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# PERPETUAL ASPHALT PAVEMENTS

## A Synthesis

PREPARED BY:

**DAVID E. NEWCOMB**
NATIONAL ASPHALT PAVEMENT ASSOCIATION

**RICHARD WILLIS**
NATIONAL CENTER FOR ASPHALT TECHNOLOGY

**DAVID H. TIMM**
NATIONAL CENTER FOR ASPHALT TECHNOLOGY

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# INTRODUCTION

## Background

The concept of Perpetual Pavements was introduced in 2000 by the Asphalt Pavement Alliance (APA). They defined a Perpetual Pavement as "an asphalt pavement designed and built to last longer than 50 years without requiring major structural rehabilitation or reconstruction, and needing only periodic surface renewal in response to distresses confined to the top of the pavement" *(APA, 2002)*. At that time, it was recognized that many well-built, thick asphalt pavements that were categorized as either full-depth or deep-strength pavements had been in service for decades with only minor periodic surface rehabilitation to remove defects and improve ride quality. The advantages of such pavements include:

1. Low life-cycle cost by avoiding deep pavement repairs or reconstruction,
2. Low user-delay costs since minor surface rehabilitation of asphalt pavements only requires short work windows that can avoid peak traffic hours, and
3. Low environmental impact by reducing the amount of material resources over the pavement's life and recycling any materials removed from the pavement surface.

A somewhat unified approach to designing Perpetual Pavements was adopted by a number of experts *(Thompson and Carpenter, 2004; Timm and Newcomb, 2006)* based on mechanistic-empirical concepts originally proposed by Monismith *(1992)* in the design of the I-710 freeway in California. The premise to this approach was that pavement distresses with deep structural origins could be avoided if pavement responses such as stresses, strains, and deflections could be kept below thresholds where the distresses begin to occur. Thus, an asphalt pavement could be designed for an indefinite structural life by designing for the heaviest vehicles without being overly conservative.

This contrasts to empirical methods that predated the Perpetual Pavement design approach. In those design procedures, greater volumes of heavy vehicles resulted in greater pavement thickness. This was due largely to the way these empirical methods were developed. For instance, the 1993 American Association of State Highway and Transportation Officials (AASHTO) Guide for the Design of Pavement Structures was based on the results of a road test conducted from 1958 to 1961. In this study, pavements were subjected to 1 million axle load applications, and failures were monitored

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over time. The heaviest single axle load used at the AASHO Road Test (30,000 lb) applied about 8 million equivalent single axle loads (ESAL) (18,000 lb equivalents) to the thickest asphalt section. Since that time, pavement structures have been designed for heavy traffic volumes that exceed the 8 million ESAL level by 25 times, thus forcing pavement designers to extrapolate the road test results far beyond the conditions for which they were developed. The result of this extrapolation was ever-increasing thickness with traffic volume, instead of recognizing the pavement thickness at which the heaviest loads could be sustained without additional structure. Thus, the idea of Perpetual Pavements came into existence as much to prevent over-design as to provide a long-life structure.

Since the time of the introduction of Perpetual Pavements in 2000, some of the important milestones have been:


*The Arkansas State Highway and Transportation Department received a 2009 Perpetual Pavement Award for Highway 134-North, Miller County, US 71, Section 2.*

- The Asphalt Pavement Alliance has presented 69 Perpetual Pavement awards from 2001 to 2009.
- The International Society for Asphalt Pavements dedicated a special session to Perpetual Pavements in 2002.
- Three international conferences have been held on the topic, one at Auburn University in 2004 and the others at Ohio University in 2006 and 2009.
- The Transportation Research Board held a workshop session on Perpetual Pavements in 2001.

- The Federation of European Highway and Road Laboratories (FEHRL) has undertaken a series of efforts to define long-life pavements *(Ferne and Nunn, 2004; Ferne, 2006)*.
- Three major national studies on Perpetual Pavements were initiated through the National Cooperative Highway Research Program (NCHRP).
- State studies on Perpetual Pavements have been or are currently being conducted in Kansas *(Romanoschi et al., 2006)*, Ohio *(Sargand et al., 2006)*, Wisconsin *(Crovetti et al., 2008)*, Pennsylvania *(Solaimanian et al., 2006)*, Oklahoma *(Gierhart, 2008)*, Texas *(Scullion, 2006)*, Michigan *(Von Quintus, 2001b; Von Quintus and Tam, 2001)*, New Mexico *(TRB, 2009)*, Illinois *(Thomson and Carpenter, 2004)*, Washington *(Mahoney, 2001)*, and California *(Monismith et al., 2009)*.
- Perpetual Pavement design workshops have been held in Ohio, Kansas, Oregon, Colorado, Texas, Minnesota, Tennessee, Georgia, Hawaii, Wisconsin, Oklahoma, and Indiana.
- The National Center for Asphalt Technology (NCAT) Test Track has pavement test sections designed as Perpetual Pavements which are instrumented to validate the design concepts.
- Two pavement design computer programs specifically for Perpetual Pavements have been developed at Auburn University.
- The concept of the endurance limit has been incorporated in the new American Association of State Highway and Transportation Officials (AASHTO) Mechanistic-Empirical Pavement Design Guide (MEPDG) (AASHTO, 2008).

State departments of transportation and local agencies are considering methods to incorporate the concepts of Perpetual Pavement design into their asset management strategies to more wisely spend their infrastructure funds.

## Objectives

This document aims to:

1. Capture the activities that have taken place over the past decade.
2. Synthesize the information in a way useful to providing guidance for Perpetual Pavement design and construction.
3. Provide a vision for further research and development to refine Perpetual Pavements.

## Scope

This synthesis begins with a discussion of Perpetual Pavement design for both new and rehabilitation designs. This is followed by a summary of material characteristics to be considered in design and performance. Construction practices are reviewed, followed by a discussion on the performance of long-lasting asphalt pavements.

# DESIGN

## Overview

Pavement engineers have been producing long-lasting asphalt pavements since the 1960s. Research has shown that well-constructed and well-designed flexible pavements can perform for extended periods of time *(Mahoney, 2001; Harvey et al., 2004)*. Many of these pavements in the past forty years were the products of full-depth or deep-strength asphalt pavement designs, and both have design philosophies that have been shown to provide adequate strength over extended life cycles *(APA, 2002)*. It is significant that these pavements have endured an unprecedented amount of traffic growth. For instance, from 1970 to 1998, the average daily ton-miles of freight increased by 580 percent, and the average freight loading continues to increase 2.7 percent per year *(D'Angelo et al., 2004)*. As the demand on existing pavements in the U.S. increases with potentially minimal funding for expansion and rehabilitation, efficient design of new and rehabilitated sections

*The Kentucky Transportation Cabinet received a 2009 Perpetual Pavement Award for Louisville-Tennessee Road, I-65, Hart County.*

through Perpetual Pavement design will become increasingly important. Congestion on the existing system is at a point that requires pavements that can be maintained with minimal disruption of traffic.

Full-depth pavements are constructed by placing asphalt layers on modified or unmodified soil or subgrade material. Deep-strength pavements consist of asphalt layers on top of a thin granular base. Both of these design scenarios allow pavement engineers to employ a thinner total pavement section than if a thick granular base were used. By reducing the potential for fatigue cracking and confining cracking to the upper removable/replaceable layers, many of these pavements have far exceeded their design life of 20 years with minimal rehabilitation; therefore, they are considered to be superior pavements *(APA, 2002)*.

