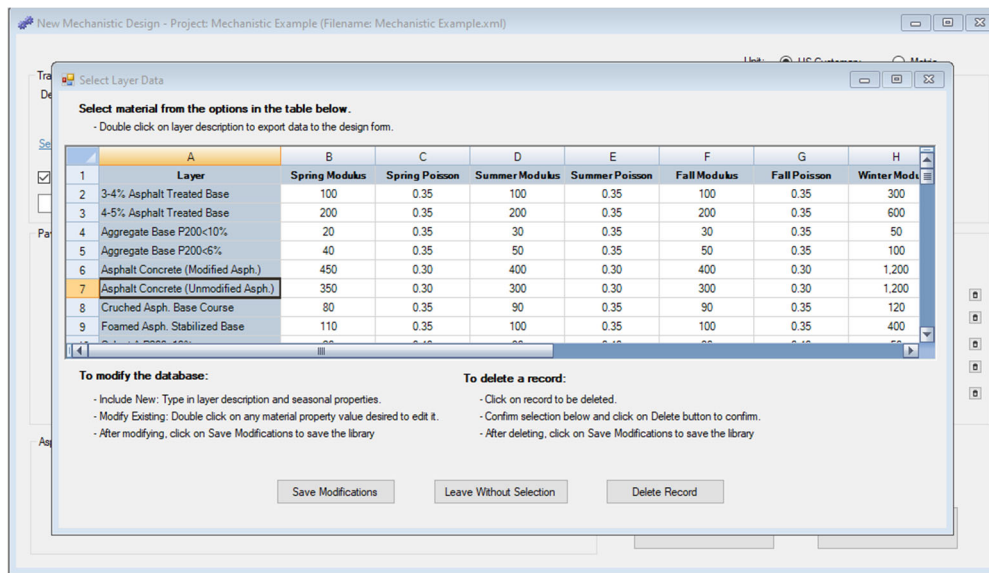
Screen Clip 4-11



Screen Clip 4-12

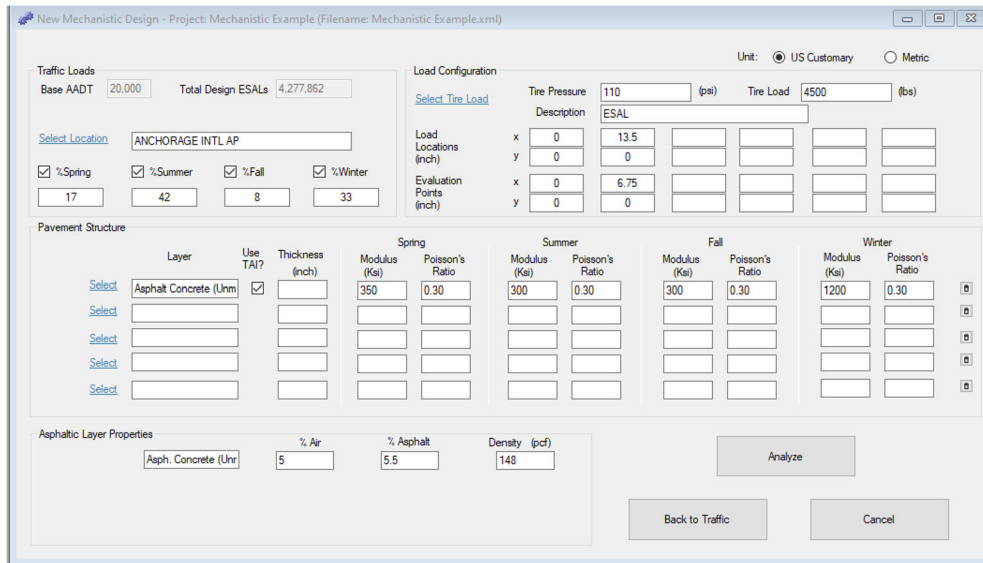
Note that the Load Configuration has been entered. (Screen Clip 4-12)

Step 10. The Pavement Structure must now be defined in the box labeled “Pavement Structure.” Up to 5 layers are available for analysis. As explained earlier, the properties (thickness, modulus, and Poisson’s ratio) of each layer must be defined. Refer to Chapter 5 or work with your materials section to obtain appropriate values. You can either enter the values manually or click on *Select* to bring up a menu of materials. In most cases it is most efficient to go to the materials menu and use an existing material or modify an existing material. At this point click on *Select* for the first layer. The “Select Layer Data” dialog box will appear. (Screen Clip 4-13)



Screen Clip 4-13

In most cases, the first layer will be some form of Asphalt Concrete. The values shown here represent a typical set of properties. Note the properties vary with season. However, the properties for your project may be different. As before, follow the instructions provided to make changes. For this example, double click on “Asphalt Concrete (Unmodified Asphalt.)”

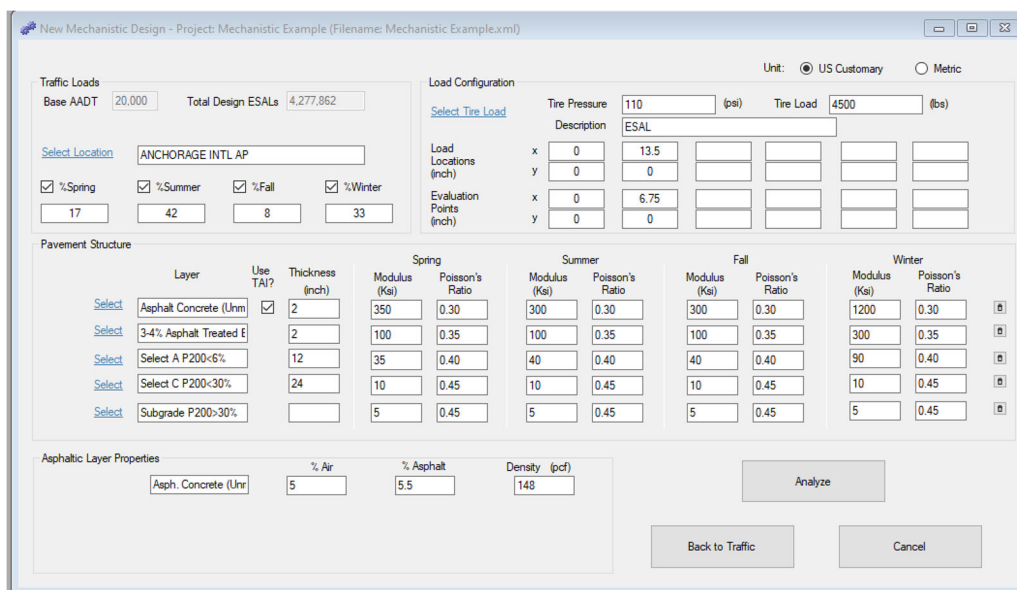


Screen Clip 4-14

The materials properties associated with Asphalt Concrete (Unmodified Asph.) have been inserted. (Screen Clip 4-14) Note that the column labeled “Use TAI?” has been checked. This tells the software that the layer is an asphalt bound material and that The Asphalt Institute fatigue equation should be used to compute the number of cycles to failure. This is defined in the materials database. Also note that the air voids, percent asphalt and density for the asphalt concrete are shown in the “Asphaltic Layer Properties” box. These may be modified based on information from the Materials Section. You may wish to make these changes in the database if these properties will be commonly used.

Now enter the thickness of the Asphalt Concrete. With experience, the designer will learn to estimate the thickness based on the number of loadings (ESALs) and the materials properties. For instructive purposes, this example will assume the asphalt thickness is 2 inches.

Repeat Step 10 for each layer in your pavement structure. Again, refer to Chapter 5 or work with the materials section to determine the appropriate properties.



Screen Clip 4-15

The pavement structure will look something like Screen Clip 4-15. Note that the Asphalt Treated Base which has 3-4 % asphalt did not require the use of the TAI. This is because, asphalt treated base with low asphalt cement content is not considered a bound material. Asphalt treated bases above 4% asphalt cement are considered bound. This boundary has been determined through experience and should be used as guidance.

Notice the Subgrade has a thickness of "0." This indicates that the bottom layer is semi-infinite. The last layer will always have a thickness of 0. If the user inserts a value other than 0, the software will change the value to 0.

Step 11. As stated earlier, this procedure does not determine the pavement thickness. Rather, it will indicate whether your design meets Miner's Law and the criteria established by the Department. Consequently, check the design by clicking *Analyze*.

Layer	Critical Z Coordinate (in)	Asphalt Properties	Season	Modulus (Ksi)	Poisson's Ratio	Tensile Micro Strain	Compressive Stress (psi)	Million Cycles to Failure	Past Damage (%)	Future Damage (%)	Total Damage (%)
Thickness (in): 2 Name: asphalt Concrete (Unmodified Asphalt) Use TAI: Yes	1.99	Air: 5 Asphalt: 5.5 Density (pcf) 148	Spring Summer Fall Winter	350 300 300 1200	0.30 0.30 0.30 0.30	202 184 184 69		2.96 4.67 4.67 35.99		24.55 38.49 7.33 3.92	24.55 38.49 7.33 3.92
Thickness (in): 2 Name: 3-4% Asphalt Treated Base Use TAI:	2.01	Air: Asphalt: Density:	Spring Summer Fall Winter	100 100 100 300	0.35 0.35 0.35 0.35	72.1 76.0 76.0 67.9		1.15 0.97 0.97 50.20		63.35 185.76 35.38 2.81	63.35 185.76 35.38 2.81
Thickness (in): 12 Name: Select A P200<6% Use TAI:	4.01	Air: Asphalt: Density:	Spring Summer Fall Winter	35 40 40 90	0.40 0.40 0.40 0.40	38.6 41.9 41.9 34.5		0.29 0.34 0.34 9.01		254.13 529.96 100.94 15.67	254.13 529.96 100.94 15.67
Thickness (in): 24 Name: Select C P200<30% Use TAI:	16.01	Air: Asphalt: Density:	Spring Summer Fall Winter	10 10 10 10	0.45 0.45 0.45 0.45	5.5 5.3 5.3 3.0		1.84 2.04 2.04 12.56		39.62 87.99 16.76 11.24	39.62 87.99 16.76 11.24
Thickness (in): 0 Name: Subgrade P200>30%	40.01		Spring Summer Fall Winter	5 5 5 5	0.45 0.45 0.45 0.45	1.2 1.2 1.2 0.9		16.52 17.39 17.39 57.31		4.40 10.39 1.98 2.46	4.40 10.39 1.98 2.46
Total Damage:								74.29		287.30	287.30
Total Damage:								900.70		900.70	900.70
Total Damage:								155.61		155.61	155.61
Total Damage:								19.23		19.23	19.23

Screen Clip 4-16

Screen Clip 4-16 shows the output of the analysis.

The first column (column B, below row 13) provides the input description of each layer.

The second column (column C) provides the critical Z coordinate (depth) of the analysis. Note that the depth of the analysis of the Asphalt Concrete is 0.01 inches above the bottom of that layer. This is because we are looking for the horizontal strain at the bottom of the asphalt concrete layer. By locating the analysis point just above the bottom of the layer, we ensure the software analyzes the layer at the appropriate point.

All unbound layers are analyzed just below the top of the layer for the same reason except that the vertical compressive stress will be used to estimate the number of cycles to failure. It is always a good idea to check these values to ensure the software is selecting the analysis points correctly.

The third column (columns D-E) shows the Design Loadings for each season based on the seasonal distribution shown in column B. These values are used to estimate the damage in each layer for each season. Below that, the asphalt properties are provided for the asphalt concrete layers.

Columns F, G, H, and I (above row 11) display the loading input and the seasonal properties provided by the user. Check these to ensure the values you input. If not, return to the input screen to make the appropriate modifications.

Columns I and J (below row 13) display the output generated by WESLEA for each layer and season. The horizontal strain for each asphaltic layer and for each season is provided in Column I. Column J provides the vertical stress at the top of each layer and season for every unbound layer.

Column K displays the number of cycles to failure for each layer and for each season. The equations/transfer functions are discussed earlier in the chapter.

Columns L and M provide the damage (i.e. actual traffic divided by the number of cycles to failure shown in column K) that are caused by the traffic both past and future. Since this is a new pavement structure, there is no past traffic. (Column L is empty).

Finally, column N shows the total damage as the sum of the past and future damage. Since there is no past traffic, column M and N should be equal.

All values in column N must be equal to or less than 100 %. If any of the values exceed 100%, the proposed pavement structure is inadequate and therefore rejected. If this is the case, go back to the input screen and modify the pavement structure. As you gain experience, the process will become more efficient.

The software highlights any damage value exceeding 100% in yellow and issues the warning “At least one-layer damage is more than 100%” in the upper right corner under Project Status. The warning is highlighted in yellow.

Note that the second layer (3-4% ATB) is the one that failed. Modifying the layers below will have limited effect on reducing the damage in the layer. To reduce the vertical stress at the top of the layer requires increasing the thickness or stiffness of the ATB or/and the asphalt concrete layer.

Step 12. To return to the Mechanistic Input Screen, click on **Modules** in the ribbon and click **Mechanistic Design**. You will be returned to Screen Clip 4-15. Change the thickness of the Stabilized Base to 4 inches and the Asphalt Concrete thickness to 4 inches. Click **Analyze**. Screen Clip 4-17 shows the output.

Layer	Thickness (in)	Name	Use TAI	Critical Z	Air%	Asphalt%	Density (pcf)	Spring Modulus (Ksi)	Summer Modulus (Ksi)	Fall Modulus (Ksi)	Winter Modulus (Ksi)	Poisson's Ratio	Tensile Micro Strain	Compressive Stress (psi)	Million Cycles to Failure	Past Damage (%)	Future Damage (%)	Total Damage (%)
1	4	sphalt Concrete (Unmodified Asph	Yes	3.99	5	5.5	148	350	300	300	1200	0.30	166	6.54	6.54	6.54	6.54	47.80
2	4	3-4% Asphalt Treated Base		4.01				100	100	100	300	0.35	166	44.5	5.52	6.20	0.44	10.49
3	12	Select A P200<6%		8.01				35	40	40	90	0.40	14.1	11.9	8.77	3.90	0.48	9.55
4	24	Select C P200<30%		20.01				10	10	10	10	0.45	3.2	3.2	10.45	3.27	1.75	7.19
5	0	Subgrade P200<30%		44.01				5	5	5	5	0.45	0.9	0.6	47.87	0.71	0.74	1.56

Screen Clip 4-17

Note the Project Status has been changed to “*All layer damages less than 100%*” highlighted in green. The proposed pavement structure is acceptable. However, this may not be the most cost-effective solution. The designer will want to look at alternative designs to find the most cost-effective design. This will be discussed further in Chapter 8.

Step 13: It is suggested that the results be printed as documentation of the design by clicking *File* to the left of the ribbon. Then click *Print* and follow the instructions.

Step 14. Before creating a new pavement structure, create a copy of the existing structure to modify. Simply go to the ribbon, click *File* then *Copy To* and enter a new name such as Mechanistic Design A. This will allow the user to document several alternatives without the need to recreate the data input.

Now click “*Modules*” then “*Mechanistic Design*” from the ribbon. Modify the pavement structure and analyze the structure. Repeat the process until an optimal pavement structure is realized.

4.6 Design Strategy

The underlying premise used in pavement design is to limit the stress or strain on each layer such that the layer will perform as desired. Experience has shown that it is best to work from the bottom up. In general, it is best to reduce the load at the top of each unbound layer either by increasing one or more of the layers above that layer or to stiffen one or more layers above the layer in question. Changing the stiffness below that layer will typically have little impact. The designer must contemplate whether it is less expensive to increase the failing layers stiffness or to reduce the vertical loading by stiffening one or more layers using a more expensive material.

If the bound layers are failing, there is no choice but to alter the material below the bound layer. In most cases, it is more effective to modify the thickness or stiffness immediately below the bound layer that is failing.

It is critical to get the materials properties of the wearing course and the subgrade correct. The properties of the wearing course effectively determine the relationship between the horizontal strain and the number of cycles to fatigue. The subgrade properties are essentially fixed by the local geology. However, these properties affect the response of the pavement structure above. Take a few moments to test this by fixing the pavement structure and modifying the subgrade modulus.

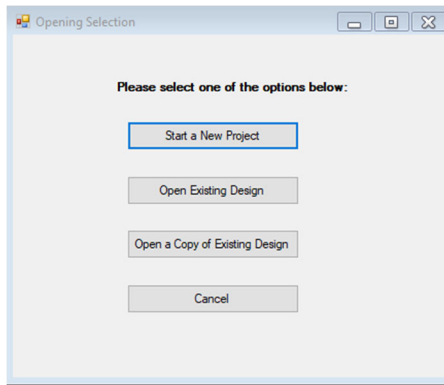
Designers are encouraged to explore the impacts of modifying layer properties on the other layers in the pavement structure.

4.7 Example 2—An Overlay Design

This example makes use of the Mechanistic Overlay Design procedures contained within AKFPD-2 to perform overlay design analyses. The concept of an overlay design is to determine how much pavement thickness needs to be added to an existing pavement to satisfy the requirements of future design ESALs. During the overlay design process, AKFPD accounts for structural life expended due to past ESAL applications. Past ESALs are those that were applied to the original pavement structure before the overlay placement.

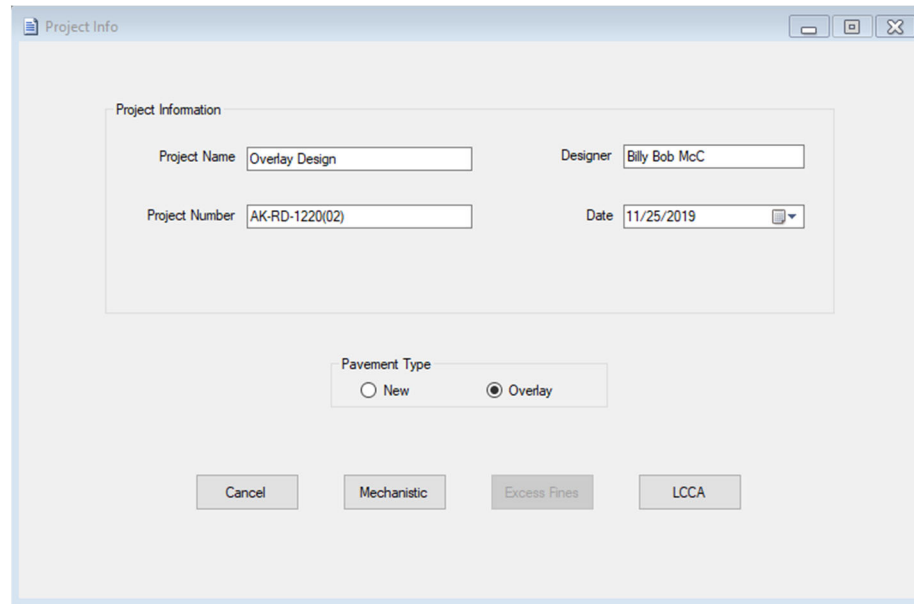
You will gain cumulative experience by going through each design example because each example builds on information and tips contained in the previous one. This example uses many of the program operation techniques presented in Section 4.5.2, example 1. **If you have not thoroughly familiarized yourself with example 1, do so before proceeding with this example.**

Step 1. Begin this example by starting a new project. (Clip 4-18) Click *Start a New Project*.



Screen Clip 4-18

Step 2. Fill in the project information. Change the Pavement Type from **New** to **Overlay** by clicking **Overlay**. Now click **Mechanistic** to save the project information (Screen Clip 4-19)



Screen Clip 4-19

Step 3. Note the Historical Data box is now available. (Screen Clip 4-20) The Planning/Traffic Data Section will provide the data for this input screen. As before, you can either fill in the data or select “**Manual ESALs**”. If you input the data click **Calculate**. If the “Computed Design ESALs” and the “Computed Historical ESALs” look reasonable click **Next**.

ESAL Calculation - Project: Overlay Design (Filename: Overlay Design.xml)

Base Data

Design Period (yrs) 15
 Design Construction Year 2020
 Base Year 2019
 Base Year Total AADT 20,000
 Growth Rate % per Year 2

Manual ESALS

Historic Data

Calculate Historic ESALS
 Historic Construction Year 2000
 Backcast % per Year 2.6

Lane No.	% of Base Year AADT by Lane	Lane No.	% of Historic AADT by Lane
1	45	1	45
2	55	2	55
3	0	3	0
4	0	4	0
5	0	5	0
6	0	6	0

Truck Category	Load Factor	% AADT in Truck Category	Truck Category	Load Factor	% AADT in Truck Category
2-Axle (Class 5)	0.50	8	2-Axle (Class 5)	0.50	8
3-Axle (Class 6, 8)	0.85	2	3-Axle (Class 6, 8)	0.85	2
4-Axle (Class 7, 8)	1.20	2	4-Axle (Class 7, 8)	1.20	2
5-Axle (Class 9, 11)	1.55	4	5-Axle (Class 9, 11)	1.55	4
>=6-Axle (Class 10, 12, 13)	2.24	0	>=6-Axle (Class 10, 12, 13)	2.24	0

Computed Design ESALS 10,127,509 Computed Historical ESALS 9,043,796

Back Close Calculate Next

Screen Clip 4-20

Step 4. Note that both the future and historical ESALs have been brought forward from the traffic data input. (Screen Clip 4-21) As before click **Select Location** to choose project location. Note that the Pavement Structure has two sections, the **Overlay** and the **Existing Structure**. The Existing Structure represents the in-situ structure. Guess an overlay thickness. It doesn't matter what the guess is since the software will start with 0.5 inches of asphalt concrete with the properties specified and increment by 0.5 inches until an acceptable overlay is obtained. The minimum overlay has been set to 1.5 inches. Once the data has been entered click **Analyze**.

Overlay Mechanistic Design - Project: Overlay Design (Filename: Overlay Design.xml)

Unit: US Customary Metric

Traffic Loads

Base AADT 20,000 Total Design ESALs 10,127,509
 Total Past ESALs 9,043,796

Select Location ANCHORAGE INTL AP

%Spring %Summer %Fall %Winter

17 42 8 33

Load Configuration

Select Tire Load

Tire Pressure 110 (psi) Tire Load 4500 (lbs)

Description ESAL

Load Locations (inch)	x	y	x	y
ESAL	0	13.5		
	0	0		
Evaluation Points (inch)	0	6.75		
	0	0		

Pavement Structure

Layer	Use TAI?	Thickness (inch)	Spring		Summer		Fall		Winter			
			Modulus (Ksi)	Poisson's Ratio	Modulus (Ksi)	Poisson's Ratio	Modulus (Ksi)	Poisson's Ratio	Modulus (Ksi)	Poisson's Ratio		
Overlay	Select	Asphalt Concrete (Ulm)	<input checked="" type="checkbox"/>	1.5	350	0.30	300	0.30	300	0.30	1200	0.30
Existing Structure	Select	Asphalt Concrete (Ulm)	<input checked="" type="checkbox"/>	4.0	350	0.30	300	0.30	300	0.30	1200	0.30
	Select	3-4% Asphalt Treated E		5	100	0.35	100	0.35	100	0.35	300	0.35
	Select	Select A P200<6%		24	35	0.40	40	0.40	40	0.40	90	0.40
	Select	Subgrade P200>30%		0	5	0.45	5	0.45	5	0.45	5	0.45

Asphaltic Layer Properties

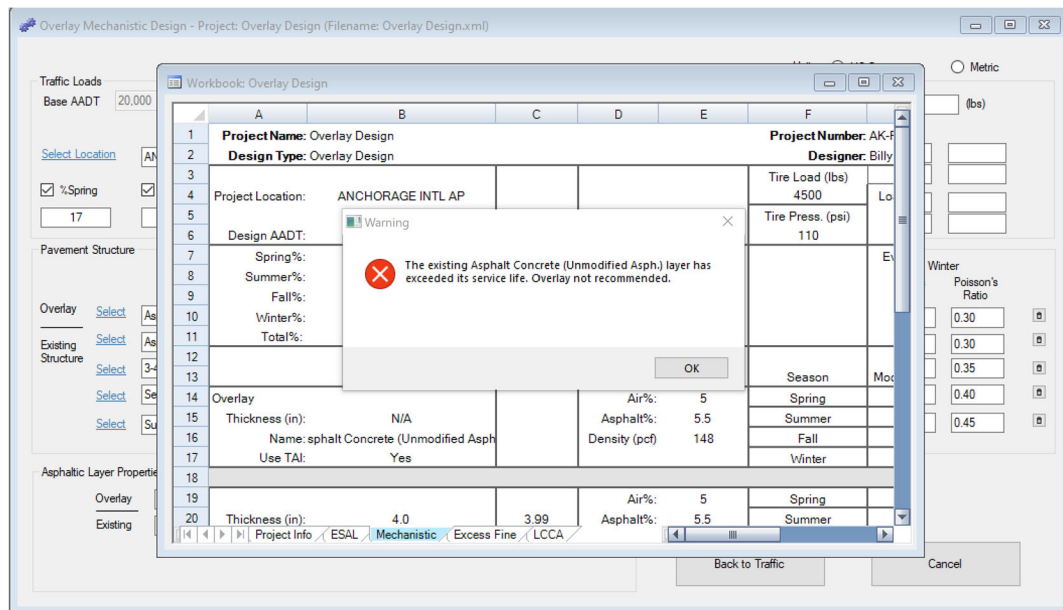
	Overlay	Existing
Asphaltic Layer Properties	Asph. Concrete (Ulm)	Asph. Concrete (Ulm)
% Air	5	5
% Asphalt	5.5	5.5
Density (pcf)	148	148

Analyze

Back to Traffic Cancel

Screen Clip 4-21

Step 5. In this case a warning appears indicating the existing asphalt concrete has already exceeded 80 % of its available life. (Screen Clip 4-22) Consequently, an overlay is not allowed as stated in Section 2.2.3 in Chapter 2. In this case consider removing and replacing the asphalt concrete layer or full depth reclamation.



Screen Clip 4-22

Click **OK**. Look at the column labeled “Past Damage (%)” in Clip 4-23. Note that the existing asphalt layer has already experienced about 82 % damage (i.e. exhausted 82% of its available life). The existing pavement has therefore already exceeded the allowable fatigue life allowed by the policy outline in Section 2.2.3 **For Designing Overlays of Existing Highway Pavements**. Click **Modules** in the ribbon then **Mechanistic Design** to return to the mechanistic input screen.

Alaska DOT&PF Flexible Pavement Design Software, Version 2.0 - Project: Overlay Design (Filename: Overlay Design.xml) - [Workbook: Overlay Design]

File Clipboard Modules View Help

	A	B	C	D	E	F	G	H	I	J	K	L	M	N
1	Project Name: Overlay Design		Project Number: AK-RD-1220(02)		Analysis Date: 4/8/2020		Project Status		Overlay not recommended					
2	Design Type: Overlay Design		Designer: Billy Bob McC		Unit: US Customary									
3	Project Location: ANCHORAGE INTL AP		Tire Load (lbs): 4500		Load Description: ESAL									
4	Design AADT: 20,000		Past Loadings		Design Loadings		Tire Press. (psi): 110		Load Loc (in): X: 0, Y: 0		Eval Loc (in): X: 0, Y: 0		ESAL	
5	Spring%: 17		1,537,445		1,721,677									
6	Summer%: 42		3,788,394		4,253,554									
7	Fall%: 8		723,504		810,201									
8	Winter%: 33		2,884,453		3,342,076									
9	Total%: 100		9,043,796		10,127,509									
10	Layer		Critical Z Coordinate (in)		Asphalt Properties		Season		Modulus (Ksi)		Poisson's Ratio		Tensile Micro Strain	
11	Overlay		3.99		Air%: 5 Asphalt%: 5.5 Density (pcf): 148		Spring		350		0.30		32	
12	Thickness (in): 4						Summer		300		0.30		38	
13	Name: sphalt Concrete (Unmodified Asph)						Fall		300		0.30		38	
14	Use TAL: Yes						Winter		1200		0.30		8	
15							Total Damage:						0.77	
16							Spring		350		0.30		90	
17							Summer		300		0.30		91	
18							Fall		300		0.30		91	
19							Winter		1200		0.30		30	
20							Total Damage:						83.70	
21							Spring		100		0.35		15.9	
22							Summer		100		0.35		17.3	
23							Fall		100		0.35		17.3	
24							Winter		300		0.35		14.4	
25							Total Damage:						5.37	
26							Spring		35		0.40		6.7	
27							Summer		40		0.40		7.5	
28							Fall		40		0.40		7.5	
29							Winter		90		0.40		5.8	
30							Total Damage:						7.57	
31							Spring		5		0.45		0.8	
32							Summer		5		0.45		0.8	
33							Fall		5		0.45		0.8	
34							Winter		5		0.45		0.4	
35							Total Damage:						9.54	
36							Spring		5		0.45		0.8	
37							Summer		5		0.45		0.8	
38							Fall		5		0.45		0.8	
39							Winter		5		0.45		0.4	
40							Total Damage:						9.54	

C:\AKDOT&PF\Alaska Flexible Pavement Design\My FPD Projects\Overlay Design.xml

Screen Clip 4-23

Step 6. For the purposes of this example, change the ATB -Stabilized Base Course thickness from 5.0 to 6.0 inches in the Existing Structure (Clip 4-24), then click *Analyze*.

The screenshot shows the 'Overlay Mechanistic Design' software interface. The title bar indicates the project is 'Overlay Design (Filename: Overlay Design.xml)'. The unit is set to 'US Customary'.

Traffic Loads: Base AADT is 20,000. Total Design ESALs is 10,127,509. Total Past ESALs is 9,043,796. The location is 'ANCHORAGE INTL AP'. Seasonal percentages are: %Spring (17), %Summer (42), %Fall (8), and %Winter (33).

Load Configuration: Tire Pressure is 110 (psi) and Tire Load is 4500 (lbs). Description is 'ESAL'. Load Locations (inch) are x: 0, y: 0. Evaluation Points (inch) are x: 0, y: 0.

Pavement Structure:

Layer	Use TAI?	Thickness (inch)	Spring		Summer		Fall		Winter	
			Modulus (Ksi)	Poisson's Ratio	Modulus (Ksi)	Poisson's Ratio	Modulus (Ksi)	Poisson's Ratio	Modulus (Ksi)	Poisson's Ratio
Overlay: Asphalt Concrete (Unm)	<input checked="" type="checkbox"/>	1.5	350	0.30	300	0.30	300	0.30	1200	0.30
Existing Structure: Asphalt Concrete (Unm)	<input checked="" type="checkbox"/>	4.0	350	0.30	300	0.30	300	0.30	1200	0.30
Existing Structure: 3-4% Asphalt Treated E		6	100	0.35	100	0.35	100	0.35	300	0.35
Existing Structure: Select A P200<6%		24	35	0.40	40	0.40	40	0.40	90	0.40
Existing Structure: Subgrade P200>30%		0	5	0.45	5	0.45	5	0.45	5	0.45

Asphaltic Layer Properties:

		% Air	% Asphalt	Density (pcf)
Overlay	Asphalt Concrete (U)	5	5.5	148
Existing	Asphalt Concrete (U)	5	5.5	148

Buttons: Analyze, Back to Traffic, Cancel.

Screen Clip 4-24

Step 7. Interpret the results (Clip 4-25). The total damage in the "Total Damage (%)" column is less than 100%. Note the damage in the overlay is almost zero. This is because this layer is subjected to small tensile strains. The purpose of the overlay is to reduce the tensile strain in the existing layer such that the number of cycles remaining is increased enough to extend the life by the design amount. Thus, the total damage in the existing asphalt layer will be almost 100% at the end of the overlay design period. As a result, future overlays will not be allowed. This will become important to consider in the Life Cycle Costing Module.

Alaska DOT&PF Flexible Pavement Design Software, Version 2.0 - Project: Overlay Design (Filename: Overlay Design.xml) - [Workbook: Overlay Design]

File Clipboard Modules View Help

	A	B	C	D	E	F	G	H	I	J	K	L	M	N										
1	Project Name: Overlay Design				Project Number: AK-RD-1220(02)				Analysis Date: 4/8/2020				Project Status											
2	Design Type: Overlay Design				Designer: Billy Bob McC				Unit: US Customary				All layer damages less than 100%.											
3	Project Location: ANCHORAGE INTL AP		Past Loadings		Design Loadings		Tire Load (lbs)		Load Description: ESAL															
4							4500																	
5	Design AADT: 20,000						Tire Press. (psi)		Load Loc (in)															
6							110		X: 0															
7	Spring%: 17		1,537,445		1,721,677				Y: 0															
8	Summer%: 42		3,798,394		4,253,554				X: 0															
9	Fall%: 8		723,504		810,201				Y: 0															
10	Winter%: 33		2,984,453		3,342,078																			
11	Total%: 100		9,043,796		10,127,509																			
12	Layer		Critical Z Coordinate (in)		Asphalt Properties		Season		Modulus (Ksi)		Poisson's Ratio		Tensile Micro Strain		Compressive Stress (psi)		Million Cycles to Failure		Past Damage (%)		Future Damage (%)		Total Damage (%)	
14	Overlay		2.99		Air%: 5		Spring		350		0.30		24		3163.64				0.05		0.05			
15	Thickness (in): 3				Asphalt%: 5.5		Summer		300		0.30		31		1681.18				0.25		0.25			
16	Name: sphalt Concrete (Unmodified Asph				Density (pcf) 148		Fall		300		0.30		31		1681.18				0.05		0.05			
17	Use TAL: Yes						Winter		1200		0.30		5		198415.26				0.00		0.00			
18															Total Damage:				0.35		0.35			
19															30.85		19.98		5.58		25.56			
20	Thickness (in): 4.0		6.99		Air%: 5		Spring		350		0.30		99		33.75		44.59		12.60		57.19			
21	Name: sphalt Concrete (Unmodified Asph				Asphalt%: 5.5		Summer		300		0.30		101		33.75		8.49		2.40		10.89			
22	Use TAL: Yes				Density (pcf) 148		Fall		300		0.30		101		416.36		2.68		0.80		3.48			
23							Winter		1200		0.30		33		Total Damage:		75.74		21.38		97.12			
24															30.85		19.98		5.58		25.56			
25	Thickness (in): 6		7.01		Air%: 5		Spring		100		0.35		20.3		71.10				2.42		2.42			
26	Name: 3-4% Asphalt Treated Base				Asphalt%: 5.5		Summer		100		0.35		22.0		54.99				7.74		7.74			
27	Use TAL: Yes				Density: 148		Fall		100		0.35		22.0		54.99				1.47		1.47			
28							Winter		300		0.35		18.7		3343.62				0.10		0.10			
29															Total Damage:		11.73		11.73					
30	Thickness (in): 24		13.01		Air%: 5		Spring		35		0.40		7.1		72.78				2.37		2.37			
31	Name: Select A P200<6%				Asphalt%: 5.5		Summer		40		0.40		7.8		81.57				5.21		5.21			
32	Use TAL: Yes				Density: 148		Fall		40		0.40		7.8		81.57				0.99		0.99			
33							Winter		90		0.40		6.1		2535.76				0.13		0.13			
34															Total Damage:		8.70		8.70					
35	Thickness (in): 0		37.01				Spring		5		0.45		0.8		68.12				2.53		2.53			
36	Name: Subgrade P200>30%						Summer		5		0.45		0.8		72.53				5.86		5.86			
37							Fall		5		0.45		0.8		72.53				1.12		1.12			
38							Winter		5		0.45		0.4		602.91				0.55		0.55			
39															Total Damage:		10.06		10.06					

C:\AKDOT&PF\Alaska Flexible Pavement Design\My FPD Projects\Overlay Design.xml

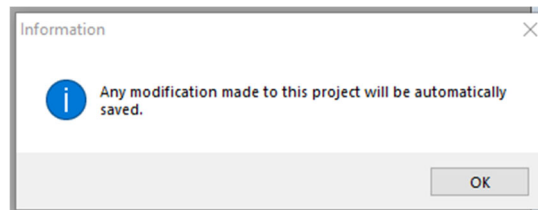
Screen Clip 4-25

4.8 Saving, Recalling, and Modifying Files

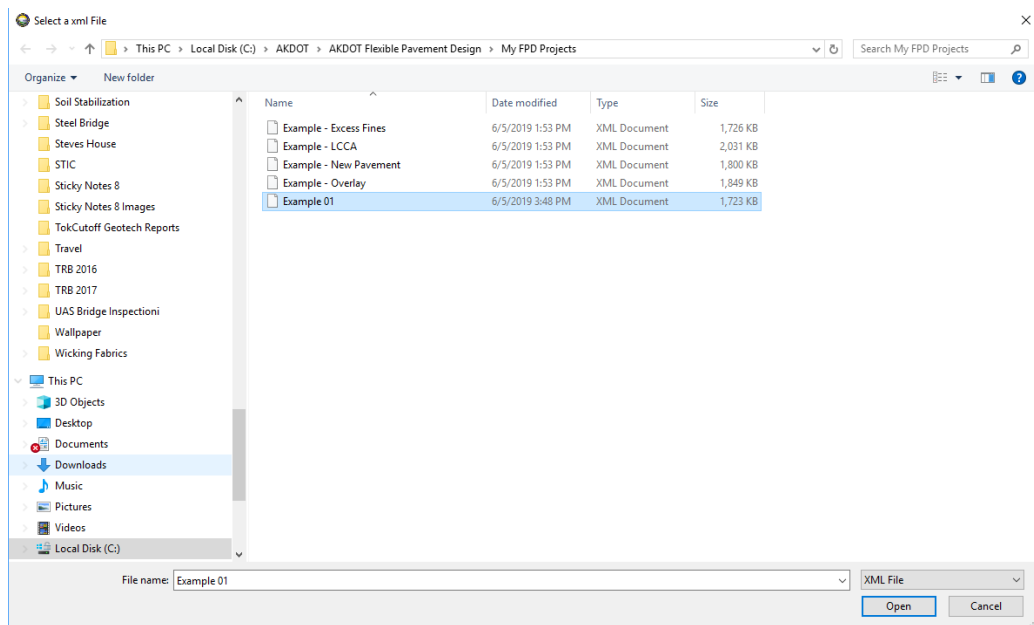
AKFPD software is designed to ensure no work is lost. The software uses a series of project specific spreadsheets compiled into an XML document used to save data. These spreadsheets are not available to the user except through AKFPD. Each time the user moves to a new screen or analyzes the data, the data are saved to the appropriate spreadsheet. When the user leaves the software, a warning appears requesting the user confirm the data have been saved.

At any time, a copy of the current file can be created and made current by clicking on **File** then **Copy To** on the ribbon at the top and entering a new name making sure the name is unique.

There are two ways of recalling an existing file. By clicking the **Open Existing Design** button on the landing page shown in Screen Clip 4-1, Screen Clip 4-26 will appear warning the user that modifications will be saved. Saved data will overwrite the existing data. The warning screen will be replaced with a list of available files (Screen Clip 4-27).