Pavements which are either under-designed or poorly constructed exhibit structural distresses, such as fatigue cracking and rutting *(Mahoney, 2001),* before their design life is achieved. The successes seen in the full-depth and deep-strength pavements are the results of designing and constructing pavements that resist these detriments to the pavement's structure. In recent years, pavement engineers have begun to adopt

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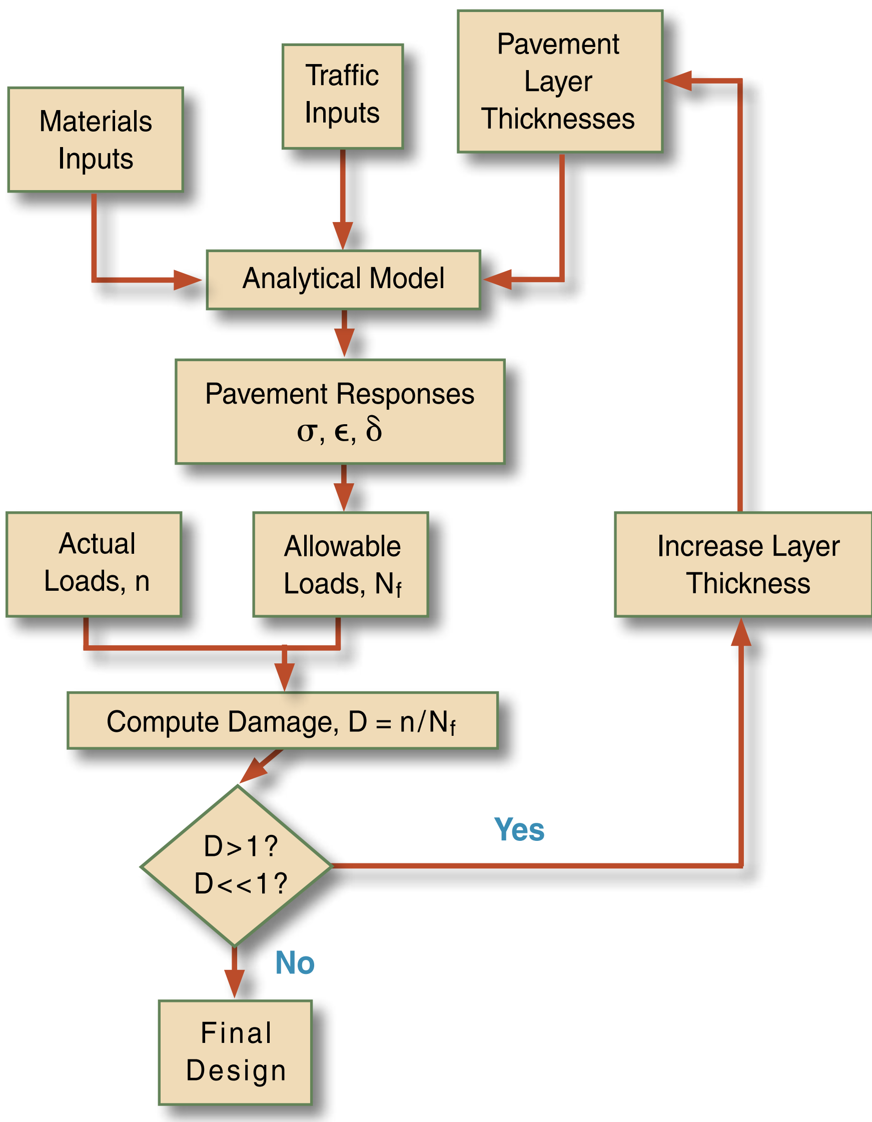
The approach to the design of long-life or Perpetual Pavements requires a different strategy than that which has normally been applied to pavement design in the past. Empirical pavement design must rely on relationships between observations of pavement performance, a scale that represents traffic, some gross indicator of material quality such as a structural coefficient, and the thickness of the layers. For a given level of material quality, the thickness of the pavement increases with increasing traffic. However, there comes a point beyond which the thickness of the pavement is more than adequate for the heaviest loads expected and any additional pavement results in an overly-conservative cross section and an unnecessary added cost. In addition to being extravagant from a cost standpoint, such an overuse of resources does not fit within an environmental sustainability framework. As a case in point, Huber et al. *(2009)* found that the 1993 AASHTO pavement design guide *(AASHTO, 1993)* typically over-designed pavements in Indiana by 1.5 to 4.5 inches which amounts to approximately 600 to 1800 tons of material per lane-mile beyond what is needed.

A better approach to the design of Perpetual Pavements is the mechanistic-empirical method. This approach uses the elements of a rational engineering analysis of the reaction of the pavement in terms of stresses, strains, and displacements in the context of the pavement's expected life. A flowchart showing a typical mechanistic-empirical design approach is shown in Figure 1. This is an iterative approach in which the pavement response in terms of stresses, strains, or deflections is used to estimate the allowable number of loads to failure ($N_f$) for a given condition of loading and material properties. The actual number of anticipated traffic loads (n) is divided by $N_f$ to define the degree of damage (D). The point at which the damage equals one is considered failure. This was originally defined by Miner *(1959)* as a way of describing metal fatigue. In many cases, engineers consider pavement failure to occur at either 20% fatigue cracking in the wheelpath or 0.5 inches of rutting *(Von Quintus, 2001a)*. Currently, there are existing M-E

Figure 1. Simplified Flowchart for M-E Design



pavement design methodologies *(AI, 1982; Monismith, 1992; KTC, 2007; Timm et al., 1998)*, but as the new M-E Pavement Design Guide (MEPDG) is being completed and implemented, more attention is being spent on proper material and pavement response characterization *(Timm and Priest, 2006)*. In Perpetual Pavement design, there are limiting strains below which damage does not occur, and thus damage is not accumulated. This concept is illustrated in Figure 2.

Most pavement engineers in the U.S. approach the idea of Perpetual Pavements with a 50 year structural design life in mind. However, while the structural integrity of the pavement should be intact during the entirety of the pavement's life, periodic resurfacing generally needs to occur within 20 years to improve friction, reduce noise, and mitigate surface cracking *(Newcomb et al., 2001)*. The basic concept of a Perpetual Pavement is illustrated in Figure 3. While the importance of proper design for a long-lasting pavement

Figure 2. Simplified Flowchart of Perpetual Pavement Design

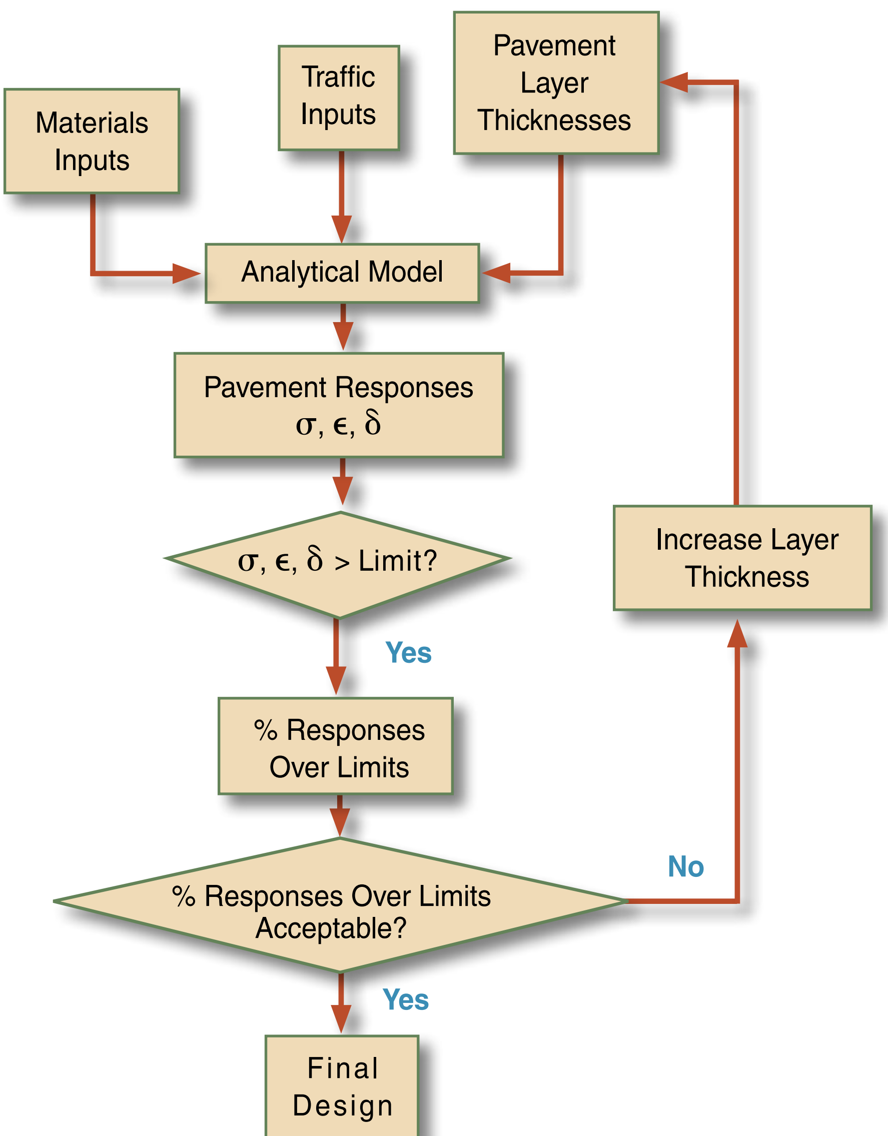
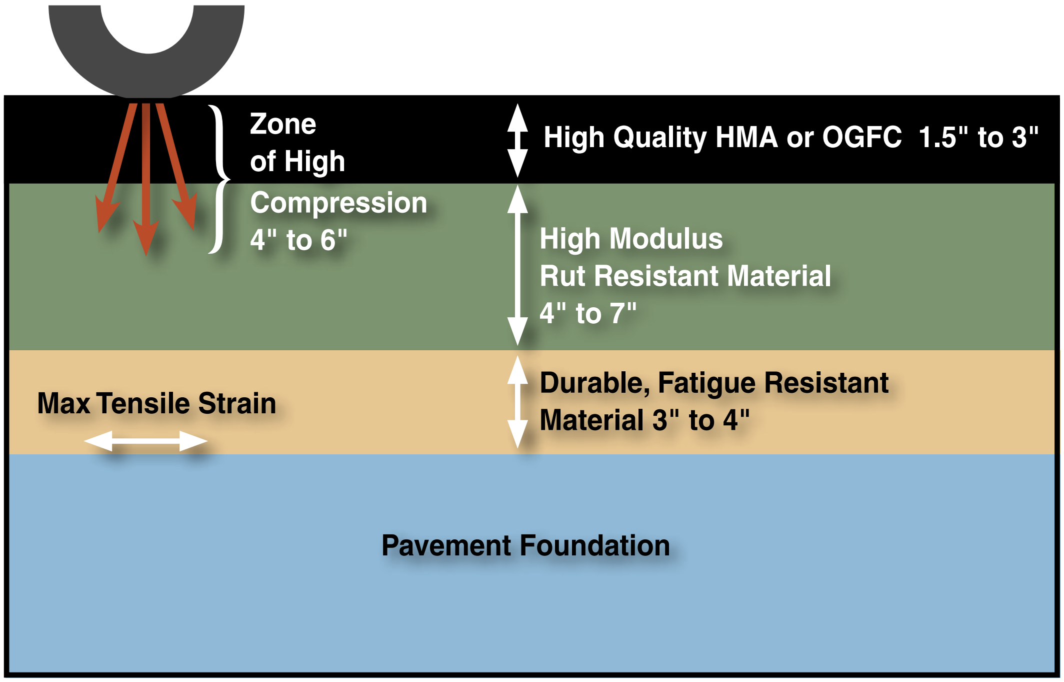