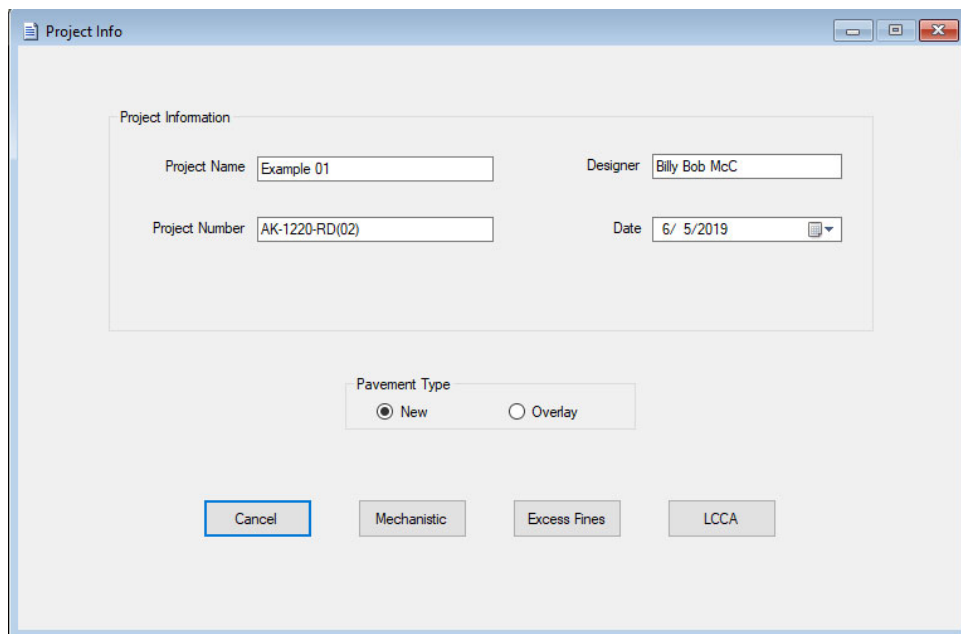


Screen Clip 4-26



Screen Clip 4-27

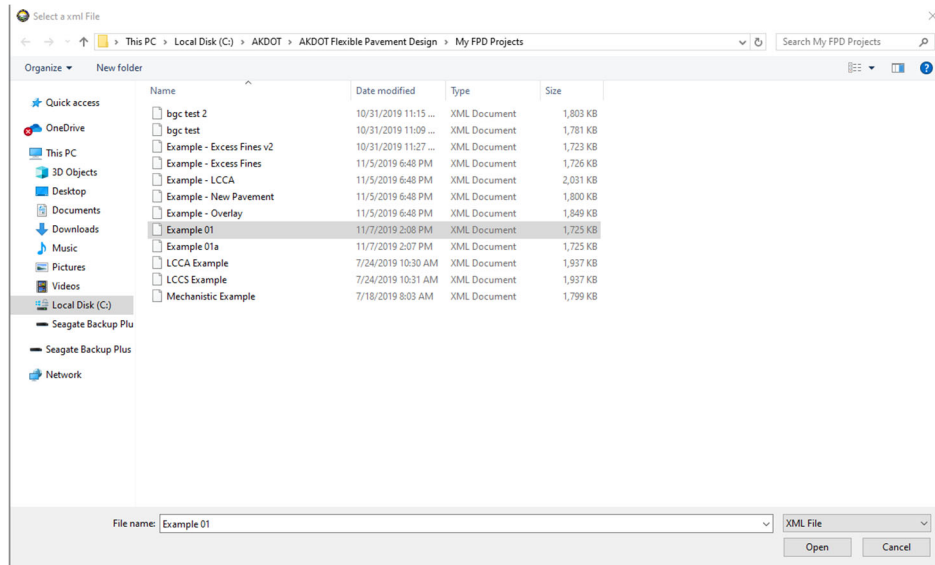
Type in or select “Example01” and then click *Open*. The Project Info page will open. (Screen Clip 4-28)



Screen Clip 4-28

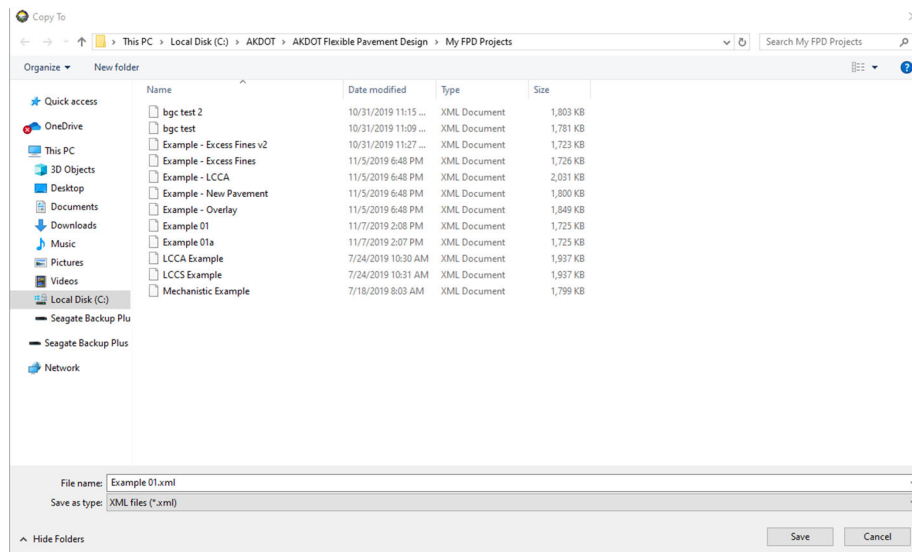
At this point, any changes to the data will overwrite the data in that file causing the loss of previously stored data.

If the user wishes to save the existing data, select *Open a Copy of Existing Design* from the opening screen (Clip 4-1). Available files will be displayed (Screen Clip 4-29).



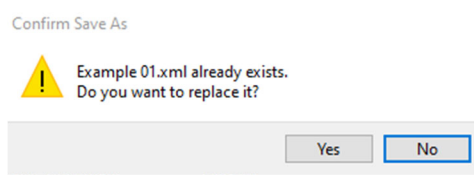
Screen Clip 4-29

Select the desired file and click *Open*, in this case Example 01. The screen shown in Screen Clip 4-30 will appear.



Screen Clip 4-30

Note that this screen looks much like the previous screen except the button in the lower right has been changed from “Open” to “Save”. Give the filename a unique name such as “Example 01a” indicating this is a variation of Example 01. If a unique name is not selected or the user forgets to change the name a message will appear as shown in Screen Clip 4-31.



Screen Clip 4-31

Select *No* to go back to the file list to revise the name. Select *Yes* to accept the name and the data in that file will

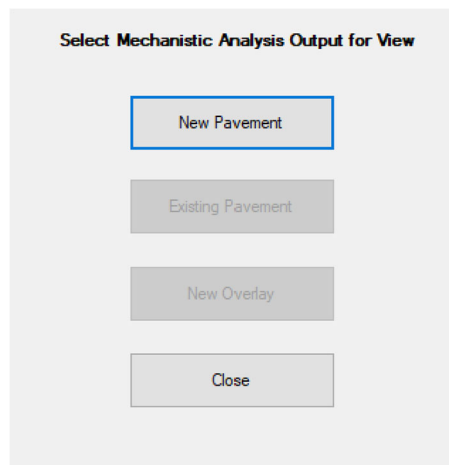
be replaced with the new data. From this point forward any changes to the file will overwrite existing data. If the user does not wish to make changes to the file either click **File**, then **Exit** and restart the program or click **File** then **Close All**.

4.9 Advanced Users

The AKFPD mechanistic module provides the advanced user with a summary of all stresses and strains in the pavement structure at the desired location. These points are defined as the top of unbound layers and at the bottom of bound layers. Consequently, the stresses and strains reported can be controlled by the definition of layers. The steps to produce the report are the same as for a mechanistic-empirical design with one exception. From the report shown in Screen Clip 4-17 click *Modules* followed by *Mechanistic Responses (WESLEA Output)*. Select the desired report as shown in Screen Clip 4-32. Note that only *New Pavement* option is available if there is no overlay.

A .csv spreadsheet with the same name as the current analysis will be created in the “C:\AKDOT&PF\Alaska Flexible Pavement Design\My FPD Projects” subdirectory. Note, this file may be opened directly into Excel at any time. The spreadsheet contains the stresses and strains at the specified locations as shown in Screen Clip 4-33. The results shown in Screen Clip 4-33 are the partial results in the interest of saving space. The user may wish to save the file.

A full discussion of the results is not provided here since it is assumed that anyone wishing to use these results has a well-founded understanding of soil mechanics as they apply to mechanistic pavement design.



Screen Clip 4-32

The screenshot displays a Microsoft Excel spreadsheet titled "Overlay Design_Existing.cov". The spreadsheet is divided into several sections:

- INPUT DATA (Rows 1-15):** Contains material properties for different layers. Row 3 lists Modulus (psi) for Spring, Summer, Fall, and Winter. Row 4 lists Poisson's Ratio for the same seasons. Row 5 lists Thickness (in) for layers 1 through 4. Row 6 lists Load #, x (in), and y (in) for layers 1 through 4.
- LINEAR ELASTIC MECHANISTIC RESULTS (Rows 16-49):** A large table with columns for Season, Eval_X, Eval_Y, Eval_Z, Layer #, StressXX, StressYY, StressZZ, StressXY, StressXZ, StressYZ, StrainXX, StrainYY, StrainZZ, DigItX, DigItY, DigItZ, StressPrin1, StressPrin2, StressPrin3, StrainPrin1, StrainPrin2, and StrainPrin3. Each row represents a specific evaluation point.

Screen Clip 4-33

5 Design Input—Materials Properties for Mechanistic Design

- 5.1. Characterizing Materials within the Pavement Structure as Input for the AKFPD
- 5.2. Materials Properties—Recommended Presumptive Values for New Construction and Reconstruction Designs
- 5.3. Materials Properties—Laboratory Testing to determine Values for New Construction and Reconstruction Designs
- 5.4. Materials Properties—Values determined from Field Tests for Overlay Designs

5.1 Characterizing Materials within the Pavement Structure as Input for the AKFPD

This manual provides methods for determining the thickness of pavement structural layers in new construction projects and overlay for pavement rehabilitation projects. Design procedures presented here require design inputs that accurately represent real loadings and materials conditions. Considerable engineering judgment is required to properly select design inputs. In most cases, the designer will work with the Materials Section to obtain these values.

Aggregate layers are characterized in terms of their elastic properties. Specifically, these properties are repeated load, i.e., “dynamic” elastic modulus (a measure of stiffness called resilient modulus and noted by the symbol “ M_R ”) and Poisson’s ratio (deformational characteristic, noted by the symbol “ μ ”). M_R and μ are defined in Section 4.3.1 where program input values are discussed in detail.

For designing most new pavement structures, mechanistic properties of the various layers are often obtained from DOT&PF-approved tables. These tables provide reasonably accurate estimates of mechanistic properties for aggregate materials based on P_{200} content.

Values for M_R can also be obtained through laboratory testing or through the process of “backcalculation”, using data collected in the field by deflection testing equipment. Derivation of mechanistic properties from laboratory tests and backcalculation is applicable mostly to the design of overlays for existing pavements. Laboratory test methods recommended for determining the modulus values of asphalt concrete and soil/aggregate materials are, respectively, ASTM D7369 Resilient Modulus for Asphalt Mixtures and AASHTO T307 Resilient Modulus for Soils and Aggregate.

5.2 Materials Properties—Recommended Presumptive Values for New Construction and Reconstruction Designs

5.2.1 Resilient Modulus (M_R) Values

Usually layer resilient modulus values (M_R) for materials used in new construction are not known. The designer may use presumptive modulus values as shown in Tables 5-1 and 5-2. Material containing excess fines may cause significant thaw weakening of the overlying pavement structure. As a design quality control measure, seek concurrence in the selected modulus values from regional or headquarters pavement materials experts.

Table 5-1 Pavement Layer Moduli (ksi)

Material Type	P ₂₀₀	Spring	Summer & Fall	Winter
Asphalt Concrete (Modified Asphalt)	—	450	400	1,200
Asphalt Concrete (Unmodified asphalt)	—	350	300	1,200
Aggregate Base	<6%	40	50	100
Selected Material Type A	<6%	35	40	90
Subbase F	<6%	35	40	90
Selected Material Type B	<10%	20	30	50
Selected Material Type C & Subgrade	<30%	10	10	10

Table 5-2 Pavement Layer Moduli (with excess fines) (ksi)

Material Type	P ₂₀₀	Spring	Summer &	Winter
Aggregate Base	<10%	20	30	50
Selected Material Type A, B	<10%	20	30	50
Selected Material Type B	<18%	10	10	10
Selected Material Type C	>30%	10	10	10
Subgrade	>30%	5	5	5

5.2.2 Resilient Modulus Values for Stabilized Base Course Materials

Table 5-3 shows presumptive modulus values for stabilized base course materials.

Table 5-3 Stabilized Base Course Moduli (ksi)

Material Type	Spring	Summer & Fall	Winter
CABC ; RAP (50:50) ¹	80	90	120
EATB, 3% Emulsion ¹	100	100	300
ATB, 4% Asphalt ²	200	200	600
Foamed Asphalt Stabilized Base ¹	110	100	400

1. lightly bound: use Ullidtz
2. heavily bound: use TAI

Design with lightly bound asphalt-treated base courses (containing < 4% asphalt cement) using the mechanistic design procedure and by controlling the vertical compression stress at the top of the treated bases and horizontal tensile strain at the bottom of the asphalt concrete pavement layer.

Design with heavily bound asphalt-treated base courses (containing ≥ 4% asphalt cement) using the mechanistic

design procedure and by controlling the horizontal tensile strain at the bottom of the asphalt-treated base courses and the asphalt concrete pavement layers.

Verify the validity of presumptive modulus values with regional or headquarters Materials section personnel.

5.2.3 Poisson's Ratio Values

Table 5-4 shows recommended Poisson's ratios for various pavement structure materials. As a design quality control measure, seek concurrence in the selected Poisson's ratio values from regional or headquarters pavement materials experts.

Table 5-4 Poisson's Ratio Values

Material Type	Poisson's Ratio (μ)
Asphalt Concrete	0.30
Aggregate Base	0.35
Selected Material Types A and B	0.40
Selected Material Type C	0.45
Subgrade Materials	0.45

5.3 Materials Properties—Laboratory Testing to determine Values for New Construction and Reconstruction Designs

Much of the mechanistic pavement design work done for DOT&PF relies on presumptive M_R and Poisson's ratio values (see Section 5.2), although backcalculated values for M_R are often obtained based on deflection test data for overlay design work (see Section 5.4). This section provides guidance for those rare design situations where laboratory testing may be required. Testing might be required, for example, for designs involving unusual or experimental material types not listed in the Section 5.2 tables.

5.3.1 Resilient Modulus Values

For determining M_R of unbound soils (including subgrade soils) or unbound or lightly bound bases or subbase materials, use AASHTO T307 *Test for Determining the Resilient Modulus of Soils and Aggregate Materials*

For determining M_R of asphalt-bound materials such as asphalt concrete or other heavily asphalt-bound base materials, use ASTM D7369 *Standard Test Method for Determining Resilient Modulus of Bituminous by Indirect Tension Test*.

For heavily bound pavement, base, or subbase materials with a cementing agent other than asphalt, consult regional or headquarters Materials personnel for a recommended test method.

5.3.2 Poisson's Ratio Values

For all normal pavement design work the designer will use presumptive Poisson's ratio values. Table 5.4 contains recommended Poisson's ratios for common pavement structure materials. As a design quality control measure, the

designer should seek concurrence in the selected Poisson's ratio values from regional or headquarters pavement design experts. Also solicit materials expertise for determining Poisson's ratio values for unusual materials.

5.4 Materials Properties—Values determined from Field Tests for Overlay Designs

5.4.1 Backcalculation Program

DOT&PF recommends the backcalculation program ELMOD, developed by Dynatest Consultants Inc. Backcalculation of layer modulus values should be done only by personnel with experience in performing backcalculations.

The minimum asphalt concrete thickness for which a modulus can be backcalculated is 3.5 inches, because of the plate size on the falling weight deflectometer. For thinner layers, the asphalt concrete pavement must be cored and tested in the laboratory or presumptive moduli values in Table 5-1 can be used.

5.4.2 Deflection Testing

DOT&PF currently uses a falling weight deflectometer (FWD) to measure the dynamic deflection for pavement rehabilitation design. Seek advice from regional or headquarters pavement design experts to schedule and perform such testing.

5.4.3 Selecting Test Locations

Working with the Materials Section, select test locations as an average representation of the present surface condition and where the original pavement structure is free from patching. If alligator cracking is not prevalent, adjust test locations to avoid the cracking. If alligator cracking cannot be avoided, note it in the data. If alligator cracking is prevalent, assume a reduced modulus (contact the regional materials engineer). Select a test section to represent each type of terrain the project passes through.

Choose a minimum of 20 evenly spaced deflection test locations within a selected mile. If the pavement structure is thought to be frost-susceptible, consider increasing the number of tests. It is preferable to mark the test locations with paint for repeated testing of exact points in subsequent weeks. White painted markings on the centerline have been found to last longer and are easier to locate by field crews.

5.4.4 When to Test

Perform FWD deflection testing during the spring thaw period when pavement strength is at a minimum. A weekly set of deflection tests should begin when the pavement structure begins to thaw and must continue through the period when peak deflections occur. Perform at least one set of readings in the summer and another in the fall.

Base the decision of when to begin testing on actual field evidence, such as small test pits, frost tubes, or soil temperature data, if available. If deflection testing cannot be performed during the peak period, contact the regional materials engineer for a seasonal adjustment factor.

Testing during periods when night temperatures are below freezing should not begin before late morning. This is to prevent the bridging effect of the temporarily frozen surface layer from depressing the true rebound deflection readings.

5.4.5 Testing Procedure

See the falling weight deflectometer operation manual available from the Materials Engineer.

5.4.6 Safety Equipment and Precautions

Because of frequent stops when FWD testing, take all necessary safety precautions. Use appropriately attired flaggers as necessary to control traffic. High-level warning devices, such as vehicle-mounted arrow boards, are best (see part 6 of the *Manual on Uniform Traffic Control Devices for Streets and Highways*,⁽²⁰⁾ for detailed procedures). One or two vehicles with warning signs will be required, depending on traffic levels and sight distances.

6 Design Input—Equivalent Single Axle Loads

- 6.1 Introduction
- 6.2 Calculate the Load Factor for Each Vehicle Category
- 6.3 Calculate Design ESALs
- 6.4 Historical ESALs

6.1 Introduction

This chapter applies to designing normal pavement structures for highway projects. Highway pavement structures are designed to withstand the design number of standardized loadings derived from a known mix of truck-types in the traffic stream. Standardized vehicle loadings used in highway design are termed “equivalent single axle loads” (ESALs), i.e., the pavement structure will be subjected to a specific number of ESALs during its design life. This chapter describes how to calculate standardized ESAL loadings for the project. To facilitate the ESAL computation, planning personnel will provide information concerning vehicle loadings as well as traffic growth rate and traffic lane distribution data. Similar concepts involving standardized vehicle loadings are used to calculate design aircraft loadings for aviation projects, although that methodology is not presented here.

With every application of an ESAL, the surface of a pavement structure, that structure experiences a quantifiable amount of structural damage. In other words, every ESAL application subtracts a finite amount of life from the pavement structure’s available life. If the project involves construction of a new pavement structure or replacing an old one, one ESAL value will be needed for design, i.e., future ESALs (ESALs estimated for the design life of the new pavement structure). If the project involves placing an overlay on an existing pavement layer, two ESAL values will be required: (1) historical ESALs (ESALs accumulated on the existing pavement structure during its past service life), and (2) future ESALs.

Categories of truck-type vehicles are defined in terms of standard ESAL loadings for each category. Only medium and large trucks are assigned ESAL equivalency. Automobiles, pickup trucks, and other small vehicles do negligible damage to the pavement structure. Consequently, they are not considered in the structural design. An old rule-of-thumb is that pavement structural damage done by the passage of a single large truck is equivalent to that done by about 9,000 automobiles.

ESAL Defined: One ESAL represents the loading that produces an amount of damage to the pavement structure equivalent to one pass of a single 18,000-pound, dual-tire axle with all four tires inflated to 110 psi.

ESAL Truck Category and Load Factor Defined: DOT&PF defines five truck categories (2-axle, 3-axle, 4-axle, 5- axle, and ≥ 6 -axle). Trucks assigned to the 2-axle category have one front axle and one rear axle. Trucks assigned to the 3-axle category have one front axle and a tandem rear axle set. Trucks assigned to the 4-axle category are “semi” tractor/trailer combinations having one front axle on the tractor, a tandem set of driver axles on the tractor, and one axle at the rear of the trailer. Trucks assigned to categories higher than the 4-axle type are simply tractor/ trailer or tractor/multi-trailer combinations having a total of two or more trailer axles.

The truck categories represent a simplification of the FHWA truck classifications. Table 6-1 provides a summary of the DOT&PF truck categories and the FHWA truck classifications.

Each of the five truck categories, is assigned an ESAL equivalency based on scalehouse data. The assigned ESAL equivalency is termed the **load factor**—and every truck in that category is assigned that load factor.

Table 6-1 Summary of DOT&PF Truck Categories and FHWA Truck Classification

DOT&PF Truck Categories	FHWA Truck Classification	Load Factors
2-axle truck	Class 5	0.50
3-axle truck	Class 6 and 8	0.85
4-axle truck	Class 7 and 8	1.20
5-axle truck	Class 9 and 11	1.55
6 or more axle truck	Class 10, 12, and 13	2.24

6.2 Calculate the Load Factor for Each Vehicle Category

This section provides a somewhat generalized description of how a load factor for each DOT&PF truck category is determined. Even though load factor data will be supplied to the designer, basic load factor computations are briefly discussed here to promote more thoroughly understanding of the concept of standardized loadings.

Load factor is defined as the average number of ESALs associated with each truck of a truck size category. Load factors for all categories of truck size are usually calculated by regional planning sections and will be supplied to when the designer requests traffic data for your project. Load factors are necessary input data for the design ESAL calculations described in Section 6.3. One ESAL equivalent loading is defined for various axles or axle group configurations loaded as shown in Table 6.-2.

An axle is considered part of an axle group when it is less than 8 feet from another axle or group. For example, if two single axles are less than 8 feet apart, they are considered a tandem axle. If a single axle is less than 8 feet from a tandem axle, the three are considered a tridem axle.

Load factors are determined from scalehouse weight data obtained from many trucks. At the scalehouse, axle and axle group weights are sampled from numerous trucks representing each truck category. Each truck to be weighed is first assigned to a category. Then each axle and axle group (single, dual, tandem, etc.) for that truck is weighed individually. An ESAL value is calculated for each axle and axle group (using the following ESAL equation).

Table 6-2 ESAL Load Equivalent for Axle Groupings

Type of Axle or Axle Group	Load Equivalent to One ESAL (kips)
single tire single axle	12
dual tire axle	18
tandem axle group	32
tridem axle group	48
4 or more axle group	48

The ESAL equation from truck axle weight and axle spacing for all axles or groups is:

$$ESAL = \left(\frac{W_1}{W_2} \right)^{4.3}$$

where:

W_1 = weight in kips of the loaded axle or axle group.

W_2 = weight in kips of the standard axle or axle group (see Table 6-2)

The total ESAL value, i.e., load factor, for that truck is the sum of ESAL values for all axles and axle groups of that truck.