Figure 3. Perpetual Pavement Design Concept

*(Newcomb et al, 2000)*




must be recognized, one must also understand that design life is a function of the design requirements, material characteristics, construction practices, layer thicknesses, maintenance activities, and the failure criterion.

Ferne *(2006)* expanded upon this idea by saying a "long-life pavement is a well-designed and constructed pavement that could last indefinitely without deterioration in the structural elements provided it is not overlooked and the appropriate maintenance is carried out." Pavement performance is more than a function of design. Trafficking, climate, subgrade and pavement parameters (such as modulus), pavement materials, construction, and maintenance levels all contribute to how a pavement will perform over the course of its life *(Von Quintus, 2001a; Walubita et al., 2008)*.

Assuming that pavements will be constructed adequately, engineers approach designing Perpetual Pavements using the following philosophy *(Walubita et al., 2008; Merrill et al., 2006)*:

- Perpetual Pavements must have enough structural integrity and thickness to preclude distresses such as fatigue cracking, permanent deformation, and structural rutting.
- Perpetual Pavements must be durable enough to resist damage from traffic (such as abrasion) and the environment.

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While one might think pavements designed to last longer would incur more or have higher initial costs than pavements with shorter life-cycles, it has been shown that Perpetual Pavements have the following benefits *(Timm and Newcomb, 2006):*

- They provide a more efficient design, eliminating costly overly conservative pavement sections.
- They eliminate reconstruction costs by not exceeding a pavement's structural capacity.
- They lower rehabilitation-induced user delay costs.
- They reduce use of non-renewable resources like aggregates and asphalt.
- They diminish energy costs while the pavement is in service.
- They reduce the life-cycle costs of the pavement network.

In order to provide the above advantages, it is necessary to know what thickness of pavement section will support the heaviest anticipated traffic loads without grossly over-designing the pavement. Research has shown that this can be identified mechanistically by identifying the stresses, strains, or displacements in a structure which are low enough to avoid the initiation of cracking or rutting deep in the pavement structure. These thresholds are often referred to as limiting pavement responses.

# Limiting Pavement Responses

Perpetual Pavement design requires defining the point in critical pavement responses below which structural damage does not accumulate. Practically, this means that structural damage is considered to be zero below this point in the M-E design process. If the pavement can be designed so that the vast majority of loads expected produce stresses, strains, or displacements lower than those which would cause structural damage, then the design can be said to be a Perpetual Pavement. Currently, most approaches to Perpetual Pavement design focus on pavement responses related to structural rutting and bottom-up fatigue cracking.

## Structural Rutting

Structural rutting occurs when the overall strength of the pavement structure is such that large permanent deformation can take place either in the granular base or subgrade under the imposed traffic. Structural rutting failures are relatively rare in modern pavement structures, but require very expensive major rehabilitation or reconstruction when they do occur. Studies at the National Center for Asphalt Technology (NCAT) Pavement Test Track *(Brown et al., 2002)* and by Rolt *(2001)* have shown that thick pavement structures tend to prevent structural rutting in the subgrade and limit rutting to the surface layers of the pavement structure. The difference between structural rutting and surface rutting is that surface rutting is confined to the upper few inches of the pavement and can be remedied with removal and replacement of the pavement surface.

Harvey et al. *(2004)* and Walubita et al. *(2008)* elected to use the vertical compressive strain at the top of the subgrade as the limiting design parameter. Their approach was to use a value of 200 $\mu\epsilon$ (microstrain) as the limiting strain for the subgrade criterion. It was reasoned that plastic deformation in the lower layers would not occur if the compressive strain in the subgrade was kept below this value. This is achieved by increasing either the thickness of the total pavement structure or the stiffness of one or more of the pavement layers.

A different approach was proposed by researchers at the University of Illinois *(Bejarano et al., 1999; Bejarano and Thompson, 2001)*. They used the ratio of the subgrade stress to the unconfined compressive strength of the soil, known as the Subgrade Stress Ratio (SSR). They noted that for clay soils in their study, the transition from a stable to an unstable condition occurred when the SSR was in the range of 0.50 to 0.60. For design purposes, they recommend using an SSR of 0.42, although they acknowledge that this rutting criterion is not well established. However, this approach allows the designer to account for the strength of the subgrade in determining the limiting response.

## Fatigue Cracking

When bottom-up fatigue cracking occurs in the asphalt pavement, it may eventually propagate to the surface affecting all the layers of the pavement structure allowing water to change the material properties of the unbound material layers. This phenomenon results in accelerated surface deterioration, pumping, and rutting.

Fatigue cracking typically begins due to high repeated strains at the bottom of an asphalt layer from heavy loads *(Huang, 1993)*. Research has shown that limiting the horizontal strains at the bottom of the asphalt base can help control fatigue cracking *(Shook et al., 1982; AI, 1982)*. A schematic of the fatigue cracking mechanism driven by tensile strain at the base of the asphalt pavement is shown in Figure 4.

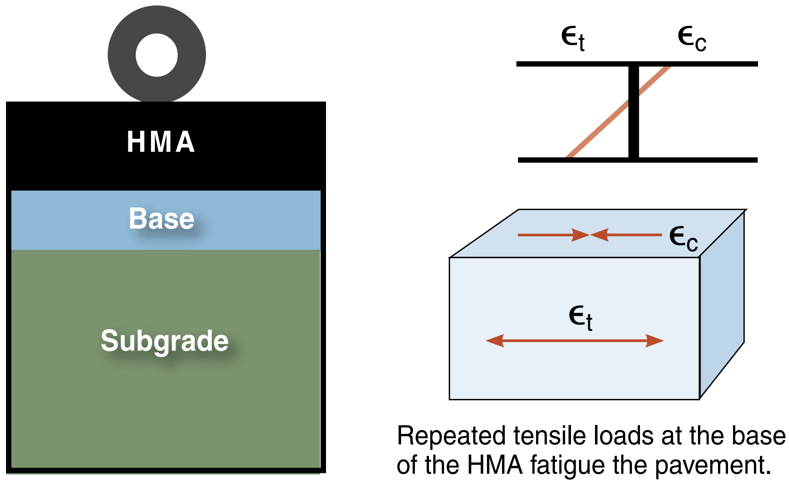
One way to decrease the probability of bottom-up fatigue cracking is to increase the thickness of the pavement structure. Thick pavements have been shown to limit cracking to the surface of pavements by reducing the maximum strain at the bottom of the asphalt pavement *(APA, 2002; Merrill et al., 2006; Romanoshci, 2008; Al-Qadi et al, 2008; Newcomb et al., 2000; St. Martin et al., 2001)*. The longitudinal strain at this pavement location has proven to be critical in thinner pavements, and in a fully-bonded pavement, it is always the location of highest tensile strain *(Al-Qadi et al., 2008)*.