The load factor for each truck category is determined by averaging the total ESAL values for all trucks in that category. See Table 6.1

6.3 Calculate Design ESALS

Estimate total ESALS for the design period by projecting forward the construction year ESALS for each truck category.

6.3.1 Outline of Computation Steps

1. Obtain basic information from traffic/planning personnel. These data are based on studies of scalehouse data, weigh-in-motion data, traffic counts, administrative studies/projections, and miscellaneous observations.
 - AADTs for base year (both directions). Base year data is a best estimate of design data for the design location and for a specific year—usually for the year that the project is being designed. Using compound growth calculations, a past year or future year, AADT may be calculated from the base year AADT. The base year must be identified by whoever supplies the AADT data.
 - Traffic growth rate (% per year) from base year
 - Forward

- Backcast (if applicable for historical ESAL calculations)
- Number of driving lanes
- Lane split(s)
 - Determine design lane as lane having highest portion of lane split
- Load factor information for each truck category
- Percent of AADT in each truck category

To facilitate collecting previously indicated data, submit a request to your regional planning section. Traffic reports and maps are also available online at:

http://dot.alaska.gov/stwdplng/transdata/traffic_maps_home.shtml.⁽²¹⁾ Large projects may require more than one pavement structural design which, in turn, requires multiple sets of traffic data. Exercise engineering judgment to determine if more than one pavement design is required for your project. In general, road segments within a project that are expected to have significantly different traffic volumes and/or vehicular mix may warrant separate pavement designs.

2. For the year of construction, determine number of ESALs in each truck category based on AADT of the design lane.
 - a. Calculate total AADT for year of construction using $i_{B\ to\ D}$ and “single payment” compound amount factor $P = (1 + i_{B\ to\ D})^n$

where:

n = year of construction – base year

$i_{B\ to\ D}$ = growth rate from base year to last year of design period

Construction year AADT = (base year AADT) × (P); (values for i and n are obtained from the traffic section).

- b. Calculate construction year AADT in the design lane.

Construction year design lane AADT = (construction year total AADT) × (% AADT in design lane/100)

- c. For each truck category, calculate the construction year ESALs in the design lane. Use the following equation for each truck category.

Construction year ESALs for truck category = (construction year AADT in design lane) × (%total AADT in truck category/100) × (load factor for truck category) × 365

- d. Calculate the total of construction year ESALs for all truck categories.

Total construction year ESALs = Σ construction year ESALs for every truck category

3. Calculate total number of ESALs for each truck category accumulated from the year of construction through the end of the design period.

Calculate total ESALs for design. Project total construction year ESALs for all truck categories (calculated in step 2d) forward to end of design period using $i_{B\ to\ D}$ and “uniform series,” compound amount factor $[(1 + i)^n - 1] / i$

where:

n = last year of design period – construction year

Total design ESALs = (total construction year ESALs) × (compound amount factor)

6.3.2 An Example (Calculate Design ESALs Forward in Time)

Input Data:

The project will be constructed in 2021 and has a 15-year design life (end year of design period = 2036). Design work was done in 2019, the base year chosen for estimating the AADT growth rate.

For this example, Table 6.3 shows an example of basic traffic data obtained,

Table 6-3 Example Traffic Data

Base Year: 2019	Construction Year: 2021	Lane Distribution (% AADT)	
Base Year AADT: 1,600		Lane 1	60 (50 Historic)
AADT Growth Rate:		Lane 2	40 (50 Historic)
Forward (%/year): 2.5	End Year: 2036		
Past (%/year): 1.6	Begin Year: 2006		

Truck Category	Load Factor (ESALS/Truck)	% of total AADT
2-axle	0.50	2
3-axle	0.85	4
4-axle	1.20	4
5-axle	1.55	3
≥6-axle	2.24	1

Calculations:

Design ESAL computations for this example follow the Section 6.3.1 outline.

Step 1

Already completed with the collection of the indicated input data (use Table 6-3).

Step 2

- Construction year total AADT = (base year total AADT) × $(1 + i_{BtoD})^n = (1600) × (1 + 0.025)^2 = 1,681$
- Construction year AADT in design lane = (construction year total AADT) × (% AADT in design lane/100) = $(1681) × (0.6) = 1,009$
- Construction year ESALs for specific truck category = (construction year AADT in design lane) × (% of total AADT in truck category/100) × (load factor for truck category) × 365

See rows of computations in Table 6-4.

- Total construction year ESALs = Σ of construction year ESALs for all truck categories = $3,683 + 12,522 + 17,678 + 17,125 + 8,250 = 59,258$

See summation in Table 6-4.

Table 6-4 Computation of Construction Year ESALs

(1)	(2)	(3)	(4)
Truck Category	Load Factor (ESALS/Truck)	% of total AADT	ESALs in Construction Year
2-axle	0.5	2	3,683
3-axle	0.85	4	12,522
4-axle	1.2	4	17,678
5-axle	1.55	3	17,125
≥6-axle	2.24	1	8,250
Total ESALs in Construction Year:			59,258

$$\text{Truck Category ESALs in Construction year(4)} = (1) \times (2) \times (3) \times 3.65$$

$$\begin{aligned} \text{Total design ESALs} &= (\text{total construction year ESALs} \times [(1 + i_{B \text{ to } D})^n - 1] / i_{B \text{ to } D}) \\ &= (59258) \times \frac{[(1 + .025)^{15} - 1]}{.025} = 1,062,610 \end{aligned}$$

6.4 Historical ESALs

For rehabilitation projects, the historical ESALs are required for the analysis of the remaining pavement life. If historical AADTs exist for each year since the last pavement construction project, calculate ESALs for each year to the present—using methods in Section 6.3. The total historical ESAL is the sum of the past yearly ESALs. If load factors for past years are not available, use base year (present) load factors, and perhaps adjust based on careful consideration and judgment. Past growth rate information must be obtained from planning personnel. Request that historical traffic data if historical ESALs need to be calculated. After obtaining the historical growth rate from planning and collecting or estimating the other input data, calculate historical ESALs using the same steps outlined in Section 6.3.1.

6.4.1 An Example (ESAL Calculation Extended Backwards and Based on Previous Example)

Input Data:

Input data for calculating historical ESALs are like those used for calculating future ESALs (see section 6.3.2 example) except that data come from the construction year and years prior to that time.

The project will be constructed in 2021 and the backward (backcast) projection of ESALs will extend to the previous surfacing application in 2006, i.e., the historical construction year.

Use data pertaining to historical ESAL calculations in Table 6-3. Since construction work will be done in 2021, this now becomes the “base” year for estimating the AADT in 2006.

AADT for (2021) = 1,681 (from previous example)

The traffic growth rate from the last historical construction event (2006) to the original base year (2019) of design

period was 1.6% (this historical growth rate is identified as $i_{H to C}$)

Load factors and percent of AADT in each truck category are the same data used in the previous example for projecting ESALs into the future (a simplification for this example). For historical ESAL projections, use actual historical truck category data if they are available.

Calculations:

Historical ESAL computations generally follow the method outlined in Section 6.3.1.

Step 1

Already completed with the collection of the indicated input data (use Table 6.3).

Step 2

$$\begin{aligned}
 \text{a. Historical construction year total AADT} &= (\text{construction year AADT}) \times \frac{1}{(1+i_{H to C})^n} \\
 &= (1681) / (1 + 0.016)^{15} = 1325
 \end{aligned}$$

Where:

- $i_{H to C} = 0.016$
- $n = \text{construction year} - \text{historical construction year} = 2021 - 2006 = 15$
- b. Historical construction year AADT in design lane = (historical construction year total AADT) × (historical % of total AADT in truck lane/100) = (1325) × (0.5) = 663
- c. Historical construction year ESALs for specific truck category = (historical construction year AADT in design lane) × (historical % of total AADT in truck category/100) × (historic load factor for truck category) × 365

See Table 6-5

$$\begin{aligned}
 \text{d. Total construction year ESALs} &= \Sigma \text{ of construction year ESALs for all truck categories} \\
 &= 2,420 + 8,228 + 11,616 + 11,253 + 5,421 = 38,938
 \end{aligned}$$

See summation in Table 6-5.

Table 6-5 Historical Construction Year ESAL Calculations

	(1)	(2)	(3)	(4)
Truck Category	Load Factor (ESALS/Truck)	% of total AADT	Historical Lane AADT	ESALs in Historical Construction Year
2-axle	0.5	2	663	2,420
3-axle	0.85	4	663	8,228
4-axle	1.2	4	663	11,616
5-axle	1.55	3	663	11,253
≥6-axle	2.24	1	663	5,421
Total ESALs in Historical Construction Year:				38,938

$$\text{Truck Category ESALs in Construction year(4)} = (1) \times (2) \times (3) \times 3.65$$

Step 3

$$\begin{aligned} \text{Total historical ESALs} &= (\text{total historical construction year ESALs}) \times [(1 + i_{H \text{ to } C})^n - 1] / i_{H \text{ to } C} \\ &= (39,938) \times [(1 + 0.016)^{15} - 1] / 0.016 = 654,247 \end{aligned}$$

7 Surface Course and Pavement Layers Selection Guide

- 7.1. General Considerations
- 7.2. Unstable Embankments
- 7.3. Available Surfacing Types
- 7.4. Stabilized Layers

7.1 General Considerations

The selection of the surface course for a road or highway project should be based on initial cost, annual costs, embankment stability, and other considerations. Variables to consider for initial cost include mobilization, materials, materials availability, and other design and construction costs. Variables for annual costs include maintenance and user costs. Also, consider traffic speed and volume, truck volume, rural versus urban, severity of roughness, embankment stability, expected life, rainfall, temperature, and type of maintenance equipment available. Consult the regional maintenance section when selecting surface courses.

7.2 Unstable Embankments

The thawing of ice-rich permafrost foundation soils is the main cause of embankment instability in Alaska. Consolidation of thick organic soils that have not been adequately surcharged can also cause instability. Because frozen soils consolidate during thawing, thaw settlement instability problems progress at a varying rate from year to year. Sometimes an embankment can take more than 50 years to stabilize. The severity of the consolidation depends on the depth and volume of ice in the soil. Variations in ice content lead to inconsistent consolidation, resulting in differential settlement of the driving surface.

The life of a surface course on unstable embankments should closely match the life of the embankment and foundation. In rural areas, this may result in the selection of a double-layer asphalt surface treatment or a high-float asphalt surface treatment.

In urban areas, the selection of a surface course depends on traffic volume and speed, severity of the pavement roughness, possibility of vehicle damage due to airborne aggregate, and the possibility that vehicles will be coated with emulsified asphalt. When a hot mix asphalt surface is used in an urban area with unstable soils, the thickness should be 2 inches, the minimum allowable thickness.

If a project has well-defined sections of unstable and stable embankment, consider selecting two types of surface courses, each appropriate to a level of embankment stability. Variables affecting this decision include the length and number of unstable areas, the percentage of the project with unstable embankment, the increased costs of multiple equipment spreads, and the variables discussed above in General Considerations.

7.3 Available Surfacing Types

Asphalt Concrete or Hot Mix Asphalt (HMA): HMA consists of a mixture of asphalt cement and well-graded aggregate. HMA provides the smoothest asphalt surface, the longest life, and contributes to the structural strength of the entire pavement structure. HMA generally requires less maintenance than other surfaces, and it is recommended for use in all stable embankment areas with AADTs greater than 1,000 or significant truck volumes. Consider HMA as an alternate for AADTs less than 1,000, based on the variables given in General Considerations.

Double-Layer Asphalt Surface Treatment (Double-Layer AST): Double-layer ASTs are typically made of two applications of asphalt emulsion and a single-sized aggregate. Double-layer ASTs are usually placed on a granular base. Design the pavement structure, including the base, to provide all the required strength, since the double AST provides no structural strength. The aggregate for a double-layer AST is more expensive than aggregate for a HMA or high-float AST, because more material is wasted making the single-sized aggregate. Double-layer ASTs

are a good alternative for unstable foundations and should be considered as an alternative to HMA for projects with AADTs less than 1,000. A double-layer AST is not recommended for high-speed, high-traffic urban areas.

Single-Layer Asphalt Surface Treatment (Seal Coat AST): A seal coat AST is constructed by spraying emulsified asphalt material followed immediately by a thin stone covering. An aggregate seal is typically used to extend the life of pavement. It produces an all-weather surface, renews weathered and cracked pavements, improves skid resistance, seals pavement, and gives no additional strength to pavement. An aggregate seal typically has a longer life in dry climates. An aggregate seal of imported, higher-quality aggregate may be designed for a new pavement if local aggregate wears poorly.

High-Float Asphalt Surface Treatment (High-Float AST): A high-float AST consists of one application of high-float emulsified asphalt followed by a single application of crushed gravel, and it is usually placed on a granular base. A high-float AST provides a less expensive alternative to double-layer AST because it uses a single layer of aggregate with a less restrictive gradation and provides the same design life. Because of the aggregate gradation and properties of the high-float emulsified asphalt, a high-float AST surface is rougher than a double-layer AST—this is especially true for the first few years following construction. As with double-layer ASTs, high-float ASTs are not considered to be structural layers. That is, a high-float AST does not contribute to the load capacity of the pavement structure. High-float ASTs may have a significant P₂₀₀ content, an undesirable trait for use in areas with high rainfall. In wet conditions, it is difficult to keep the P₂₀₀ fraction dry. Wet aggregate clumps in the distributor and makes uniform spreading nearly impossible. High-float AST is a good alternative in areas with unstable foundations.

Sand and Slurry Emulsion Seals: A sand emulsion seal is comprised of sand and emulsified asphalt. A slurry seal is usually comprised of fine, dense-graded aggregate and emulsified asphalt and is placed using specialized slurry seal equipment. Fine-grained seal coats increase skid resistance, seal against water intrusion, and correct minor surface irregularities.

Stone Mastic Asphalt Concrete (SMA): SMA is a coarse-graded HMA mix that resists rutting caused by studded tires. SMAs do not have good fatigue properties and should only be used as a surface layer on top of an HMA. Consider SMA as an alternative for projects with an AADT greater than 10,000.

Stabilized Base Course: Stabilized bases are composed of aggregate and a stabilizing (bonding) material. The aggregates are generally well graded but can be open graded. Bonding materials include asphalt cement, asphalt emulsion, Portland cement, lime, various proprietary chemical products, and even recycled asphalt concrete. Use stabilized bases as required in chapter 2.

Reclaimed Asphalt Pavement (RAP): Create RAP by removing and crushing old asphalt concrete pavement. The RAP often contains a substantial portion of normal base course material picked up during removal of the old pavement surface (sometimes as high as 25% to 30% by weight). RAP can be used as a component in recycled asphalt pavement or as a substitute for base course. A base course can be constructed exclusively from RAP or with a mixture of RAP and virgin crushed aggregate. Always consider the use of RAP in the design of a pavement rehabilitation project.

Recycled Asphalt Pavement: Recycled asphalt pavement is processed from reclaimed asphalt pavement by crushing and mixing it with additional components such as aggregate, asphalt cement, or recycling agents. It is then re-laid and compacted. Hot recycling of pavement is usually processed in a plant, while cold recycling of pavement is usually done in place. Always consider recycling existing asphalt as an alternative on a pavement rehabilitation project.

Gravel Surface: For very low-volume roads, always analyze the cost of grading and replacing gravel versus the cost and maintenance of any other surface. Consider the continual loss of the crushed gravel surface (through maintenance operations and wind erosion) in the cost analysis. Calcium chloride or other dust palliatives may be used, especially for higher traffic volumes. These usually last only a year or two. Also consider the cost of future applications in the maintenance costs. Gravel surface courses require a higher P₂₀₀ content than base courses for

hot mix asphalt surfaces (10 to 15% versus less than 6% for HMA). If a road is to be paved in the future, the surface course will need to be replaced.

Portland Cement Concrete (PCC): Because of the high cost of PCC and the necessity of a thick, non-frost-susceptible base, PCC is rarely used in Alaska. PCC pavement sections have seen limited use at some Alaska airports. Consider PCC as an alternative to HMA. Local conditions may result in PCC being cost-effective. However, this manual does not cover PCC design.

An NHI course is available for anyone interested in developing expertise in PCC design. See basic reference materials in reference.⁽²²⁾

7.4 Stabilized Layers

Use the following in conjunction with policies detailed in Chapter 2.

7.4.1 Stabilized Base

Stabilized base is a typical granular base course material that has been stabilized with a binder component. No minimum amount of additive is required, although stabilized base material must (1) achieve a M_R value $\geq 80,000$ psi, and (2) exhibit some other form of improvement that is directly applicable to improving the structural design of the pavement, e.g., reduced frost susceptibility. The modulus value improvement and/or other improvement(s) gained through the addition of the binder must be documented or otherwise verified by regional or statewide materials personnel with pavement design expertise. Acceptable documentation will cite previous experience with similar materials or will be based on test data using current test method(s).

Stabilized bases are normally defined as standard base course materials containing one or more of the following components:

- Emulsion
- Asphalt cement
- Foamed asphalt cement
- Lime
- Portland cement
- Recycled asphalt concrete pavement *
- A mixture of recycled asphalt concrete pavement and base course material *

** These stabilized base materials, incorporating recycled asphalt concrete, are usually created through an unheated, mechanical mixing process. Such mixtures may require a significant period of time after construction (perhaps a year or more) before the expected stabilization effect is fully achieved.*

A stabilized base is considered a lightly bound material. Therefore, use the Per Ullidtz equation as the failure criterion for stabilized base material (see Section 4.3.2).

7.4.2 Asphalt-Treated Base

Asphalt-treated base (ATB) is defined as a typical granular base course material that has been stabilized with a minimum of 4% asphalt cement (residual asphalt cement) binder additive. The minimum amount of asphalt cement additive required is that necessary to achieve an M_R value $\geq 150,000$ psi. The modulus value used for mechanistic design must be justifiably based on experience or on M_R current test method(s). Achievement of the 150,000-psi minimum modulus value, whether by test or presumption, must be documented or otherwise verified by regional or statewide materials personnel with pavement design expertise. Acceptable documentation will cite

previous experience with similar materials or will be based on test data using current test method(s).

Asphalt-treated base is considered a heavily bound material. Therefore, use The Asphalt Institute (TAI) fatigue equation as the failure criterion for asphalt-treated base material (see Section 4.3.2).

7.4.3 Alaska Renewable Pavement

Alaska Renewable Pavement (ARP) is a pavement layering system that provides an acceptable alternative to stabilized base or can be used to amplify the benefits of a stabilized base. ARP can also be combined with a stabilized base or asphalt-treated base to satisfy requirements for pavement designs involving high ESALs. The ARP system is similar to normal asphalt concrete pavement except that it is thicker and is composed of two sub-layers. Figure 7-1 illustrates ARP layering and examples of applications both with and without a stabilized base course.

The upper layer of the ARP, called the *wearing course*, consists of asphalt concrete containing components that: maximize resistance to abrasion wear (addresses tire-stud rutting), minimize surface roughness (addresses ride quality), minimize plastic deformation (addresses displacement rutting), and minimize permeability (addresses premature weathering and aging of the asphalt concrete).

The lower layer of the ARP, called the *binder course*, consists of asphalt concrete containing components that maximize fatigue resistance (addresses fatigue cracking) and minimize plastic deformation (addresses displacement rutting).

The ARP design provides for a minimum 30-year service life because the ARP concept anticipates periodic replacement of the upper ARP layer by mill-and-fill construction methods. Periodic mill-and-fill reconditioning can be done without ever penetrating the lower ARP layer. Therefore, vehicle traffic will never be subjected to an unpaved surface during future reconditioning events. Because of these objectives, apply an extended design life (30 years minimum) when considering an ARP pavement system.

Details regarding selection of materials and mix design requirements for ARP wearing and binder layers are outside the scope of this manual. However, the following points generally apply:

- The design procedure and mix design class may vary for upper and lower layers.
- Aggregate gradation type need not be the same for both layers.
- Minimum thickness for wearing course is 2 inches.
- Minimum thickness for binder course is 2 inches (if stabilized base is used) and 3 inches (if stabilized base is not used).

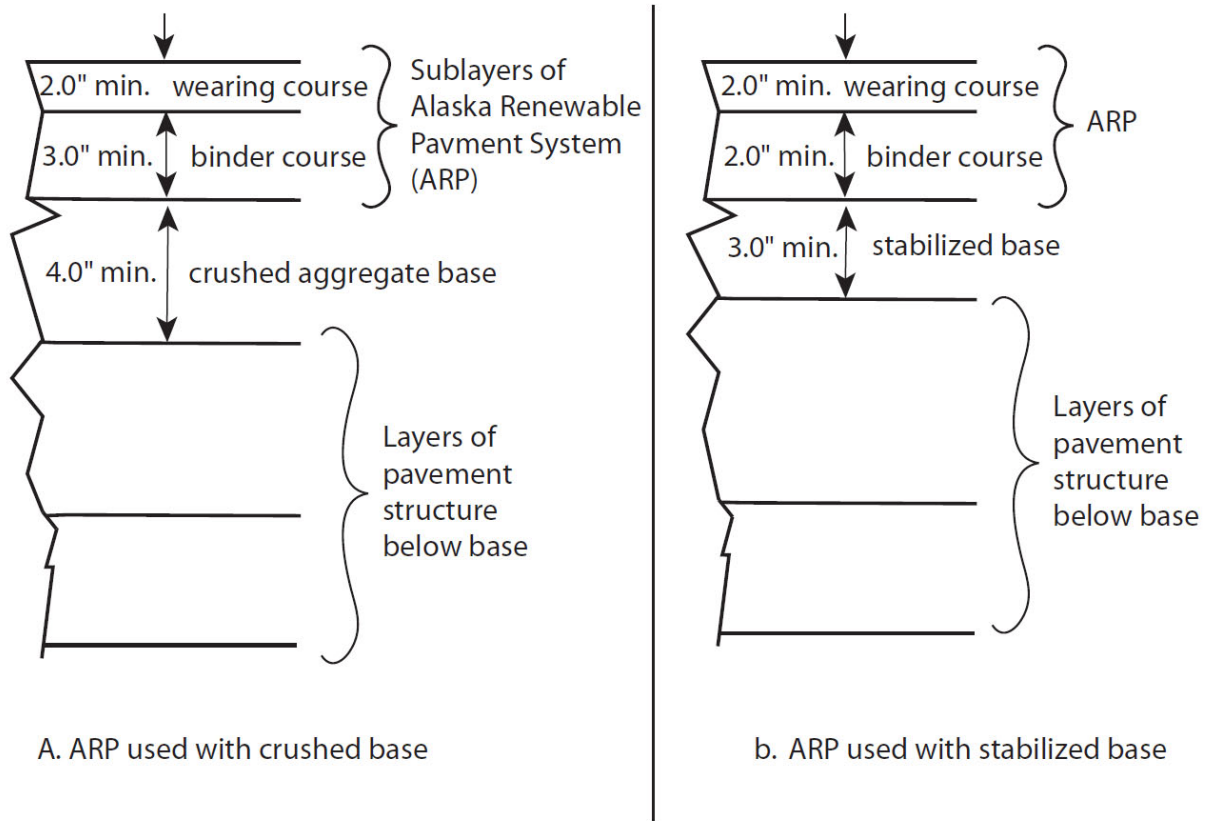


Figure 7-1 Pavement Structures Showing ARP Layers

8 Life-Cycle Cost Analysis

- 8.1 Introduction
- 8.2 Recommended Steps in the Analysis Process
- 8.3 Sample Hypothetical Analysis
- 8.4 LCCA Using AKFPD
- 8.5 Word of Caution

8.1 Introduction

The purpose of this life-cycle cost analysis (LCCA) chapter is to set forth guidelines for comparing alternative design strategies on an economic basis. Concepts of engineering economics are used to calculate the net present value of costs for each of the alternatives being considered, including not only first cost but also any preservation, rehabilitation, and reconstruction costs over the analysis period.

Both federal and state guidelines recommend such analyses. Prior to 1998, federal legislation mandated such studies for projects over a certain value. The requirement was removed in 1998, but Federal Regulations now requires that economic considerations be given to alternative combinations of materials during pavement design.⁽²³⁾ At the state level, Alaska Statute 19.10.160 provides, in part, “Design for proposed major upgrade and new construction projects for highways in federally recognized metropolitan planning areas must be conducive to safety, durability, and economy of maintenance ...”⁽²⁴⁾ Also, life cycle cost analysis is addressed several times in the DOT&PF Preconstruction Manual including: 1) definition of Value Engineering—Section 450.16; 2) items included in the design study report—Section 450.5.1; 3) use of surface treatments—Section 1180.3.2; 4) use of binder courses—Section 1180.4; and, 5) use of stabilized base courses—Section 1180.5.

The chapter is divided into three parts. In the first, seven steps in the analysis process are recommended, including establishing alternative design strategies, determining performance periods and activity timing, estimating agency and user costs, developing cash flow diagrams, computing net present value of costs for each alternative, performing a sensitivity analysis, and analyzing the results and reevaluating strategies. The second section contains an example that analyzes the costs of two hypothetical projects by utilizing the seven suggested steps, using a spreadsheet as the means of calculation. The third part describes the LCCA module within the AKFPD 2.0 software.

Many factors impact the selection of a design strategy, including safety, local needs, available materials, environmental concerns, and appearance. Economic considerations are but one of the factors in such a decision. The method outlined here provides a rational, straightforward means for analyzing the initial and future costs of pavement construction.

Keep in mind that some social, safety or political issues may not be quantifiable according to the standard economic analysis concepts presented in this chapter. Such issues may nevertheless weigh heavily – even critically – on selecting a pavement design alternative. Be aware of all decision-making issues, in addition to life cycle cost analysis; then be prepared to defend your selected design alternative on all accounts.

8.2 Recommended Steps in the Analysis Process

The steps suggested here are based on a condensed and modified version of the eight-step process set forth in *Life Cycle Cost Analysis in Pavement Design: In Search of Better Investment Decisions*.⁽²⁵⁾

8.2.1 Establish alternative design strategies

The process begins during the technical design, when the various design alternatives have been identified sufficiently that their costs can be reasonably estimated. The strategy of each alternative is a combination of initial pavement design and associated future preservation, rehabilitation, and reconstruction activities. Typically, two or three such strategies will have been identified; thus, there is a need to compare them on an economic basis. For consistency, it is essential that all strategies be set forth for the same analysis period. It is recommended that a 35-

year analysis period be used, a length of time usually enough to include several major activities in the rehabilitation and reconstruction cycle. Moreover, a 35-year analysis period is the minimum recommended by the Federal Highway Administration and has been accepted by the Alaska DOT&PF administration for application in life cycle cost analyses for all pavement design strategies. It is not expected that the pavement's predicted service life will be exactly 35 years or some simple fraction thereof; Chapter 2 of this manual sets forth policies for design lives for various types of projects. A pavement with remaining life at the end of the 35-year analysis period will have a salvage value that will be incorporated into the analysis.

All pavement design strategies must include consideration for work zone requirements during initial construction and any subsequent activities. Traffic control, detours, temporary drainage, environmental monitoring and control, and other efforts may both add to the agency's costs and have major impacts on the traveling public in the form of user costs. Work planning can reduce user costs through such strategies as confining field work to periods of low traffic and working the job in small segments rather than impacting traffic flow through the entire work zone during the entire construction period. Further consideration is given to user costs in the section on estimating agency and user costs.

8.2.2 Determine performance periods and activity timing

Having identified the scope and nature of the activities for each alternative in step one, we then must estimate their performance periods: how long they will last. Performance life of both the initial construction and any subsequent preservation, rehabilitation and reconstruction activities has a major impact on the life cycle cost analysis results. Determination of performance periods then leads to specifying activity timing: when, relative to the time of initial construction, each subsequent activity will take place. If an initial pavement construction (at "year 0") is expected to last fifteen years, with an overlay to be placed after that performance period, it follows that the first overlay is planned for "year 15." Chapter 2 of this manual provides guidance for the planning of performance periods.

8.2.3 Estimate agency and user costs

The purpose of the life cycle cost analysis is to compare alternative design strategies; thus, only *differences* in cash flows among the alternatives need be considered in estimating initial and future costs.

Agency costs include all costs incurred by DOT&PF (or owner agency) over the life of the project. These costs include planning, design, and contract administration; construction; traffic control; environmental monitoring and control; and any other costs paid for from the project budget. Note that these costs are present not only for initial construction but also future preservation, rehabilitation, and reconstruction activities. Often routine annual preservation is not included in life cycle cost analysis of competing design alternatives; the rationale is that the cost of such preservation is likely not to vary greatly among the alternatives and thus can be ignored in an analysis whose purpose is to compare the differences between alternatives. The method presented here can easily accommodate significant differences in annual preservation costs for the various alternative strategies if such differences have been identified. However, AKFPD does consider annual maintenance or preservation costs to reduce potential confusion as to whether or not to include it.

At the end of the analysis period (usually 35 years, as recommended above), in most cases the pavement will have some remaining value. A recommended approach to determining the pavement's salvage value is to use the cost of the most recent rehabilitation and/or reconstruction activity, if it still has some remaining life, and determine its remaining value based on the proportion of its life that remains. For example, suppose \$800,000 is spent at year 32 on an overlay whose total performance life is predicted to be eight years. Therefore, at year 35 (the end of the analysis period), five of its eight years remain. Thus, it is reasonable to find that the pavement's salvage value is

$$\left(\frac{5}{8}\right) \times (\$800,000) = \$500,000$$

A full treatment of user costs would try to recognize differences in operating costs throughout the 35-year analysis period due to variations among the various alternative strategies. It is recommended that such costs not be

included, under the assumption that they are essentially the same for any of the alternatives. This assumption is based on an expected DOT&PF management requirement that all allowable strategies must provide driving surfaces of similarly acceptable quality-levels during the entire 35-year analysis period. In other words, all acceptable alternatives must provide similarly good driving surfaces regardless of age.

Turning to work zone user costs during the initial construction and subsequent rehabilitation and reconstruction periods, three types of costs are recognized: user delay costs, vehicle operating costs, and crash costs. Recognizing these costs reflects the fact that, during periods of construction, preservation and rehabilitation, the capacity of the highway is restricted, and normal traffic flow is disrupted. Figure 8.1 produced using software developed by the Asphalt Pavement Alliance ⁽²⁶⁾ and updated to 2018 dollars can be used to estimate work zone user costs. The work zone user costs recognized by this graph include extra vehicle operating costs while changing speed and being stopped, and the user time costs of changing speed, traveling more slowly than normal and being stopped. The total cost estimate is a function of the type of roadway (urban or rural), whether work is conducted during the day or night, the estimated AADT, the number of work zone lane miles, and the number of days during which construction is underway. For example, a five-mile work zone, two lane, rural project with an AADT of 6000, a project duration of 40 days, and all work performed during the day would have an estimated work zone user cost of

$$\left(\frac{\$125}{1,000 \text{ AADT} - \text{lane mile} - \text{day}} \right) \times (6,000 \text{ AADT}) \times (10 \text{ lane miles}) \times (40 \text{ days}) = \$300,000$$

Other means of estimating user work zone costs would be 1) hand calculations based on the designer's knowledge of local conditions, or 2) direct use of the software that is the basis of the graph shown here or other similar software. However, for the sort of estimate needed for a legitimate life cycle cost analysis, it is believed that the graph will produce sufficiently accurate results with minimum effort.

Whether or not to include work zone user costs at all is both a philosophical and a practical matter. Some would suggest that they should not be included, since the primary consideration is the agency's budget, which nowhere reflects such costs. On the other hand, the agency's sole purpose is to act as the proxy for the public benefit, so the counterargument suggests that the agency ought to give serious attention to such costs. From a practical standpoint, some elements of these costs are difficult to estimate, and some, even if easily obtained, would have a minor impact on the overall cost.

Crash costs are usually excluded from the analysis because they are random events. While work zones do have higher crash rates, on average they may represent a minor element in the user cost picture; however, one fatality, when it occurs, is certainly a catastrophic event.

There are many arguments in favor of limiting the number of construction events and keeping the duration of the events that do occur as short as possible. From the user standpoint, these include crash potential; health issues from dust, noise, anger, and stress; delays; increased vehicle operating costs; and loss or cessation of business. From the agency's point of view, fewer and shorter construction events can lead to better public relations and greater certainty in material and labor costs. By including work zone user costs in the life cycle cost analysis, this element will be recognized; other things being equal, the alternative with the fewest and/or shortest construction events ought to be favored.

It is important that all costs be estimated in real, or present day, or time zero, dollars. Thus, the costs shown in Figure 8.1 are in real, time zero dollars. The calculation of present value using the discount rate, to be explained below, assumes that all cash flows are in present day dollars, rather than in actual, or inflated, or nominal dollars. Thus, all costs should be those that would apply at the time of the project's first capital expenditure, without adding the effects of inflation.

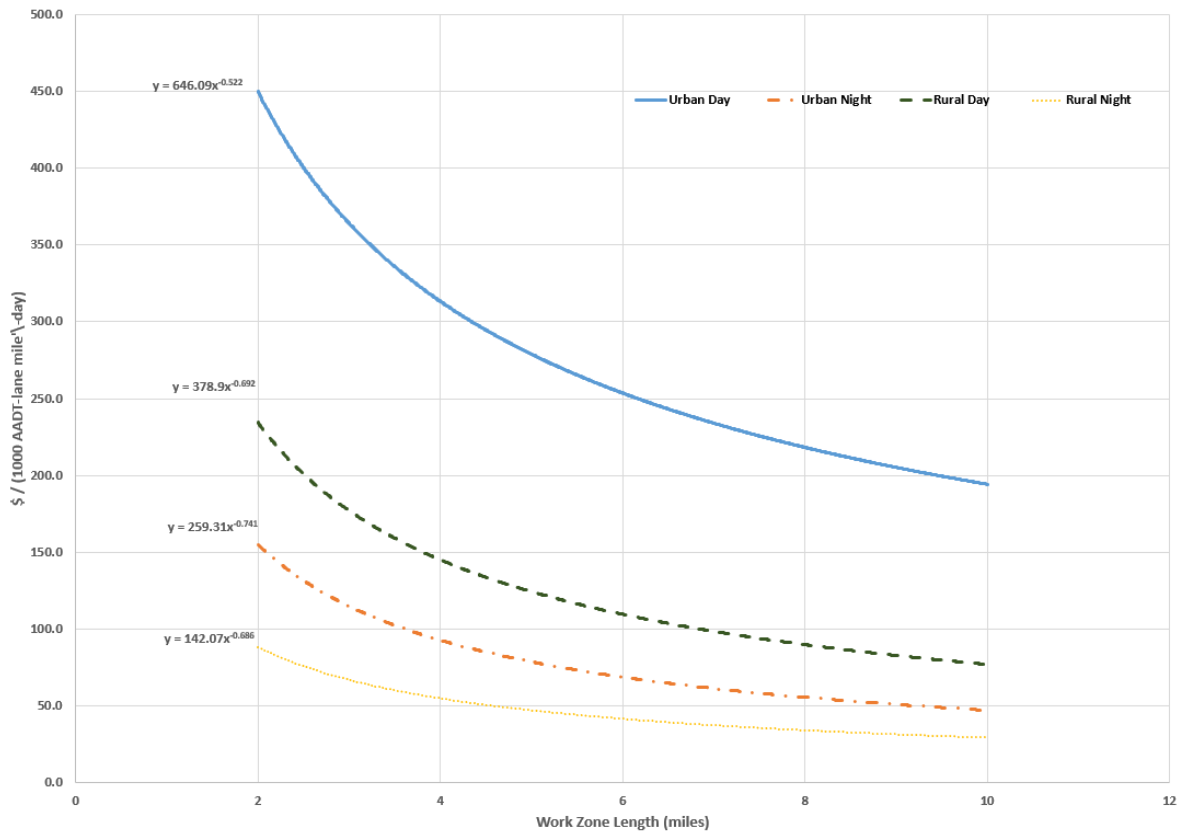


Figure 8-1 Work Zone User Delay Cost Estimates

8.2.4 Develop cash flow diagrams

A cash flow diagram is a simple graphical device for showing when, in time, the various cash flows associated with an alternative are expected to take place. Generally, the beginning of the cash flow period is taken as “time 0,” or “year 0.” The time scale of years is laid out horizontally, with each significant cash flow shown as an arrow at the year when it is expected to take place. Although each cash flow takes place during a non-instantaneous period of time, it is assumed that each occurs at one point in time. Cash flows occurring during a year are assumed to take place at the end of that year. The suggested convention is that costs, both to the agency and to the public, are shown as upward arrows, while any cash flows that are benefits, such as salvage values, are shown as downward arrows.

The following cash flow diagram represents a project whose initial cost, occurring during the year prior to year 0, is \$2,000,000; whose periodic costs, occurring just prior to or at years 14 and 28, are \$650,000 each; and whose salvage value, occurring at year 35, is \$500,000:

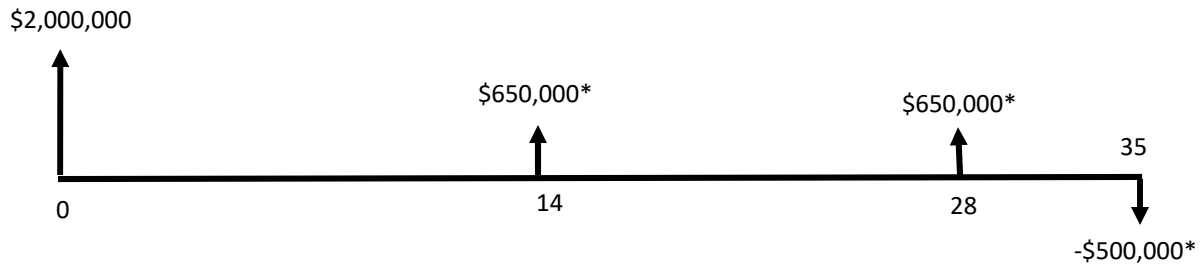


Figure 8-2 Sample Cash Flow Diagram

*Note that these cash flows are stated in real, or present day, or time zero, dollars.

Note that the length of each arrow is roughly proportional to its value, although this convention is often not followed very strictly.

8.2.5 Compute net present value of costs for each alternative

In order to compare the economic consequences of the various alternatives on a common basis, present value analysis is used. This method allows alternatives with different cash flow values and patterns to be compared in a way that reflects the time value of money. The net present value (NPV) is the discounted monetary value of the expected positive and negative cash flows, found by summing the discounted future cash flows using an appropriate discount rate and then adding the initial, time zero, cost. The result can be thought of as the amount that would be needed at the beginning of the project to fund the initial and future net costs, if any unspent amounts were invested at the given discount rate until needed. The alternative with the lowest net present value is considered the most attractive from an economic standpoint.

Two methods are available to perform the analysis, the deterministic method, and the probabilistic method. The former utilizes single values of each input variable and determines a single value of net present value for each alternative strategy. The latter considers several of the inputs as random variables and thus utilizes probability distributions and simulation to generate a distribution of possible values of net present value for each alternative. For simplicity, it is recommended that the deterministic method be used. Furthermore, it is recommended that such an approach be accompanied by a sensitivity analysis to give some measure of confidence in the results.

The present value of a future cash flow is found from the following formula:

$$PV = \frac{CF}{(1 + i_{dis})^n}, \text{ where}$$

PV = Present value,

CF = Cash flow at year n ,

i_{dis} = discount rate in decimal form, and

n = year at which cash flow occurs.

This formula is used to find the present value of each cash flow, and the individual present values are totaled to

find the net present value. Note that the present value of the initial cost is that cost itself, since it occurs at time 0 ($n = 0$), and the value of $(1 + i_{dis})^0$ will always be 1, regardless of the value of i_{dis} .

Suppose one of the cash flows that is expected to take place for a certain design strategy is a rehabilitation at year 14 that is estimated to cost \$650,000. If we use a discount rate of 4%, the calculation is as follows:

$$PV = \frac{\$650,000}{(1+.04)^{14}} = \$375,359, \text{ rounded, say, to } \$375,400$$

This calculated value is the time zero equivalent of the future cash flow. It can be thought of as the amount that would have to be invested at time zero in a fund earning 4% per annum in order to have \$650,000 in the fund at the end of year 14.

As one more example, consider the \$500,000 salvage value calculated earlier. Since this value occurs at the end of the 35-year analysis period, we can find its time zero equivalent as

$$PV = \frac{-\$500,000}{(1+.04)^{35}} = -\$126,700$$

Note that this value is negative because it has value to the owner and thus tends to reduce the cost of the project.

The results of the NPV calculations depend upon the assumed value of the discount rate. When the analyst uses real, or year zero, costs for all cash flows, as recommended above, the value of i in the formula for present value must be the discount rate, which does not include the assumed rate of inflation. The relationship among interest rate, discount rate and inflation rate are approximately the following:

$$\text{Discount rate} \approx \text{Interest rate} - \text{Inflation rate} .$$

Over many years, the difference between the interest rate on a 10-year U.S. Government Treasury note (a seemingly appropriate measure of the public's opportunity cost of money) and the inflation rate has been consistently between 3 and 5%. However, the Office of Management (OMB) recommends a real interest rate annually in OMB Circular A-94⁽²⁷⁾. This rate is used to represent an estimate of the average rate of return on private investment before taxes and after inflation.

The cash flow diagram for an alternative shows the assumed initial and future cash flows for that alternative; note once again that cash flows will be in real, or time zero, dollars. From the diagram, one can identify each cash flow and its assumed time of occurrence; application of the present value formula is then straightforward.