In a 2006 survey of accelerated pavement testing (APT) facilities in the United States, a large majority of the responding facilities measured horizontal strain at the base of the asphalt layer to study fatigue life *(Willis, 2008)*. Perpetual Pavement projects such as the I-5 in Oregon *(Estes, 2005; Sholz et al., 2006)* and the Marquette Interchange in Wisconsin, have incorporated measuring strain at the base of the asphalt layer into their research *(Hornyak et al., 2007)*. In Ohio, U.S. 30 in Wayne County was designed as 16.25 inches of asphalt over six inches of granular base and was instrumented to measure displacements, strains, pressure, temperature, and moisture as well as groundwater level *(Sargand et al., 2006; Laio and Sargand, 2009)*.

When the tensile strain at the bottom of the asphalt layer is reduced, the critical location for tensile strains in pavements is relocated from the base of the pavement to the surface of the structure where tire interaction and binder aging contribute to hardened and weaker wearing courses that are prone to top-down cracking *(Mahoney, 2001; Rolt, 2001)*. At this point, since the distresses in the pavement are confined to the wearing course, it is possible to avoid deep structural maintenance and focus on functional maintenance such as skid resistance and ride quality *(Ferne, 2006)*. To eradicate the surface cracks, a "mill and fill" maintenance plan is appropriate for extending the pavement's life *(Mahoney, 2001)*.

Monismith and McClean *(1972)* concluded that there was a strain below which there is no fatigue damage. This is sometimes referred to as the fatigue endurance limit (FEL) or the limiting strain criterion. Even before discussion began on the design of Perpetual Pavements, Nishizawa et al. *(1996)* concluded that fatigue cracking does not occur when the tensile strain at the bottom of the asphalt pavement is held to less than 200 $\mu\epsilon$, and they suggested a design value of 150 $\mu\epsilon$.

Figure 4. Fatigue Cracking Schematic




Repeated tensile loads at the base of the HMA fatigue the pavement.

Research at the NCAT Test Track *(Willis et al., 2009)* has shown that pavements can withstand bending strains greater than 70 to 100 $\mu\epsilon$. In fact, Willis *(2009)* showed that there is considerable conservatism associated with limiting strains in this range. Willis and Timm *(2009)* postulated that a design strain could be selected on the basis of the ratio of computed pavement strains to a field adjusted laboratory fatigue endurance limit. In this approach, a maximum fatigue ratio at a certain cumulative percentile of all expected strains could be used as a design value. Thus, if one wanted to design the pavement at a 95 percent confidence level, then the strain at this point would be divided by the FEL. The authors recommend a design strain ratio of 2.45 in this case. For instance, if the laboratory fatigue endurance limit was determined to be 125 $\mu\epsilon$, then 95 percent of the anticipated strains in the pavement should be less than 307 $\mu\epsilon$ (307/125 = 2.45).

The use of Perpetual Pavement design in China provided an efficient means of dealing with traffic loads that, on average, are double the legal limit in the U.S *(Yang et al., 2006)*. These researchers noted that instead of increasing pavement thickness under very heavy loads in China, Perpetual Pavement design actually resulted in a decrease from 20 to 15 inches of asphalt by eliminating unnecessary conservatism through the application of FEL of 125 $\mu\epsilon$ rather than 70 $\mu\epsilon$.

# Perpetual Pavement Design Approaches

There are a variety of pavement design approaches that have been adopted to develop Perpetual Pavement designs. These have all involved some aspect of M-E design in an effort to characterize and minimize pavement damage.

## High Volume Pavements

As the idea of Perpetual Pavement began to gain momentum in 2000, it became evident that M-E design procedures needed to be modified to adapt to the concept. The

Asphalt Pavement Alliance worked with Auburn University to develop PerRoad *(Timm, 2008)*, a computer analysis program used to design Perpetual Pavements using the M-E design philosophy. The program couples layered elastic analysis with a statistical analysis procedure (Monte Carlo simulation) to estimate stresses and strains within a pavement *(Timm and Newcomb, 2006)*. In order to predict the strains which would prove detrimental for fatigue cracking or structural rutting, PerRoad requires the following inputs:

- Seasonal pavement moduli and annual coefficient of variation (COV)
- Seasonal resilient moduli of unbound materials and annual COV
- Thickness of bound materials and COV
- Thickness of unbound materials
- Load spectrum for traffic
- Location for pavement response analysis
- Magnitude of limiting pavement responses
- Transfer functions for pavement responses exceeding the user-specified level for accumulating damage

PerRoad generally follows the M-E design process described in Figure 2. The Monte Carlo simulation is simply a way of incorporating variability into the analysis to more realistically characterize the pavement performance. The output for PerRoad consists of an evaluation of the percentage of load repetitions lower than the limiting pavement responses specified in the input, an estimate of the amount of damage incurred per single axle load, and a projected time to when the accumulated damage is equal to 0.1 (Recall, D=1.0 is considered failure). On high volume pavements, the critical parameter is the percentage of load repetitions below the limiting strains. It is generally recommended that the designer strive for a value of 90 percent or more on high volume roads.

California constructed one of the first intentionally-designed Perpetual Pavements in the U.S. on the I-710 freeway near Long Beach, California *(Monismith and Long, 1999a)*. The full-depth asphalt portions of this project consisted of a total of 12 inches of asphalt mix and had a 3-inch bottom layer in which the asphalt content was raised by 0.5 percent over optimum to 5.2 percent. This increased binder content could improve the fatigue life of the pavement *(Harvey, et al., 2004)*, however it probably better serves to improve the durability of the asphalt mix in this layer. The intermediate 6 inches were constructed with the same aggregate gradation and binder as the bottom layer, but the asphalt content was 4.7 percent. The use of a relatively stiff unmodified asphalt grade in the intermediate layer helps guard against rutting. The upper 3 inches of the pavement structure were constructed using a heavily polymer modified binder, and this was below a one-inch open-graded friction course. In tests using the California Accelerated Pavement Test Heavy Vehicle Simulator, this material was found to have less than half the rutting of other asphalt mixtures.

Thompson and Carpenter *(2004)* presented Perpetual Pavement design concepts in the context of laboratory work done at the University of Illinois. In this case the model employed to represent the pavement was a finite element program called ILLI-PAVE in which 18,000-lb and 20,000-lb axle loads served as the loading condition. These researchers reasoned that this would be the extreme case in hot weather as these loads would represent the worst condition with very few loads being greater than this. Their work showed that up to 30 percent of the fatigue life of the pavement could be consumed, yet if the remaining strains were below the FEL, there would be no fatigue cracking. They went on to verify these results with field deflection measurements. From these, they were able to conclude that many existing pavements could be classified as Perpetual Pavements.

A study in Texas (Walubita et al., 2008) concluded that the Flexible Pavement System in use by the state DOT could be used to model Perpetual Pavements, and that optimization of the existing system could lead to a reduction in pavement thickness by about 4 inches. In another paper (Walubita et al., 2009), they compared actual pavement responses from a section consisting of 17 inches of asphalt over 8 inches of cement treated base to a FEL of 70 $\mu\epsilon$ and a subgrade limiting strain of 200 $\mu\epsilon$. They found that the asphalt layer could have been 3 inches thinner and still would have met the Perpetual Pavement definition.

…they were able to conclude that many existing pavements could be classified as Perpetual Pavements.

Von Quintus *(2001b)* developed one of the earlier approaches to Perpetual Pavement design for the state of Michigan. In the development of the design tables for this effort, he used low levels of predicted distresses for criteria rather than limiting strains. Von Quintus went on to suggest rehabilitation strategies to carry the pavement for a period of 40 years. In the spirit of Perpetual Pavements these rehabilitation strategies were mill and fill operations at years 15 and 30, except for the lowest level of traffic where they were scheduled for years 32 and 40.

The AASHTO Mechanistic-Empirical Pavement Design Guide *(AASHTO, 2008)* can be used for Perpetual Pavements with regards to the fatigue endurance limit. This design procedure is currently being calibrated and adopted by a number of states across the U.S. It predicts the accumulation of a variety of pavement distresses over a user-prescribed analysis period. Based on information from NCHRP Project 9-38 *(Prowell et al., 2006),* Witczak et al. *(2006)* incorporated an optional FEL ranging between 75 and 250 $\mu\epsilon$. Researchers have begun to investigate the use of the MEPDG in conjunction with the fatigue endurance limit to optimize pavement designs *(Behbahais et al., 2009; Tarefoler et al., 2009)*. In fact, Willis and Timm *(2009)* found good agreement between PerRoad and the MEPDG in terms of thickness requirements when the FEL was employed.

There are a number of non-instrumented test sites where the performance of Perpetual Pavement designs is being observed. Rosenberger et al. *(2006)* describe a trial Perpetual Pavement constructed on a by-pass around Bradford, Pennsylvania that consisted of 13.5 inches of asphalt over 13 inches of granular base which was designed using PerRoad. Three test sections were constructed in Ontario on Highway 402, near Sarnia *(Lane et al., 2006)*. These sections included a Perpetual Pavement with a rich base, a Perpetual Pavement with a Superpave mixture as the base, and a conventionally designed pavement section. The Perpetual Pavement sections, as validated by PerRoad, were 13.4 inches of asphalt over 21.6 inches of granular base, and the conventional section, designed according to the 1993 AASHTO design guide, had 9.4 inches of asphalt over 21.6 inches of granular material.

## Lower Volume Pavements

There has also been considerable interest in applying Perpetual Pavement concepts to low-volume roads. Muench et al. *(2004)* compared a long-lasting low volume road in Washington state consisting of 5 inches of asphalt mix over 12 inches of crushed stone base with another one having 3 inches of asphalt mix over 6 inches of crushed stone. They found the Perpetual Pavement design to provide a much longer-lasting performance than the conventional pavement. Driscoll *(2009)* describes the design of a county road in Ohio using 12 inches of asphalt over 6 inches of granular base, the cost of which was actually lower than the county's estimate for a conventional pavement. A medium volume road in Hamilton, Ontario was designed as a Perpetual Pavement using the 1993 AASHTO design guide and verifying it with the PerRoad program *(Uzarowski et al., 2008).*

A means of designing Perpetual Pavements for low-volume roads was developed by the Asphalt Pavement Alliance called PerRoadXpress *(Timm, 2008b)*, which is an easy-to-use program. This program was derived from running a large number of low to medium volume pavement design cases in PerRoad. In this case, the single input screen simply consists of:

- Functional classification of the road (urban or rural collector)
- Two-way Annual Average Daily Traffic
- The anticipated traffic growth
- The soil classification and/or soil modulus
- The aggregate base thickness
- The asphalt mixture modulus

In the case of low-volume roads, the PerRoad 3.3 approach of using limiting strains would result in overly thick pavements because a low number of heavy vehicles such as garbage trucks and delivery vans would dictate the design. Instead, the PerRoadXpress designs were determined by limiting the damage occurring over a 30-year period to a value of 0.1 or less (Recall, D=1.0 is the point of failure) *(Timm et al., 2006)*.

The output of the required asphalt layer thickness appears on the same screen as the input. Like PerRoad, the help file serves as the users' manual and can be accessed by simply pressing the F1 key while the cursor is in any dialog box.

## High-modulus Pavements

High-modulus pavements offer a means to use less material and reduce the cost of Perpetual Pavements. In this design approach, a very stiff asphalt mixture is used as the base and intermediate layers. High-modulus asphalt mixes are in use in a number of European countries in both heavy duty and structural rehabilitation projects where it is desirable to minimize the impact of grade change, yet still ensure pavement longevity. In these pavements, the base course mix is made with a stiff binder combined with a relatively high binder content and low void content. This allows for a reduction in thickness between 25 and 30 percent in the pavement structure *(Corte, 2001; EAPA, 2009).*

These structures are beginning to be investigated in the U.S. A Virginia laboratory study of a dense-graded asphalt mixture with stiff asphalt (PG70-22 and PG76-22) showed that fatigue characteristics of the mix improved if the binder content was increased while the rutting behavior remained stable *(Maupin and Diefenderfer, 2006).* It was the conclusion of this study that high-modulus pavements rate further study.

## Design for Pavement Rehabilitation

The primary mode of pavement construction in the U.S. for the past 30 years has been preservation and rehabilitation. D'Angelo et al. *(2004)* noted that construction of new roadways in the U.S. increased by only 6 percent between 1970 and 1998. In order to ensure the longevity and vitality of the nation's highway system, it is critical that the existing pavements be evaluated to ascertain whether they meet or can be upgraded to Perpetual Pavements. This has been the major effort of the Second Strategic Highway Research Program (SHRP2) under Project R23 *(Jackson et al., 2009).* This project has examined methods for rapid renewal of roadway pavements with special attention to long-life designs. The methods included in this research effort include asphalt overlays of existing asphalt pavements and asphalt over rubblized concrete pavements.

As discussed in the Introduction, Perpetual Pavements have been unintentionally designed, constructed, and maintained for decades. Proof of this can be found in the 69 pavements honored since 2001 in the Asphalt Pavement Alliance's Perpetual Pavement Awards. As a part of the criteria for this award, pavement rehabilitation must not result in structural improvement over a period of at least 35 years. Thus, it has been standard for these pavements to be evaluated for structural soundness with the resulting overlay improving the functionality of the pavement surface.

The rehabilitation of I-287 in New Jersey is an excellent example of the process for evaluation and design of an overlay to an existing pavement. The New Jersey DOT investigated distresses that developed on the 26-year old pavement surface *(Fee, 2001).* The structure was a 10-inch thick asphalt pavement that had received a minimum of maintenance. The surface showed fatigue cracking, longitudinal cracking in the wheelpaths, and ruts deeper than one inch. A detailed examination of the pavement structure showed that none of the distresses extended more than 3 inches into the depth of the asphalt. As a result, the decision was made to mill off the top 3 inches and replace it with a total of 4 inches of asphalt surfacing. This work was done in 1994, and a pavement survey done in 2001 showed no signs of cracking or rutting *(Rowe et al., 2001).* Another approach to the upgrading of existing pavements to long-life pavements is explored in Loizos *(2006).*

Rubblization of concrete pavement with an asphalt overlay is a popular rehabilitation approach with seven states having 20 or more rubblizing projects, and another 10 states having five or more projects *(Von Quintus et al., 2007).* Experience has shown

> In order to ensure the longevity and vitality of the nation's highway system, it is critical that the existing pavements be evaluated to ascertain whether they meet or can be upgraded to Perpetual Pavements.

that attention must be given to the subsurface site conditions. Wet and weak subgrades must be drained in order to avoid conditions where the rubblization process may further weaken pavement structure and cause premature failure. It is also advisable to reduce the impact effort in such areas to avoid over-fracturing the concrete slabs. In some cases, it may be prudent to employ a crack and seat strategy if rubblization creates problems with subgrade weakening.

Vavrik et al. *(2009)* describe how a deteriorated 14-inch concrete pavement on the Illinois tollway was rubblized and overlaid with six inches of asphalt as the initial design for stage construction. This approach is predicated on the findings of Thompson and Carpenter *(2004 and 2006)* that as much as 30 percent of fatigue life consumption can occur without damaging the ability to achieve a Perpetual Pavement. After the first 10 years, two inches of the surface will be milled and six more inches of asphalt will be constructed. This will provide sufficient structure for a Perpetual Pavement.

Most of the previously mentioned I-710 freeway project in California was comprised of an asphalt overlay of a broken and seated concrete pavement *(Monismith and Long, 1999b).* The overlay of the cracked and seated concrete is a total of 8 inches thick, and does not have the fatigue resistant bottom layer. The cracked and seated concrete provides a stiff foundation for the asphalt and prevents the excessive bending associated with bottom-up fatigue cracking. There was an asphalt-saturated fabric placed over a one-inch leveling course on top of the concrete to guard against reflective cracking. Other than this, the materials used in the concrete overlay were the same as those planned for the full-depth pavement. As with the full-depth section, a one-inch open-graded friction course was placed on top.

The use of Perpetual Pavements in rehabilitation of concrete pavements has also been used internationally. Lande et al. *(2006)* report that the Khandahar to Hurat highway in Afghanistan was a concrete pavement originally constructed in the 1960s. War and a lack of maintenance had made the road nearly unusable in some places and very rough in others. After evaluating three alternatives for improving the road, the authors determined that the use of an asphalt overlay in a Perpetual Pavement design provided the lowest life cycle cost.

Bendana et al. *(2009)* describe the design and construction of Perpetual Pavement and standard asphalt pavement overlays of a section of Interstate 86 in western New York. The Perpetual Pavement consisted of nine inches of asphalt over nine inches of rubblized concrete while the standard pavement was eight inches of asphalt over ten inches of rubblized concrete. The conventional section was constructed in 2006 and the Perpetual Pavement was built in 2008. These sections were instrumented for in-situ measurements of pavement responses under loads, and they are being monitored.