One method for performing the net present value calculations is to use a spreadsheet; that method is illustrated in Section 8.3. Such a technique is especially useful if a sensitivity analysis is performed; varying each desired input parameter, once the spreadsheet is populated with cash flow values, is uncomplicated and efficient. Other ways to carry out the calculations include hand calculations and the inclusion of life cycle cost analysis within the pavement design module. This last feature is illustrated in Section 8.4.

8.2.6 Perform a sensitivity analysis

The costs utilized in these analyses are only estimates, even though we have eliminated any consideration of inflation. The further we investigate the future, the less confidence we have in those estimates. Thus, it is

important that the analyst have some notion of how sensitive the recommendation is to variations in the input variables. If the cost of an overlay at year 12 (in real dollars) for the preferred alternative were 20% higher than estimated, and all other costs remain as estimated, would the recommendation change? What if the life of a new type of pavement turns out to be 10 years instead of 15; would such a condition cause the cost analysis to favor the other alternative? Or, if we paid a premium for the initial construction to be done at night, would there be enough savings in road user costs to justify such a scheme?

Using a spreadsheet, the user can vary selected input values to study questions such as these to determine some notion of confidence in the recommendation resulting from the initial analysis. The analyst will have to decide which input values should be studied in this way, based on her/his knowledge of which cost values are likely to be different from those assumed in the initial analysis.

The sample hypothetical analysis in section 8.3 gives an example of a simple sensitivity study that may suggest a helpful approach.

8.2.7 Analyze results and reevaluate strategies

After the calculations have been completed, the results must be analyzed and interpreted. In some cases, it may suffice to state which of the alternatives appears to be most economically attractive, based on the calculations. Perhaps the percent difference between two strategies will be stated. In some cases, the results may be so close as to indicate that factors other than economics will decide the preferred alternative. Results of the sensitivity analysis may play a significant part in identifying a recommendation. It is important to analyze the agency cost and user cost results separately, at least initially, and interpret them considering the relative importance that each is given in making a final recommendation.

Lastly, the designer/analyst must determine whether the results indicate that adjustments or modifications to the alternative strategies should be made prior to the final design decision. Perhaps the initial construction should be changed, or rehabilitation methods modified. Perhaps the analysis has indicated excessive user costs during construction, so that construction times, both initially and in the future, should be substantially reduced. Following such adjustments, further economic analysis may be warranted.

8.3 Sample Hypothetical Analysis

Suppose it is proposed to repave an eight-mile section of two-lane rural highway. Two alternative design strategies (Section 8.2.1) have been developed. The subbase would be the same for either alternative and is thus ignored in the life cycle cost analysis. Alternative #1 includes a 3" asphalt concrete pavement and a 4" stabilized base course. In the future, annual preservation will be performed. Major rehabilitation will consist of a thick overlay, while a complete reconstruction will include a pavement structure exceeding the quality of the original (year 0) construction.

Alternative #2 consists of a 2" asphalt concrete pavement and an 8" base course. Future work includes routine annual preservation, minor rehabilitation with a thin overlay, and a major reconstruction with the same design as the original project.

The performance periods and activity timing (Section 8.2.2) for each of these alternatives are shown in the following table.

Table 8-1 Performance Periods and Activity Timing for Sample Project

	Alternative #1	Alternative #2
Initial Structure	3" asphalt concrete + 4" stabilized base	2" asphalt concrete + 8" base
Initial Construction	Year 0	Year 0
Routine Preservation	Annually	Annually
Rehabilitation	Major rehabilitation @ years 15 & 25 (thick overlay)	Minor rehabilitation @ years 15, 20 & 25 (thin overlay)
Reconstruction	Complete rebuild at year 32, lasting 20 years	Major rebuild at year 30, lasting 15 years

Estimated costs (Section 8.2.3) are given in Table 8.2 shown below. Note that these are always stated as *real*, or *year zero*, costs. (It is not necessary to account for future inflation when listing future costs because of the *real* 4% discount rate that will be used in the analysis). User costs were obtained using Figure 8.1, Work Zone User Delay Cost Estimates, assuming daytime work for an eight-mile, two lane rural highway, and an assumed number of project days appropriate to the type of work. For example, for Alternative #1, if the initial construction requires 150 days, the AADT is 4500, and the value from the graph is 90, the calculated work zone user delay cost is \$972,000.

Table 8-2 Estimated Costs for Sample Project

	Alternative #1	Alternative #2
Agency costs		
Initial construction		
Engineering & contract administration	\$1,300,000	\$900,000
Construction	\$7,500,000	\$5,500,000
Traffic control	\$200,000	\$170,000
Rehabilitation (overlays)		
Engineering & contract administration	\$500,000	\$350,000
Construction	\$3,100,000	\$1,900,000
Traffic control	\$140,000	\$80,000
Reconstruction		
Engineering & contract administration	\$1,500,000	\$1,100,000
Construction	\$9,200,000	\$6,400,000
Traffic control	\$200,000	\$170,000
User costs		
Initial construction	\$1,026,000	\$1,373,800
Rehabilitation	\$457,900 (Year 15) \$549,500 (Year 25)	\$419,800 (Year 15) \$489,400 (Year 20) \$475,200 (Year 25)
Reconstruction	\$1,404,000 (Year 32)	\$1,814,400 (Year 30)

The costs of routine preservation are assumed to be equal for both alternatives and are therefore not included in the analysis. If this is not the case, it would be necessary to include such preservation costs for the years when they are expected to occur. To find the salvage value for each alternative, using the pro-rated life method, we proceed as follows:

For Alternative #1, the \$10,900,000 reconstruction cost at year 32 (shown in Table 8.3 and Figure 8.3) has a 20-

year life. Thus, at year 35, its value will be $\left(\frac{17}{20}\right) \times (\$10,900,000) = \$9,265,000$.

For Alternative #2, the \$7,670,000 reconstruction cost at year 30 (shown in Table 8.3 and Figure 8.4) has a 15-year life. Thus, at year 35, its value will be $\left(\frac{10}{15}\right) \times (\$7,670,000) = \$5,113,333$, say, \$5,113,000.

From the above activity timing and cost estimates, we can develop the following cash flow table and the cash flow diagrams (Section 8.2.4) shown in Figures 8.3 and 8.4

Table 8-3 Cash Flow Table for Sample Project

Year	Alt #1 Agency Cost	Alt #1 User Cost	Alt #2 Agency Cost	Alt #2 User Cost
0	\$9,000,000	\$1,030,300	\$7,570,000	\$1,373,800
15	\$3,740,000	\$457,900	\$2,330,000	\$419,800
20			\$2,330,000	\$461,700
25	\$3,740,000	\$549,500	\$2,330,000	\$503,700
30			\$7,670,000	\$1,923,300
32	\$10,900,000	\$1,488,200		
35	-\$9,265,000		-\$5,113,000	

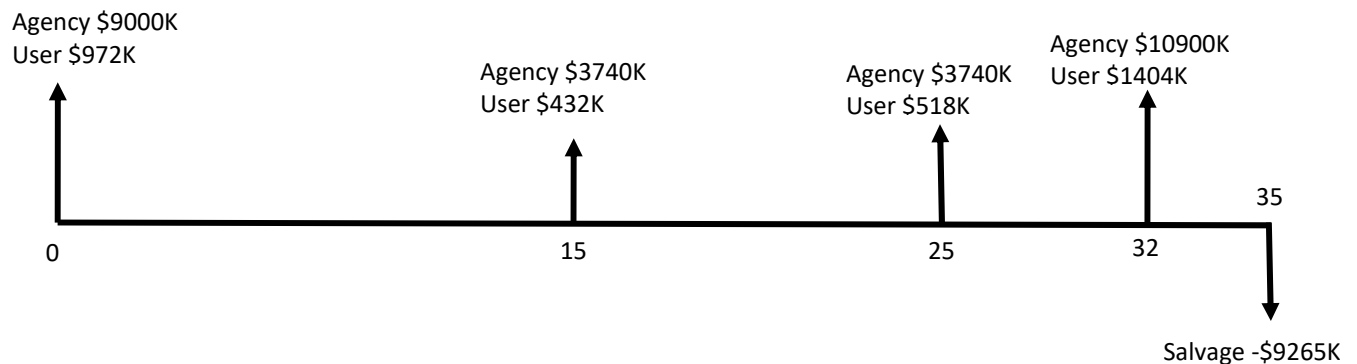


Figure 8-3 Cash Flow Diagram for Alternative #1

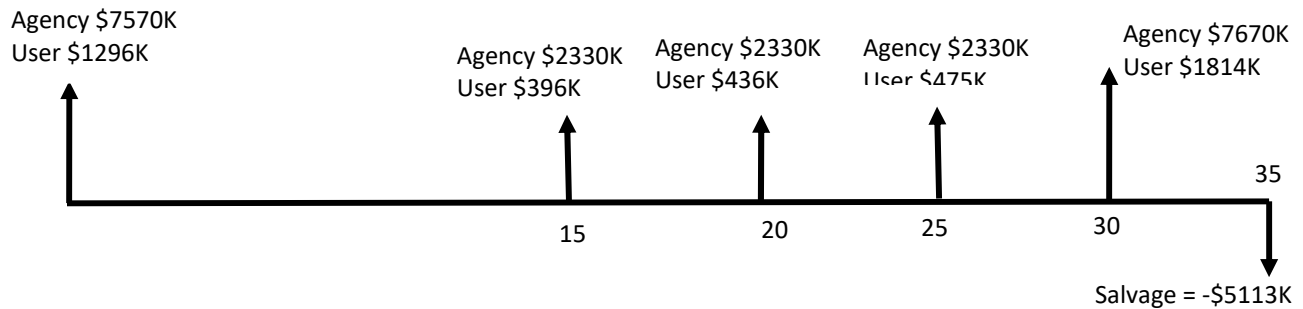


Figure 8-4 Cash Flow Diagram for Alternative #2

Computation of net present value for each alternative (Section 8.2.5) is based on the cash flow diagrams and a discount rate of 4%. We have separated the agency and user costs. The following spreadsheet shows the results of the analysis, which uses a simple $(1+i)^{-n}$ formula to calculate present value for each cash flow.

Table 8-4 Spreadsheet Output from Sample Hypothetical Analysis

Sample Hypothetical Life Cycle Cost Analysis						Discount Rate = 4%		
Column 1	Column 2	Column 3	Column 4	Column 5	Column 6	Column 7	Column 8	Column 9
Year	Alt #1 Agency Cost	PV Alt #1 Agency Cost	Alt #1 User Cost	PV Alt #1 User Cost	Alt #2 Agency Cost	PV Alt #2 Agency Cost	Alt #2 User Cost	PV Alt #2 User Cost
0	\$9,000,000	\$9,000,000	\$1,030,300	\$1,030,300	\$7,570,000	\$7,570,000	\$1,373,800	\$1,373,800
15	\$3,740,000	\$2,076,689	\$457,900	\$254,256	\$2,330,000	\$1,293,766	\$419,800	\$233,100
20					\$2,330,000	\$1,063,382	\$461,700	\$210,714
25	\$3,740,000	\$1,402,937	\$549,500	\$206,164	\$2,330,000	\$874,022	\$503,700	\$188,946
30					\$7,670,000	\$2,364,804	\$1,923,300	\$592,989
32	\$10,900,000	\$3,107,132	\$1,488,200	\$424,223				
35	-9,265,000	-2,347,894			-5,113,000	-1,295,713		
Column Sums		\$13,238,864		\$1,914,943		\$11,870,261		\$2,599,549
Sum, NPV Agency & User Costs				\$15,153,807				\$14,469,810

For this hypothetical example, Alternative #2 appears to be preferred. If only agency costs are considered (NPV of \$13,238,863 v. \$11,870,261), #2 is clearly the more economical with an NPV about \$1.5 million less than that of #1. If user costs are included, the preference is still for #2, but by considerably less (an NPV difference of about \$720,000). If user costs are considered particularly important, the agency might decide on Alternative #1.

A graphical presentation might be helpful as a visual means of seeing the results. Figure 8.5 summarizes the results of the basic analysis.

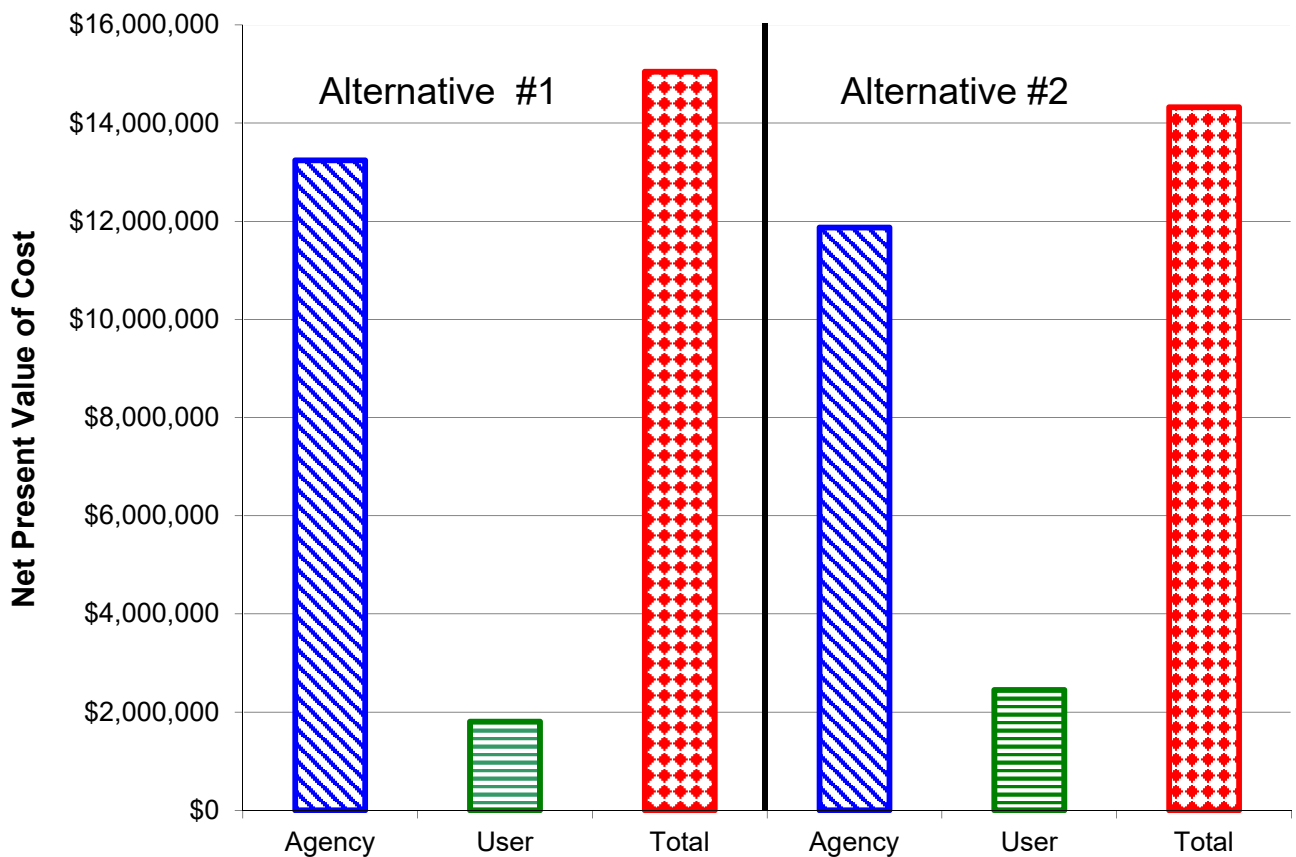


Figure 8-5 Graphical Comparison of NPV of Alternatives #1 and #2

A sensitivity analysis (Section 8.2.6) considers one of the input parameters, the year zero agency costs. One approach to studying the sensitivity of the year zero agency costs is to vary each by, say, 10% and determine whether the recommendation would change. There are many possible combinations:

- increasing both by 10%;
- increasing Alternative #1 by 10% and keeping Alternative #2 at its original value (as for example, if Alternative #1 is a relatively untried design whose construction costs are not well understood);
- decreasing one by 10% and increasing the other by 10%; and so forth.

Once the spreadsheet contains the original cost values, it is relatively easy to vary the values and identify their effects. Caution is advised that the number of combinations analyzed be kept to a manageable number representing likely actual costs.

- For our example, Table 8.5, we show four variations in the year zero agency costs:
- increase both alternatives by 10%; decrease both alternatives by 10%;
- decrease Alternative #1 by 10% while increasing Alternative #2 by 10%;
- increase Alternative #1 by 10% while decreasing Alternative #2 by 10%.

Using the 4% discount rate, the results are as follows:

Table 8-5 Results of Sensitivity Analysis for Sample Project

	Both Alternatives +10%	Both Alternatives -10%	Alt#1 -10%; Alt #2 +10%	Alt#1 +10%; Alt#2 - 10%
NPV Alt#1 Agency Costs	\$14,562,750	\$11,914,977	\$11,914,977	\$14,562,750
NPV Alt #1 User Costs	\$2,106,396	\$1,723,415	\$1,723,415	\$2,106,396
Sum NPV Alt #1 Costs	\$16,669,146	\$13,638,392	\$13,638,392	\$16,669,146
NPV Alt#2 Agency Costs	\$13,057,287	\$10,683,235	\$13,057,287	\$10,683,235
NPV Alt #2 User Costs	\$2,2,859,504	\$2, 2,339,595	\$2,662.826	\$2,339,595
Sum NPV Alt #2 Costs	\$15,916,791	\$13,022,829	\$15,720,113	\$13,022,829

Focusing only on the sum of agency and user costs, the results show that Alternative #2 is favored for three of the four cases under study. Alternative #1 is favored only if the first cost of Alternative #1 decreases by 10% while that of Alternative #2 increases by the same percentage.

The final step is to analyze the results and consider reevaluating the proposed strategies (Section 8.2.7). We have analyzed the results to some extent already. We found that the basic analysis favored Alternative #2 if both agency and user costs are considered. The difference in NPV was \$722,802, or about 5.0% of Alternative #2's NPV. Considering only agency costs, Alternative #2 had a NPV of \$1,368,602, or 11.5%, less than that for Alternative #1. User costs were higher for Alternative #2 by \$645,800, or about 35.7%, compared with those for Alternative #1. Under one scenario considered in the sensitivity analysis, Alternative #1 was favored, and it always had lower user costs. If road user costs are considered especially important for this project, a re-evaluation might study the impact of seeking proposals to perform construction at night in order to reduce road user costs.

8.4 LCCA Using AKFPD

The AKFPD Life Cycle Costing Analysis module computes the present value of a series of costs related to pavement construction, reconstruction, rehabilitation, preservation, and maintenance using the techniques described in this chapter. The LCCA module is a stand-alone module in that it does not ingest data from previous pavement structure analysis. While the module uses a database to store the anticipated life and costs of pavement related costs, it is expected that the user verifies and update these data as appropriate. It is also expected that the user develops realistic strategies for the life cycle of the pavement.

For example, if the remaining pavement life after the design period is less than the 20%, an overlay should not be employed. Consequently, it makes no sense to use an overlay as a strategy.

While developing the life cycle cost for a single strategy is useful, the power of LCCA is in the comparison of multiple strategies. For example, one might ask if investing additional funds now to increase the pavement life from 15 years to 20 years is cost effective. Is it better to overlay the pavement after 10 years or 15 years? Should a preservation strategy be applied and when? The LCCA module automates much of this process, but the user must still develop a series of actions to answer these and other questions.

Once potential strategies are on the table, the Mechanistic Design Module should be used to determine whether these strategies are feasible. If the designer randomly selects activities without ensuring they are likely to perform, LCCA is almost certainly misleading.

The order of the analysis is a personal preference. One could select strategies, perform the LCCA analysis and then run the mechanistic analysis to ensure the strategy meets the design criteria.

The other option is to define the strategy, perform the mechanistic analysis and then perform the LCCA.

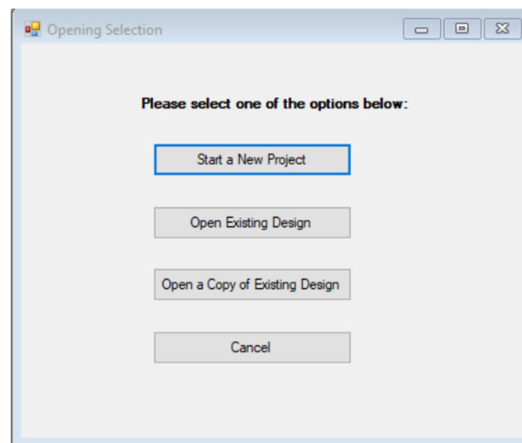
For the example presented here, assume that each strategy has been tested to ensure they are feasible.

Step 1. Develop multiple pavement structures using the Excess Fines Method or the Mechanistic Design Method. These pavement structures include those which will be used in scenarios included in the LCCA. Recognize that the initial pavement designs will often dictate the options available in the LCCA. For example, if a minimal pavement design is selected such that less than 20% of the fatigue life is available at the time an overlay is selected, an overlay is not an option. In this case full depth reclamation or similar activity may be appropriate.