A portion of the I-5 experiment in Oregon is a 12-inch thick asphalt section constructed over a rubblized continually reinforced concrete pavement (CRCP) and a jointed reinforced concrete pavement (JRCP) *(Renteria and Hunt, 2006; Sholz et al., 2006)*. The design for this pavement followed the NAPA design for rubblized concrete pavement *(Decker, 2006)* which recommended 12 inches of asphalt over the eight-inch rubblized concrete layer. The test site located on the JRCP is instrumented to monitor pavement responses and environmental conditions.

Von Quintus and Tam *(2001)* developed a procedure for designing long-life asphalt pavements over rubblized concrete for Michigan that followed the same approach he used for asphalt pavements. The thicknesses for these asphalt pavements ranged from 6 inches to 11 inches with mill and fill rehabilitation at years 20 and 32.

PerRoad 3.3 *(Timm, 2008)* may also be used to design asphalt pavements over fractured concrete pavements. This only requires that the second layer be specified as rubblized, cracked and seated, or broken and seated concrete pavement. Beyond that, it follows the same mechanistic design process for a Perpetual Pavement as described above.

# Summary

The mechanistic-empirical design process has provided a convenient format for the design of Perpetual Pavements. By simply modifying the transfer functions to allow for the input of limiting strains, pavements can be designed to account for instances where pavement responses do not add to the cumulative structural damage. The design of Perpetual Pavements has expanded to provide flexibility in a variety of applications including high-volume and low-volume pavements, high-modulus pavements, and the rehabilitation of flexible and rigid pavements.

# MATERIALS

Unlike strictly empirical pavement design procedures, mechanistic-empirical design incorporates the properties of the pavement layer materials directly as input. This requires methods to determine these properties and the means to understand how they fluctuate with environmental conditions. Defining the properties and how they vary is crucial to Perpetual Pavement design in that most of the damage will occur when the pavement structure is weakest and the loads are the highest, and it is the goal to minimize this damage. This section will focus on the characterization of the foundation and the asphalt layers, and the desirable characteristics for Perpetual Pavements.

## Foundation

The pavement foundation is critical to the construction and performance of a Perpetual Pavement. During construction, the foundation provides a working platform that supports the equipment placing the asphalt layers and provides resistance to the asphalt compactors so that the asphalt layers can achieve the desired density. Throughout the performance period, the foundation provides support to the traffic loads and is crucial to reducing variability from season to season due to freeze-thaw and moisture changes. Proper design and construction of the foundation are keys in preventing volume changes due to wet-dry cycles in expansive clays and freeze-thaw cycles in frost-susceptible soils.

*The Missouri Department of Transportation receivd a 2009 Perpetual Pavement Award for MO 47, Franklin County.*

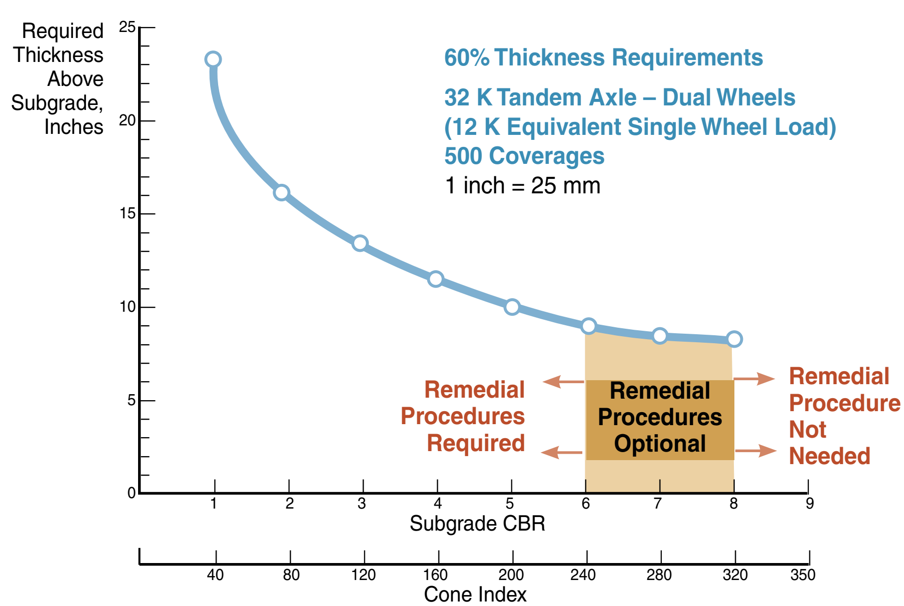
Several northern states incorporate frost design into their pavement structures in areas where the soils and conditions may lead to thaw weakening or non-uniform frost heave. In the presence of such soils, these states generally require that the total pavement structure thickness equal or exceed 50 percent of the expected design frost depth. This requirement is generally taken to be a minimum. Results from the AASHO Road Test and other countries suggest that a depth of up to 70 percent may be required. Such criteria generally require that the pavement structure be constructed of non-frost susceptible materials.

A pavement foundation may be comprised of compacted subgrade, chemically stabilized subgrade, or stabilized granular material, as well as unstabilized granular material such as crushed rock or gravel. Regardless of the kind of material employed, the foundation should meet some minimum requirement for stiffness throughout construction as well as during the life of the pavement *(Thomas et al., 2004)*. Depending upon site conditions and pavement design, this may require the chemical or mechanical stabilization of soils or base course materials. Terrel and Epps *(1979)* provide excellent guidance on the selection of the stabilization procedures for unbound materials. Furthermore, the site and climate may dictate that drainage features be included in the pavement design, and guidance on subsurface drainage may be found in the FHWA drainage manual *(Moulton, 1980)*.

The Illinois DOT (IDOT) has put forth guidance in their Subgrade Stability Manual *(IDOT, 1982)*. For constructability, Illinois requires a subgrade to have a minimum California Bearing Ratio (CBR) of about 6 to avoid excessive deformation during the construction of subsequent granular layers. Figure 5 shows that in Illinois, remedial action is required if the soil CBR is less than 6, it is optional between a CBR of 6 and 8, and it is considered unnecessary above 8. The remedial procedures provide a working platform adequate to prevent overstressing the subgrade, facilitate paving operations, and are sufficiently stable to minimize the development of surface rutting from construction traffic. The most frequently used procedure is to lime-modify the fine-grained subgrade soils that predominate in Illinois *(IDOT, 2002)*. Undercut and backfill with granular material is

Figure 5. Illinois Granular Thickness Requirement for Foundation

(IDOT, 1982)



also a commonly used procedure along with the occasional application of geofabrics. The required thickness above the subgrade is typically 12 in. For subgrade strengths less than a CBR of 4, the thickness is increased as per Figure 5.

Seasonal modulus adjustment factors are used in Washington and Minnesota for subgrade and overlying granular materials to characterize their respective behaviors during the design life. Seasonal modulus adjustment factors for unbound materials differ between eastern and western Washington State as shown in Table 1 *(Pierce and Mahoney, 1996)*. The seasons in Washington are assumed to be of equal length, and the base season is the summer with a multiplication factor of 1.00. The seasonal adjustment factors in Table 1 reflect backcalculated modulus values under pavements with asphalt thicknesses ranging from thin to thick. A slightly different approach is taken in Minnesota where the seasons are considered to be of unequal lengths as shown in Table 2, and the base season is in the fall. Because the progression of thawing results in different behavior in the upper and lower regions of the pavement, the spring period is divided into early and late spring. Ovik, et al. *(1999)* determined these seasonal factors from data collected at the Minnesota Road Research Project. The weakest condition for granular base materials is in the early spring, and for the subgrade it is in the late spring. The very high multiplication factors for the winter reflect frozen conditions. In the design of Perpetual Pavements, it is important to know how seasonal changes in the moduli of unbound materials may affect the response of the pavement. In other words, it may be necessary to consider the worst condition in order to preclude undue damage during a given season.