The LCCA module is a stand-alone module and can be used separately or attached to a pavement design. Consequently, there are multiple options for entering the LCCA module.

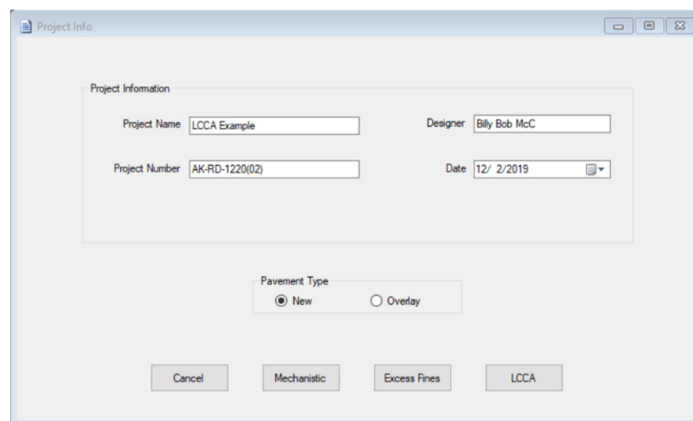
Option a: Open as a New LCCA Project

Step 2a. Open AKFPD and click on *Start a New Project*. (Clip 8-1) Entering the LCCA in this way creates a separate file with no pavement design attached.



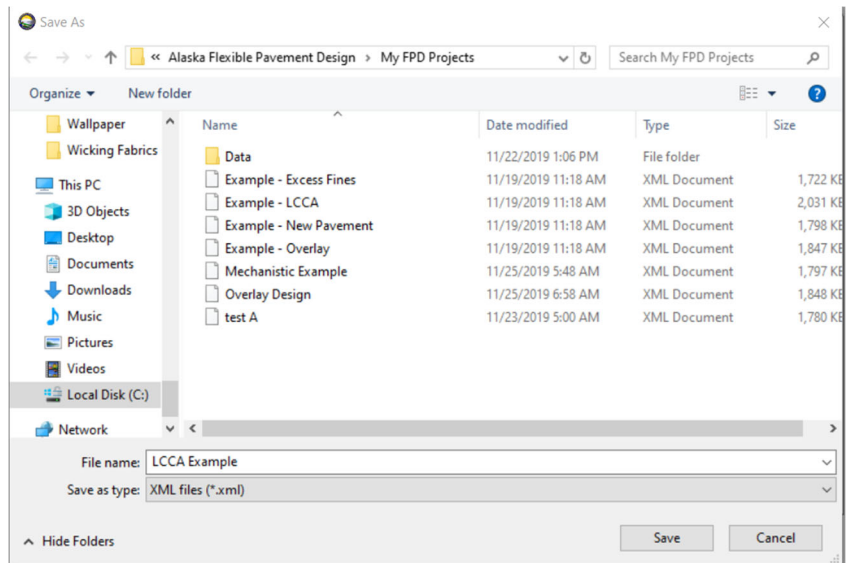
Screen Clip 8-1

Step 3a. Input Project Information and Click on *LCCA*. (Clip 8-2)



Screen Clip 8-2

Step 4a. Save File. Change name if appropriate. (Clip 8-3)



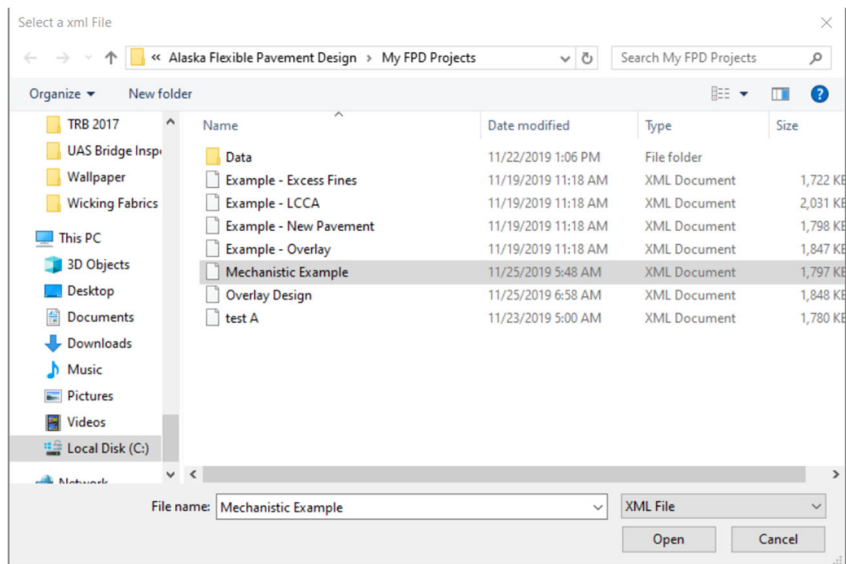
Screen Clip 8-3

Go To Step 5. Attach LCCA analysis to an existing project

Option b: Open LCCA module while attaching to an existing project.

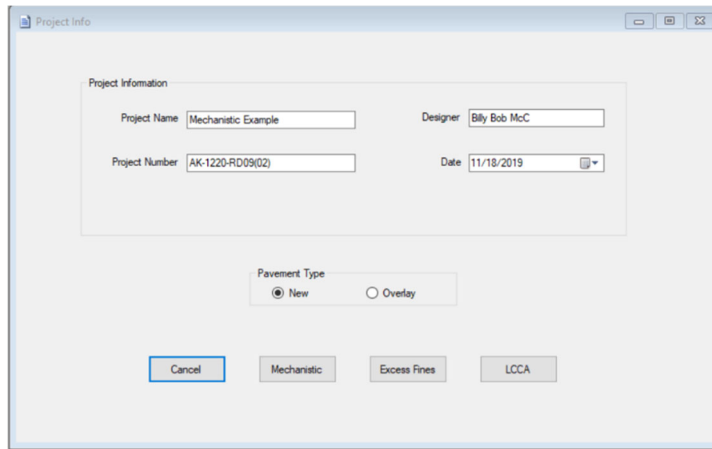
Step 2b: Open AKFPD and click on *Open Existing Project*. (Clip 8-1) *Open Copy of Existing Project* may be used to preserve an unaltered copy of the original project. (Screen Clip 8-1)

Step 3b. Select the desired file. Modify the name to create a new file and click *Open*. (Screen Clip 8-4)



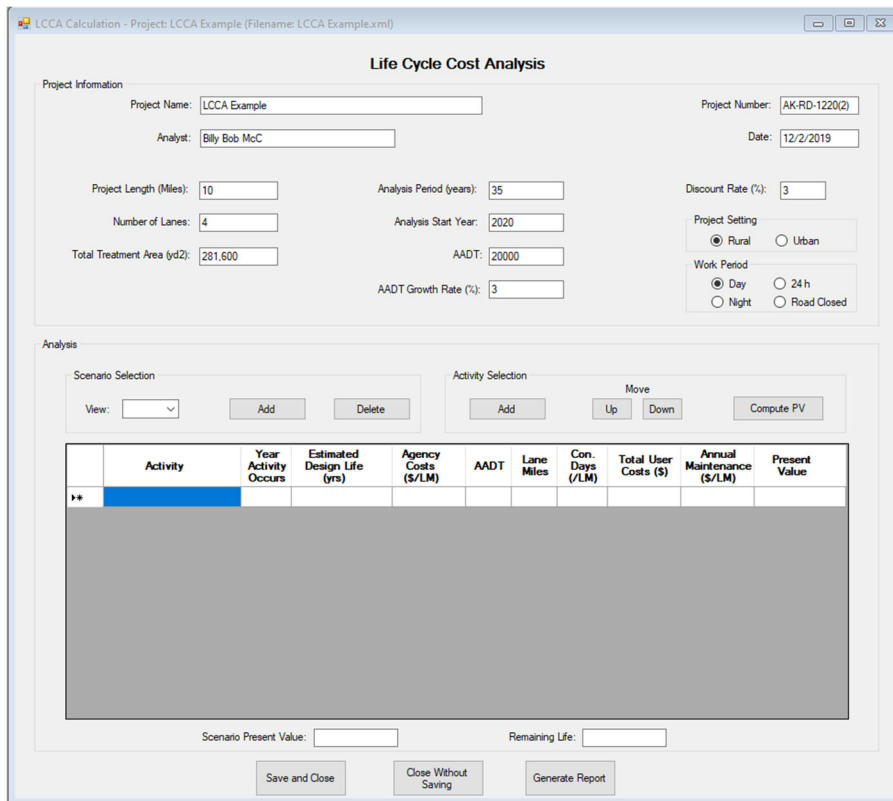
Screen Clip 8-4

Step 4b. If an existing project was used, the project information will be shown. (Clip 8-5)



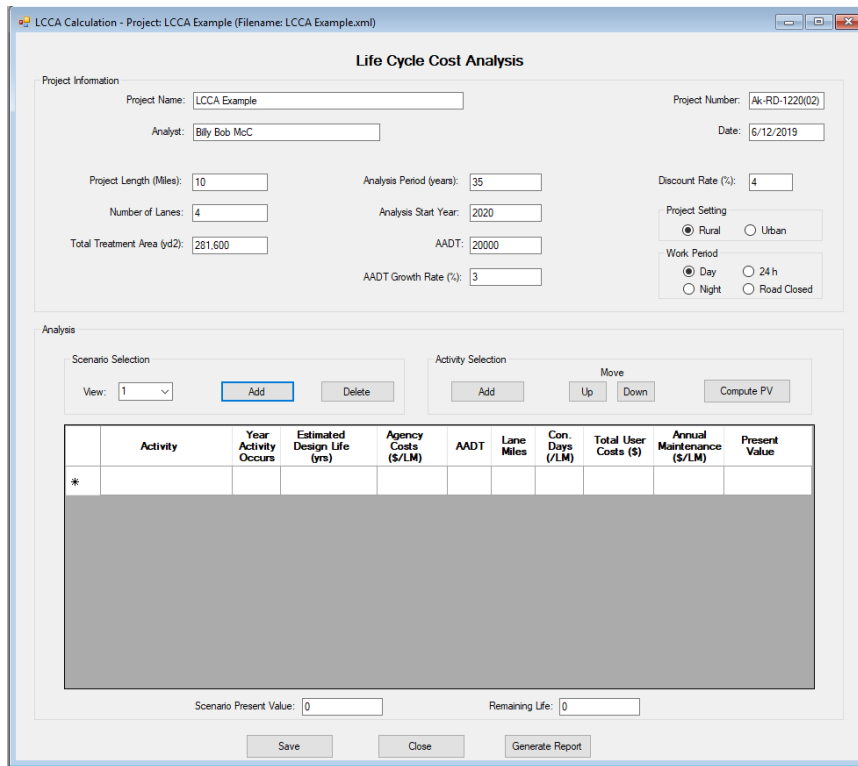
Screen Clip 8-5

Step 5 From this point forward the process will be the same for both approaches. In this case a new project will be used for the example. Complete the project information. The analysis period must be 35 years and the discount rate must be 3%. These values may be changed by directive from the Chief Engineer. Be sure to check the Project Setting (Urban or Rural) and the Work Period. These will impact the user delay costs. The screen should look like Clip 8-6.



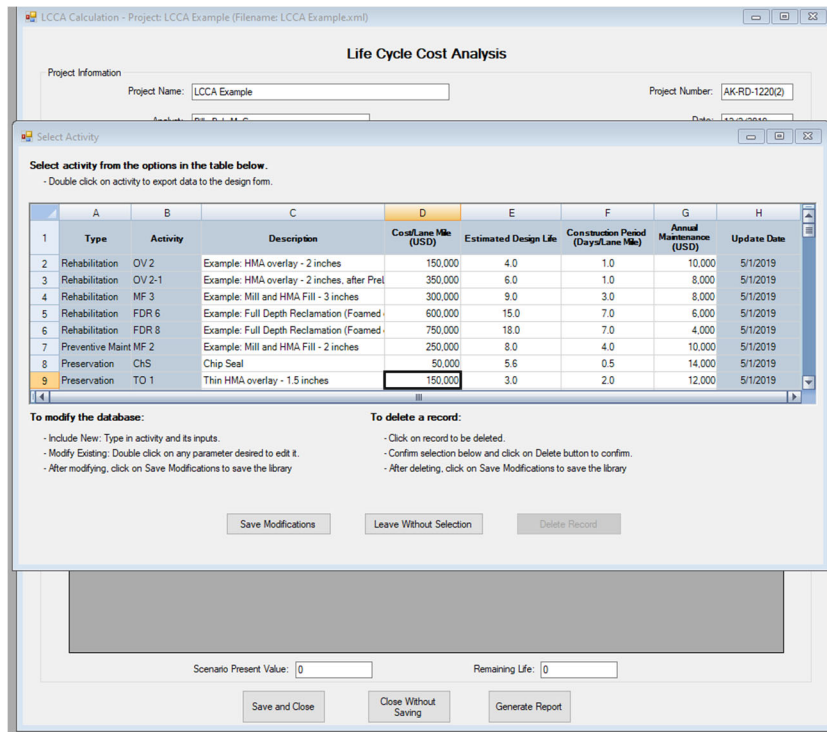
Screen Clip 8-6

Step 6. The first of multiple Scenarios will be developed at this point. Under *Scenario Selection* Box, click on **Add** to create a scenario. You may also remove a scenario by clicking on **Delete**. Once you click on **Add** in the *Scenario Selection* Box, the software will allow activities to be added to the scenario. (Clip 8-6)



Screen Clip 8-7

Step 7. Enter each activity by clicking on **Add** in the *Activity Selection* Box. A menu of existing alternatives will appear. (Clip 8-8)

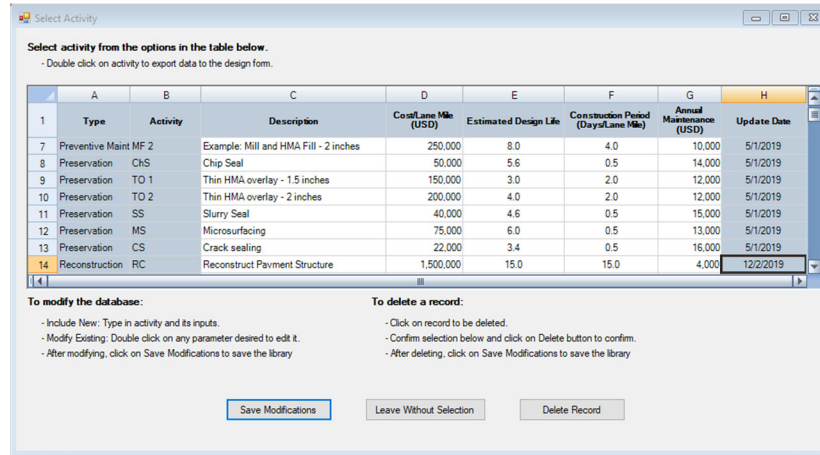


Screen Clip 8-8

Step 8. For purposes of this example, the design calls for reconstruction of the pavement structure with a life of

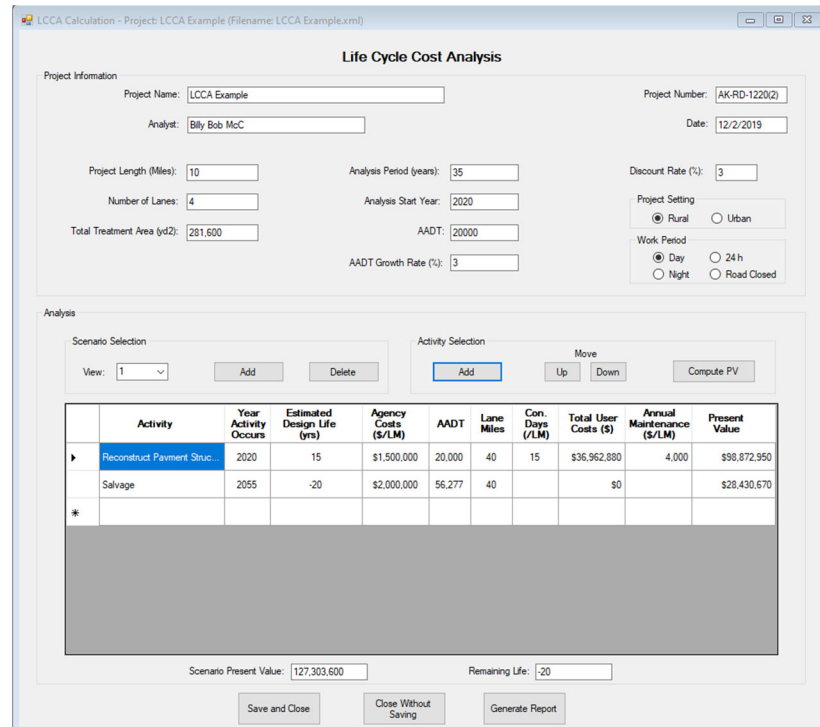
15 years at a cost of \$1,500,000/lane mile. This does not exist in the database, so it must be added. Add a new activity to the database by clicking on the first empty line and enter the data as shown in Clip 8-9. Once the data has been entered, click on **Save Modifications**.

Step 9 Double click on the activity, in this case “Reconstruction.”



Screen Clip 8-9

The user will be returned to the Life Cycle Cost analysis screen. The Reconstruction activity data has been entered as shown in Clip 8-10.



Screen Clip 8-10

Step 10. Repeat Step 7 until the remaining life shown below the table becomes positive (alternatively the estimated design life of the Salvage activity becomes positive), i.e., the sum of the estimated design lives equals or exceeds 35 years. The order of the activities can be altered by highlighting the activity you wish to move and using the buttons labeled “Up” and “Down” to move the activity to the appropriate point. An activity can be deleted by highlighting the activity and hitting the delete key on the keyboard. The completed scenario should look like Clip 8-11.

LCCA Calculation - Project: LCCA Example (Filename: LCCA Example.xml)

Life Cycle Cost Analysis

Project Information

Project Name: Project Number:
 Analyst: Date:

Project Length (Miles): Analysis Period (years): Discount Rate (%):
 Number of Lanes: Analysis Start Year: Project Setting: Rural Urban
 Total Treatment Area (yd2): AADT: Work Period: Day 24 h
 AADT Growth Rate (%): Night Road Closed

Analysis

Scenario Selection: View: Add Delete

Activity Selection: Add Move Up Down Compute PV

	Activity	Year Activity Occurs	Estimated Design Life (yrs)	Agency Costs (\$/LM)	AADT	Lane Miles	Con. Days (/LM)	Total User Costs (\$)	Annual Maintenance (\$/LM)	Present Value
▶	Reconstruct Pavement Struc...	2020	15	\$1,500,000	20,000	40	15	\$36,362,880	4,000	\$98,872,950
	Example: HMA overlay - 2 in...	2035	6	\$350,000	31,159	40	1	\$3,839,131	8,000	\$12,562,930
	Example: Full Depth Reclam...	2041	15	\$600,000	37,206	40	7	\$32,088,860	6,000	\$31,690,660
	Salvage	2055	1	(\$40,000)	56,277	40		\$0		(\$568,613)
*										

Scenario Present Value: Remaining Life:

Save and Close Close Without Saving Generate Report

Screen Clip 8-11

Step 11. Create Scenario 2 by Clicking *Add* in the *Scenario Selection* box.

Screen Clip 8-12

Step 12. Click *Add* under the Activity Selection box. Add Reconstruct Pavement Structure 20-year life as in Step 8. In this example this will be Reconstruct pavement structure with a 20-year life at a cost of \$1,700,000/lane-mile and a construction time of 17 days/lane-mile. (Screen Clip 8-13). Click *Save Modifications*.

	A	B	C	D	E	F	G	H
	Type	Activity	Description	Cost/Lane Mile (USD)	Estimated Design Life	Construction Period (Days/Lane Mile)	Annual Maintenance (USD)	Update Date
11	Preservation	SS	Slurry Seal	40,000	4.6	0.5	15,000	5/1/2019
12	Preservation	MS	Microsurfacing	75,000	6.0	0.5	13,000	5/1/2019
13	Preservation	CS	Crack sealing	22,000	3.4	0.5	16,000	5/1/2019
14	Reconstruction	RC	Reconstruct Pavment Structure	1,500,000	15.0	15.0	4,000	12/2/2019
15	Reconstruction	RC	Reconstruct Pavment Structure 20 yr	1,700,000	20.0	17.0	4,000	12/2/2019
16								
17								
18								

Screen Clip 8-13

Step 14. Double click on the new activity to add the 20-year reconstruction design to the Scenario 2. Add additional activities to Scenario 2 until the sum of the estimated design lives equals or exceeds 35 years. Scenario 2 should look like Screen Clip 8-13.

LCCA Calculation - Project: LCCA Example (Filename: LCCA Example.xml)

Life Cycle Cost Analysis

Project Information

Project Name: LCCA Example Project Number: AK-RD-1220(2)
Analyst: Billy Bob McC Date: 12/2/2019

Project Length (Miles): 10 Analysis Period (years): 35 Discount Rate (%): 3
Number of Lanes: 4 Analysis Start Year: 2020
Total Treatment Area (yd²): 281,600 AADT: 20000
AADT Growth Rate (%): 3

Project Setting
 Rural Urban

Work Period
 Day 24 h
 Night Road Closed

Analysis

Scenario Selection: View: 2 Add Delete

Activity Selection: Add Move (Up, Down) Compute PV

	Activity	Year Activity Occurs	Estimated Design Life (yrs)	Agency Costs (\$/LM)	AADT	Lane Miles	Con. Days (/LM)	Total User Costs (\$)	Annual Maintenance (\$/LM)	Present Value
▶	Reconstruct Pavement Situ.	2020	20	\$1,700,000	20,000	40	17	\$41,891,270	4,000	\$112,271,700
	Salvage	2055	-15	\$1,275,000	56,277	40		\$0		\$18,124,550
*										

Scenario Present Value: 130,396,200 Remaining Life: -15

Save and Close Close Without Saving Generate Report

Screen Clip 8-14

Step 14. Create Scenario 2 by repeating steps 6-10. The screen for Scenario 2 should look something like Screen Clip 8-15.

LCCA Calculation - Project: LCCA Example (Filename: LCCA Example.xml)

Life Cycle Cost Analysis

Project Information

Project Name: LCCA Example Project Number: AK-RD-1220(2)
Analyst: Billy Bob McC Date: 12/2/2019

Project Length (Miles): 10 Analysis Period (years): 35 Discount Rate (%): 3
Number of Lanes: 4 Analysis Start Year: 2020
Total Treatment Area (yd²): 281,600 AADT: 20000
AADT Growth Rate (%): 3

Project Setting
 Rural Urban

Work Period
 Day 24 h
 Night Road Closed

Analysis

Scenario Selection: View: 2 Add Delete

Activity Selection: Add Move (Up, Down) Compute PV

	Activity	Year Activity Occurs	Estimated Design Life (yrs)	Agency Costs (\$/LM)	AADT	Lane Miles	Con. Days (/LM)	Total User Costs (\$)	Annual Maintenance (\$/LM)	Present Value
▶	Reconstruct Pavement Situ.	2020	20	\$1,700,000	20,000	40	17	\$41,891,270	4,000	\$112,271,700
	Chip Seal	2040	6	\$50,000	36,122	40	0.5	\$2,225,303	14,000	\$4,019,094
	Example: HMA overlay - 2 in...	2046	6	\$350,000	43,132	40	1	\$5,314,256	8,000	\$9,759,734
	Chip Seal	2052	6	\$50,000	51,502	40	0.5	\$3,172,750	14,000	\$3,114,635
	Salvage	2055	3	(\$25,000)	56,277	40		\$0		(\$355,383)
*										

Scenario Present Value: 128,809,700 Remaining Life: 3

Save and Close Close Without Saving Generate Report

Screen Clip 8-15

Step 15. Add as many scenarios as appropriate. Once all scenarios have been added click **Generate Report**. A report will be generated and displayed. See Screen Clip 16.

Alaska DOT&PF Flexible Pavement Design Software, Version 2.0 - Project: LCCA Example (Filename: LCCA Example.xml) - [Workbook: LCCA Example]

File Clipboard Modules View Help

1	Project Name: LCCA Example			Project Number: AK-RD-1220(2)			Analysis Date: 12/22/2019		AADT: 20000		
2	Design Type: New Design			Project Length: 10 Miles			Discount Rate: 3 %		AADT Annual Growth: 3 %		
3	Designer: Billy Bob McC			No. Lanes: 4			Project Setting: Rural/Day		Analysis Period: 35		
4									Start Year: 2020		
5	SUMMARY										
6	Scenario	Total Net Present Value (USD)	Rank	Remaining Life (yrs)							
7	2	128,809,744	1	3							
8	1	142,557,936	2	1							
9											
10	DETAILED CALCULATIONS										
11											
12	Scenario	Description of Activity	Year Activity Occurs	Estimated Design Life (yrs)	Agency (\$)	User Cost Input			Estimated User Cost (\$)	Annual Maintenance (\$/LM)	Present Value (\$)
13						AADT	Lane Miles	Days (LM)			
14	1	Reconstruct Pavement Structure	2020	15	60,000,000	20,000	40	15	36,962,880	4,000	98,872,950
15		Example: HMA overlay - 2 inches, after PreLevel	2035	6	14,000,000	31,159	40	1	3,839,131	8,000	12,562,930
16		Example: Full Depth Reclamation (Foamed or emuls)	2041	15	24,000,000	37,206	40	7	32,088,860	6,000	31,690,660
17		Salvage	2055	1	-1,600,000	56,277					
18											
19	2	Reconstruct Pavement Structure 20 yr	2020	20	68,000,000	20,000	40	17	41,891,270	4,000	112,271,700
20		Chip Seal	2040	6	2,000,000	36,122	40	1	2,225,303	14,000	4,019,094
21		Example: HMA overlay - 2 inches, after PreLevel	2046	6	14,000,000	43,132	40	1	5,314,256	8,000	9,759,734
22		Chip Seal	2052	6	2,000,000	51,502	40	1	3,172,750	14,000	3,114,635
23		Salvage	2055	3	-1,000,000	56,277					
24											
25											

Screen, Clip 8-16

The summary box shows the scenarios ranked from lowest total net present value (NPV) to the highest. In this case the lowest NPV is for Scenario 2. Adding an additional five years to the life of the pavement shown in Scenario 2 requires about ½ inch of additional pavement. However, the impact to the public is delayed by five years and eliminates the final overlay.

Detailed computations are provided so that the user can evaluate the results in detail.

The report can be printed by clicking **File** then **Print**,

8.5 Word of Caution

The validity of Life Cycle Costing is dependent on the quality of input to the analysis. While LCCA module of AKFPD simplifies the computation of life cycle costing, the development of cost data requires considerable effort. There is no escaping the need for good cost data. While the software does not directly aid in the development of high-quality cost data, the embedded data base does provide the ability to record the data which can then be applied to similar projects. In doing so, the designer is afforded the ability to reuse prior work.

The designer must recognize that the initial design will have considerable impact on the future maintenance, preservation, and rehabilitation strategies. For example, a minimal pavement structure may force reconstruction after 15 years, while a heavier pavement structure may allow an overlay after a similar period. Consequently, the designer is encouraged to explore these options to identify the best life cycle cost.

User costs often dominate the LCCA. Over the years, the debate as to whether user costs should or should not be part of the LCCA continues. Some argue that the Departments of Transportation are responsible for the development and maintenance of the State's transportation system and the cost to the user should not be included in the decision process. Others argue that the DOTs are responsible for providing a transportation system for the benefit of the public including the impact to the public for maintenance and construction activities. The Federal Highway Administration supports the latter philosophy and requires that user costs be included in any LCCA for the nation's highway network. As such the designer will be well advised to develop strategies that minimize the impact of construction and maintenance of highways on the public.

A closer look at the example above illustrates this principle. Scenario 1 uses a 15-year life for the initial construction while Scenario 2 uses a 20-year life. In this example, the LCCA is dominated by user costs. Scenario two clearly has a lower impact on the public and consequently has the lower LCCA.

It is unrealistic to assume the scenarios used in the LCCA will be followed exactly. As time passes, technology, funding, and design philosophies change. Perhaps the greatest value of LCCA is to develop an initial design that allows future designs the flexibility to reduce costs. For example, an overlay is likely less expensive than reconstruction, but can only be an option if the initial design allows for the overlay.

9 Glossary

The definitions for the following terms are those commonly used in the transportation industry and particularly by the Alaska DOT&PF. Although some of these terms may seem fundamental, we provide them so that everyone, regardless of field experience, can develop an understanding of this nomenclature from this quick reference guide.

AASHTO: The acronym for the American Association of State Highway and Transportation Officials. A “T” designates AASHTO tests (example: AASHTO T195). An “M” designates AASHTO specifications (example: AASHTO M156).

Agency costs – All costs incurred by the owning agency over the life of the project, including preliminary engineering, contract administration, initial construction, construction supervision, preservation, rehabilitation, administration, and the impact of salvage value.

Aggregate: Any combination of one or more hard granular mineral materials, either natural or crushed, from very fine to large rocks. It is selected because of its characteristics for a specific purpose, such as sand, gravel, crushed stone, ballast, etc., used for mixing in graduated fragments.

Alaska Renewable Pavement (ARP): Pavement layering system that is an acceptable alternative to stabilized base or can be used to amplify the benefits of a stabilized base. ARP can also be combined with a stabilized base or asphalt-treated base to satisfy requirements for pavement designs involving very high ESALs. The ARP system is similar to normal asphalt concrete pavement except that it is thicker and is composed of two sublayers. The ARP design provides for exceptionally long service life because the ARP concept anticipates periodic replacement of the upper ARP layer by mill-and-fill construction methods.

Analysis period – The time period over which alternative design strategies are analyzed.

Asphalt: A dark brown to black cementitious material in which the predominating constituents are bitumens that occur in nature or are obtained as residue in petroleum distillation. Asphalt imparts controllable flexibility to mixtures of mineral aggregates, with which it is usually combined. It is highly resistant to most acids, alkalis, and salts. Although it is a solid or semisolid at ordinary atmospheric temperatures, asphalt may be liquefied by applying heat, dissolving it in petroleum solvents of varying volatility, or emulsifying it.

Asphalt Cement: Asphalt that is refined to meet specifications for paving, industrial, and special purposes. The term is often abbreviated to AC or referred to as binder when used in an asphalt hot mix.

Asphalt Concrete: Also referred to as hot mix asphalt (HMA), and hot bituminous mix. It is the material most used for surfacing roadways and airports in Alaska that are subject to high traffic. It is a high-quality, controlled, hot mixture of asphalt cement and graded aggregate, thoroughly compacted into a uniform dense mass.

Asphalt Soil Stabilization (soil treatment): Treatment of naturally occurring non-plastic or moderately plastic soil with cutback or emulsified soil mixture produces water-resistant base or subbase courses of improved load-bearing qualities.

Asphalt-Treated Base: A base course constructed using hot asphalt cement as a binder, often referred to as ATB. See Treated Base Courses for further descriptions of types.

ASTM: The acronym for the American Society for Testing and Materials.

ATM: Stands for Alaska Test Methods. These tests were developed by the headquarters Design and Engineering Services/Materials section.

Annual Average Daily Traffic (AADT): The average volume for a 24-hour period for all traffic lanes. It is normally the annual total volume divided by 365, unless otherwise stated. The AADT may be reported as one-way or two-way traffic. Make sure it is clear which is being reported.

Axle Load: The total load transmitted to the pavement by all wheels of either a single or tandem axle, usually expressed in kips (1 kip = 1 kilo pound = 1,000 pounds of force).

Base Course (BC): The layer or layers of specified material of designed thickness placed on a subbase or a subgrade to support a surface course. Most base courses are constructed with crushed aggregates and therefore called crushed aggregate base course.

Binder Course: Where thick pavements are required, the asphalt concrete pavement is sometimes placed as two layers, each differing in composition but sharing approximately the same fatigue properties (see also **Wearing Course**). The binder course is the bottom portion of asphalt concrete pavement, “tuned” to provide maximum fatigue resistance (addresses fatigue cracking) and minimum plastic deformation

Cash flow diagram – A time-scaled visual means of representing cash flows as arrows at the points in time when they are expected occur, with in-flows and out-flows shown in opposite directions. Also called expenditure stream diagram.

Chip Seal: A chip seal or “single-shot” asphalt surface treatment is the spraying of emulsified asphalt material (CRS-2 or RS-2) followed immediately by a thin stone cover.

Design alternative/design alternative strategy – A combination of initial pavement design and necessary supporting pavement preservation and pavement rehabilitation activities.

Design Lane: The lane on which the greatest number of equivalent 18,000-pound single-axle loads is expected. Normally this will be either lane of a two-lane roadway or an outside lane of a multilane highway.

Deterministic LCCA – Life cycle cost analysis that utilizes a single, most likely value of each input parameter for each alternative strategy, resulting in a single output value for each strategy (see Probabilistic LCCA).

Discount rate vs. interest rate – Discount rate is the rate used for the life cycle cost analysis, when real (or constant) dollars are used; discount rate represents the real value of money over time and is used to convert future costs to present-day costs. A discount rate of 3% is recommended for LCCA. Interest rate (often referred to as market interest rate) is the cost paid to borrow money. The relationship between discount rate and interest rate is a function of inflation, with discount rate approximately equal to the difference between interest rate and inflation rate.

Equivalent Single Axle Loads (ESAL): Traffic on highways and streets varies in the number of vehicles and in the magnitude of loading. The cumulative effects of traffic loads factor in the structural design of pavement. The effect on the pavement performance of any combination of axle loads is equated to the number of standard 18,000-pound, dual tired, single-axle loads required to produce an equivalent effect (i.e., the single axle load). In design of pavement structural sections, the total number of ESALs is a summary of equivalent 18,000-pound single-axle loads expected from the combination of all vehicle classes for the design period.

Excess Fines: The fines content above the critical fines content ($P_{200} - P_{Cr}$).

Excess Fines Factor (EFF): A factor that includes the effects of the excess fines and the applied stress at a given depth $(\Delta SFR)(P_{200} - P_{Cr})^{0.8}$.

Falling Weight Deflectometer (FWD): A trailer-mounted device that drops a known weight from known heights on a pavement surface while automatically measuring the resulting peak stress and deflections. The drop stress is usually intended to simulate dynamic traffic loading. The data collected with the FWD back-calculates elastic moduli of the supporting layers. Once the elastic moduli are known, structural design can proceed to determine critical stresses and strains in the structure.

Fatigue Cracking: Interconnected cracks forming a series of small blocks resembling an alligator’s skin or chicken wire. They are caused by heavy traffic that is excessive for the given thickness of pavement and structural support provided by underlying layers

Fatigue Resistance: The ability of asphalt pavement to withstand repeated flexing or slight bending caused by the passage of wheel loads. As a rule, the higher the asphalt content and the lower the air void content in an asphalt mix, the greater the fatigue resistance. However, a mix with too high an asphalt content or too low an air void content will tend to rut under traffic loading.

Fines Content (P200): The average percentage by weight of material passing the No. 200 sieve (0.075 mm).

Flexibility: The ability of an asphalt pavement structure to conform to settlement of the foundation. It is also sometimes called the ability of asphalt pavements to heal themselves during warm weather. A high asphalt content can enhance flexibility of an asphalt paving mixture.

Flexible Pavement: Another term for asphalt concrete pavement.

Gradation: The relative size distribution of the particles in an aggregate sample. The percentages passing various sieve sizes, from the largest (100% passing) to the smallest (No. 200 sieve) show the gradation of the material.

Heavy Trucks: Two-axle, six-tire trucks or larger, including trucks with heavy-duty, wide-base tires. Pickup, panel, and light four-tire trucks are not included.

In Situ: In the natural or original position.

Intermediate Course (sometimes called binder course): An asphalt pavement course between a base course and an asphalt surface course.

Life cycle cost analysis (LCCA) – An analysis technique that builds on the well-founded principles of economic analysis to evaluate the over-all-long-term economic efficiency between competing investment options. It incorporates initial and discounted future agency, user, and other relevant costs over the life of alternate investments. It attempts to identify the best value (the lowest long-term cost that satisfies the performance objective being sought) for investment expenditures.

Net present value (NPV) is the combined present value of all cash flows for a given alternative design strategy. (See [Present value](#))

Nominal cost – Costs that include the effect of inflation or deflation over time. Also called current or data year costs.

Pavement Design Period (“n”): The number of years that a pavement is expected to carry a specific traffic volume and retain minimum serviceability without rehabilitation. This is optimized by the Pavement Management System.

Pavement Performance: The trend of serviceability in relation to load applications.

Pavement Rehabilitation: Work to extend the service life of an existing facility. This includes placement of additional surfacing material or other work necessary to return an existing roadway, including shoulders, to structural or functional adequacy. This could include the removal and replacement of the pavement structure.

Pavement Structure: The combination of select material, subbase, base, and surface course placed on a subgrade to support the traffic load and distribute it to the roadbed (42 inches below the asphalt concrete layer).

Percent Trucks (PTT): The percentage of annual average daily traffic (AADT) that is heavy truck traffic.

Performance Period: The time that an initially constructed or rehabilitated pavement structure will last before reaching its terminal serviceability; this is also referred to as the design period.

Predicted Deflection (D_p): The predicted maximum probable deflection of a proposed pavement structure due to an 18,000-pound, single-axle load.

Present value– The time zero equivalent of a single future cash flow, based on the discount rate and the number of years in the future at which the cash flow is expected to occur.

Probabilistic LCCA – Life cycle cost analysis in which some or all of the input parameters are described as probability distributions, resulting in a probability distribution of life cycle costs for each alternative.

Real cost – Costs that do not include the effect of inflation or deflation over time; they have the same purchasing power over time. Also called constant or time zero costs. Real costs are used in life cycle cost analysis in combination with the discount rate to calculate net present value of costs.

Reconstruction activities – The removal and replacement of a pavement structure, including but not necessarily limited to its base and wearing course. Reconstruction involves a complete rebuilding of the pavement structure to new condition. Accumulated fatigue and functional (rutting) damage in a newly reconstructed pavement structure is zero.

Rehabilitation activities – Structural enhancements that extend the service life of an existing pavement and/or improve its load carrying capacity. Rehabilitation activities for Life Cycle Cost Analyses are limited to the pavement structure, and do not include the other items or activities in a project.

Resilient Modulus (M_R): A measure of the repeated-load modulus, sometimes called “dynamic” modulus of elasticity of roadbed soil or other pavement material. Stresses and strains are generated on test equipment using repetitive loading conditions. M_R should not be confused with another measure of the dynamic modulus known as the complex modulus (E^*). E^* is not presently used in mechanistic design methods used by DOT&PF.

Roadway Structure: A combination of select subbase, base course, and surface course materials placed on a subgrade that supports the traffic load and distributes it to the elements of the roadbed.

Routine annual preservation – Work that is planned and performed on a routine annual basis to maintain and preserve the condition of the pavement or to respond to specific conditions and events that restore the pavement to an adequate level of service; the former term is routine annual maintenance.

Selected Material: A suitable native material obtained from a source such as a roadway cut or borrow area, having specific characteristics.

Sensitivity analysis – An enhancement of the deterministic analysis technique in which the value of a single input parameter is changed over a range of values, while holding all other input parameters unchanged, and determining the resulting changes in net present value.

Serviceability: The ability, at time of observation of a pavement, to serve traffic that uses the facility.

Salvage value – The value of an investment alternative at the end of the analysis period.

Stability: The ability of asphalt paving mixture to resist deformation from imposed loads. Stability depends on internal friction and cohesion.

Stress Reduction Factor (SRF): The factor by which the stress of an applied load at the surface of a pavement is reduced at a given depth below the surface course.

Subbase (SB): The layer or layers of specified or selected material of designed thickness placed on a subgrade to support a base course (or in the case of rigid pavements, the Portland cement concrete slab). If the subgrade soil is of adequate quality, it may serve as the subbase.

Subgrade: The top surface of a roadbed upon which the pavement structure and shoulders are constructed.

Subgrade, Improved: Subgrade, improved is a working platform achieved (1) by the incorporation of granular materials or stabilizers such as asphalt, lime, or Portland cement, prepared to support a structure or a pavement system, or (2) any course or courses of select or improved material placed on the subgrade soil below the pavement structure. Subgrade improvement does not affect the design thickness of the pavement structure.

Surface Course (SC): One or more layers of a pavement structure designed to accommodate the traffic load, the top layer of which resists skidding, traffic abrasion, and the disintegrating effects of climate. The top layer of

flexible pavements is sometimes called “wearing course.”

Tandem Axle Load: The total load transmitted to the road by two consecutive axles extending across the full width of the vehicle.

Traveled Way: The portion of the roadway for the movement of vehicles, exclusive of shoulders and auxiliary lanes.

User costs – Costs that are accrued by the users of the facility during the construction, preservation, rehabilitation and everyday use of a roadway section.

User work zone costs – Roadway user/vehicle time delay costs associated with using a facility during periods of construction, preservation, and rehabilitation activities that generally restrict the capacity of the facility and disrupt normal traffic flow.

Wearing Course: Where thick pavements are required, the asphalt concrete pavement is sometimes placed as two layers, each differing in composition but sharing approximately the same fatigue properties (see also **Binder Course**). The wearing course is the top portion of asphalt concrete pavement, “tuned” to provide maximum resistance to abrasion wear (addresses tire-stud rutting), minimum surface roughness (addresses ride quality), minimum plastic deformation (addresses displacement rutting), and minimum permeability (addresses premature weathering and aging of the asphalt concrete).

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