### Table 1. Seasonal Adjustment Factors for Unbound Materials Used in Washington State

(Pierce and Mahoney, 1996)

| Location | Material | Season | | | |
|---|---|---|---|---|---|
| | | Spring | Summer | Fall | Winter |
| Eastern WA (cold winters, hot summers) | Base | 0.65 | 1.00 | 0.90 | 1.10 |
| | Subgrade | 0.90 | 1.00 | 0.90 | 1.10 |
| Western WA (wet winters, mild summers) | Base | 0.85 | 1.00 | 0.90 | 0.75 |
| | Subgrade | 0.85 | 1.00 | 0.90 | 0.85 |

### Table 2. Seasonal Adjustment Factors for Mn/ROAD

(After Ovik, et al., 1999)

| Month | Late Nov., Dec., Jan., Feb. | March | April, May | June, July, August | Sept., Oct., early Nov. |
|---|---|---|---|---|---|
| Asphalt (120/150 pen asphalt) | 2.5 | 2.1 | 1.3 | 0.37 | 1.0 |
| Granular Base | 28 | 0.65 | 0.80 | 1.0 | 1.0 |
| Subgrade | 22 | 2.4 | 0.75 | 0.75 | 1.0 |

Nunn et al. *(1997)* encourage the use of in-situ testing for pavement foundation materials, and a number of devices for accomplishing this are reviewed by Thomas et al. *(2004)*. The British *(Nunn et al., 1997)* formulated an end-result specification founded on nuclear density tests and surface stiffness as measured by a portable dynamic plate bearing test. The foundation design practice in the UK is shown in Table 3. The CBR of the subgrade dictates the thickness of the overlying granular layers called the capping and subbase layers. For a subgrade CBR of less than 15, a minimum six-inch thickness of subbase is required. Capping material may be considered similar in quality to a lower quality base course material in the U.S., and the subbase may be considered a high quality base material. TRL set end-result requirements for the pavement foundation, both during and after its construction. Under a falling weight deflectometer (FWD) load of 9000 lb, a stiffness of 5800 psi was required on top of the subgrade and 9500 psi was required at the top of the subbase.

Table 3. Transport Research Laboratory Foundation Requirements

(Nunn et al., 1997)

| **Subgrade CBR** | **< 2** | **2 - 5** | **> 5** |
|---|---|---|---|
| Subbase Thickness, in. | 6 | 6 | 9 |
| Capping Thickness, in. | 24 | 14 | — |

The German Ministry of Transportation *(1989)* requires a minimum subgrade surface modulus of about 6500 psi when tested using a static plate-bearing test with a 12-in diameter plate. At the top of the subbase layer, they require about 17,000 psi for light traffic conditions and about 26,000 psi for heavy traffic.

The French *(LCPC, 1992)* use an end-result specification for the constructed road foundation. For support of construction traffic, either of the two following criteria must be met: a deflection of less than 0.1 in under a 14 ton axle load, or a plate bearing test modulus of more than 7300 psi. For service conditions, the required subbase stiffness is tied to the strength of the subgrade.

The design and construction of a strong, stable and consistent foundation is essential to a Perpetual Pavement. The initial concern is support of construction traffic and a firm layer for providing a reaction to compaction efforts. Long-term support of traffic loads and minimization of volume change are crucial to performance. Thus, guidelines are needed for assessment of stiffness at the time of construction, required stiffness for long-term performance as input to mechanistic design, and provisions to minimize volume change due to expansive behavior or frost heave.

# Asphalt Mix Design and Materials

It is important to use the proper asphalt mixtures in the layers of a Perpetual Pavement keeping in mind that each layer serves specific functions. For instance, the lowest layer must provide excellent durability and the resistance to fatigue cracking. The intermediate layer provides both durability and rutting resistance, and the surface must be designed to withstand traffic and direct exposure to the environment. The use of reclaimed asphalt pavement (RAP) and recycled asphalt shingles (RAS) can help stiffen mixtures in providing rutting resistance, possibly without the addition of polymer modifiers. In an effort to provide guidance on the best application for various types of mixtures according to traffic level and the lift thickness, Newcomb and Hansen *(2006)* provided the information in Table 4.

Simply increasing pavement thickness is not a guarantee that the pavement will have a long service life. Washington State's study of long-lasting pavements showed that in many cases pavements with shorter life-cycles in Washington were thicker than more fatigue resistant pavement structures *(Mahoney, 2001)*. Other studies have shown that while increasing the thickness of a pavement will decrease the tensile strain at the bottom of the asphalt layer, the magnitude by which this reduction occurs is mix dependent *(Romanoschi, 2008)*. Thus, it is important to specify the right mixture for the right application in the pavement.

Table 4. Mix Type Selection Guide for Perpetual Pavements

(Newcomb and Hansen, 2006)

| Pavement Layer | Mix Type | NMAS, mm (in.) | Lift Thickness Range, mm (in.)[1] | Traffic Level, MESAL[2,3] | | |
|---|---|---|---|---|---|---|
| | | | | <0.3 | 0.3-10 | >10 |
| Base | Dense, Fine | 37.5 (1-1/2) | 110-150 (4.5-6) | √√ | √√ | √√ |
| | | 25 (1) | 75-100 (3-4) | √√ | √√ | √√ |
| | | 19 (3/4) | 60-75 (2.5-3) | √√ | √√ | √√ |
| | Dense, Coarse | 37.5 (1-1/2) | 150-190 (6-7.5) | √√ | √√ | √√ |
| | | 25 (1) | 100-125 (4-5) | √√ | √√ | √√ |
| | | 19 (3/4) | 75-100 (3-4) | √√ | √√ | √√ |
| | ATPB | 37.5 (1-1/2) | 75-100 (3-4) | | | √√ |
| | | 25 (1) | 50-100 (2-4) | | | √√ |
| | | 19 (3/4) | 40-75 (1.5-3) | | | √√ |
| Intermediate | Dense, Fine | 25 (1) | 75-100 (3-4) | √√ | √√ | √√ |
| | | 19 (3/4) | 60-75 (2.5-3) | √√ | √√ | √√ |
| | Dense, Coarse | 25 (1) | 100-125 (4-5) | √√ | √√ | √√ |
| | | 19 (3/4) | 75-100 (3-4) | √√ | √√ | √√ |
| Surface | Dense, Fine | 19 (3/4) | 60-75 (2.5-3) | √√ | √√ | √ |
| | | 12.5 (1/2) | 40-60 (1.5-2.5) | √√ | √√ | √ |
| | | 9.5 (3/8) | 25-40 (1-1.5) | √√ | √√ | √ |
| | | 4.75 (1/4) | 15-20 (0.5-0.75) | √√ | √√ | √ |
| | Dense, Coarse | 19 (3/4) | 75-100 (3-4) | | | √√ |
| | | 12.5 (1/2) | 50-60 (2-2.5) | | | √√ |
| | | 9.5 (3/8) | 40-50 (1.5-2) | | | √√ |
| | SMA | 19 (3/4) | 50-60 (2-2.5) | | √ | √√ |
| | | 12.5 (1/2) | 40-50 (1.5-2) | | √ | √√ |
| | | 9.5 (3/8) | 25-40 (1-1.5) | | √ | √√ |
| | OGFC | 12.5 (1/2) | 25-40 (1-1.5) | | | √√ |
| | | 9.5 (3/8) | 20-25(0.75-1) | | | √√ |

Notes: 1. Lift thickness conversion is approximate for practical design.
2. MESAL – Millions of Equivalent Single Axle Loads
3. (√) indicates "Recommended," (√√) indicates "Strongly Recommended."

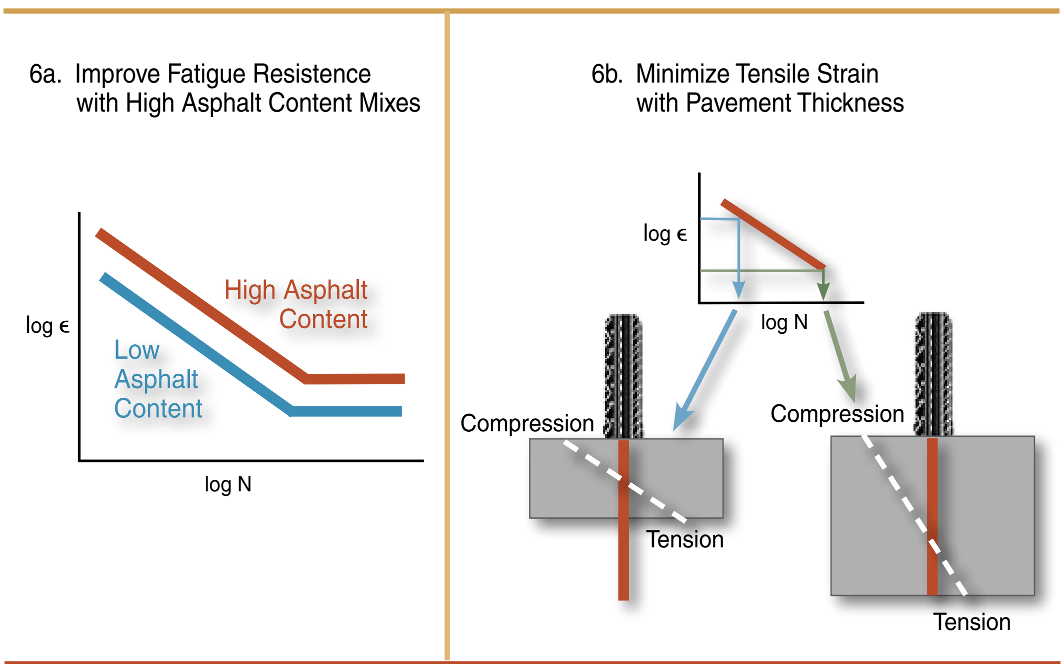
## Asphalt Base Layer

The asphalt base layer must resist the tendency to fatigue crack from bending under repeated traffic loads. Since 2001, a number of laboratory studies have been launched to characterize the fatigue endurance limit of asphalt mixtures and to discover its underlying mechanics as well as devise ways to practically implement this concept in Perpetual Pavement design.

An international workshop was held in conjunction with NCHRP Project 9-44 in order to develop a plan to validate the FEL *(AAT, 2007)*. As a part of this workshop, the Fatigue Endurance Limit was defined as: "a level of strain below which there is no cumulative damage over an infinite number of cycles." While most participants did acknowledge the long-life behavior of properly designed and constructed asphalt pavements, not all agreed that asphalt mixtures have an endurance limit. Most did agree that at low levels of strain, there is an appreciable change to the fatigue relationship resulting in less damage per cycle. It was hypothesized that this was, in part, due to healing, a lack of crack propagation, and non-linearity in fatigue relationships. The participants in this workshop concluded that to precisely define an endurance limit, there must be consideration of the effects of temperature, aging, healing, and mixture composition.

One mixture characteristic that can help guard against fatigue cracking is a higher designed asphalt content (Figure 6a) which accomplishes two important goals. It allows the material to be compacted to a higher density, and in turn, improve its durability and fatigue resistance. A summary of fatigue research studies by Epps and Monismith *(1972)* established that this behavior is consistent in many asphalt mixtures. Additional asphalt, up to a point, provides the flexibility needed to inhibit the formation and growth of fatigue cracks. Combined with an appropriate total asphalt thickness, this helps ensure against fatigue cracking from the bottom layer *(Figure 6b)*. The concept of a high asphalt content base has been employed in California *(Monismith and Long, 1999a)*, but it is important to note that it is not merely additional asphalt that improves fatigue

Figure 6. Fatigue Resistant Asphalt Base



performance, but increased density *(Crovetti et al., 2008; Crovetti, 2009)*. Many states have modified their mix design procedures by requiring compaction conditions which encourage higher asphalt content in the base layer.

Numerous laboratory studies have sought to define the FEL *(Peterson et al., 2004; Prowell and Brown, 2006)*, and some of the most extensive studies have been done by the University of Illinois *(Carpenter et al., 2003; Ghuzlan and Carpenter, 2001; Thompson and Carpenter, 2004).* Over 20 mixtures had been tested in the laboratory by these researchers and this work demonstrated the existence of the fatigue limit in all of them. In this work, Carpenter et al. *(2003)* showed that overloading for a few cycles did not destroy FEL, and that a value of 70 $\mu\epsilon$ was a lower limit for the mixtures tested. In later work, Carpenter and Shen *(2006)* found that binder type was a more important factor in establishing the FEL than binder content.

More advanced concepts in identifying the fatigue endurance limit have been introduced by Underwood and Kim *(2009)* and Bhattacharjee et al. *(2009)* by using concepts of viscoelasticity. Underwood and Kim *(2009)* used viscoelastic continuum damage modeling to incorporate the effects of healing and, ultimately, reducing the need for lengthy testing protocols. Bhattacharjee and colleagues *(2009)* used the elastic-viscoelastic correspondence principle to determine the FEL. They identified the FEL as the point at which a hysteresis loop forms between the applied stress and the pseudostrain. They found that the endurance limits identified this way were of the same order of magnitude as those from beam fatigue tests.

The asphalt content in the base should be defined as that which produces low air voids in place. This ensures a higher volume of binder in the voids in mineral aggregate (VMA), which is critical to durability and flexibility. This concept has been substantiated by Linden et al. *(1989)* in a study that related higher-than-optimum air void content to reduction in fatigue life. Fine-graded asphalt mixtures have also been noted to have improved fatigue life *(Epps and Monismith, 1972)*. If this layer is to be opened to traffic during construction, provisions should be made for rut testing the material to ensure performance during construction, at a minimum.

Another approach to ensuring the fatigue life would be to design a thickness for a stiff structure such that the tensile strain at the bottom of the asphalt layers would be minimized to the extent that cumulative damage would not occur. This would allow for a single mix design to be used in the base and intermediate layers, precluding the need to switch mix types in the lower pavement structure. This strategy is used in the TRL method proposed by Nunn and his colleagues *(1997)* as well as in the French approach *(EAPA, 2009; Corte, 2001)*. Molenaar et al. *(2009)* suggested that using a stiff base material could reduce the asphalt thickness by up to 40 percent. Their approach was to use a heavily modified asphalt binder with six to 7.5 percent SBS polymer. Xiang et al. *(2009)* evaluated mixtures with three grades of asphalt and found that the hardest of the three *(PG 82-22)* performed best as a binder in the lower pavement layers. As opposed to Molenaar and his colleagues, Harvey et al. *(2004)* found that the best way to improve fatigue life was to use a harder, unmodified asphalt at a higher asphalt content to achieve very low voids in the field.

Because this layer is the most likely to be in prolonged contact with water, moisture susceptibility needs to be considered. Kassem et al. *(2008)* examined base mixes in Perpetual Pavements in Texas for void distribution and uniformity. They found that coarse Superpave mixes could be very permeable and that could lead to moisture susceptibility problems. A higher asphalt content, which would increase the mix density, should enhance the mixture's resistance to moisture problems, but it is advisable to conduct a moisture susceptibility test during the mix design.

## Intermediate Layer

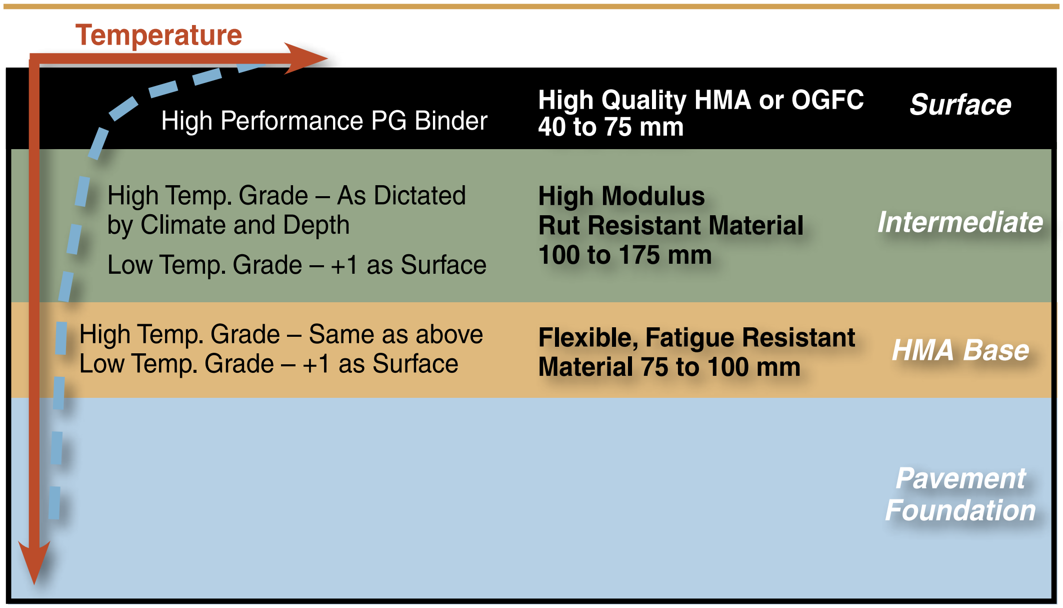
The intermediate or binder layer must combine the qualities of stability and durability. Stability in this layer can be obtained by achieving stone-on-stone contact in the coarse aggregate and using a binder with an appropriate high-temperature grading. This is especially crucial in the top four inches of the pavement where high stresses induced by wheel loads can cause rutting through shear failure.

The internal friction provided by the aggregate can be obtained by using crushed stone or gravel and ensuring an aggregate skeleton. One option would be to use a large nominal maximum size aggregate which could reduce cost due to a lower asphalt content, and guidance for the design of large-stone mixtures can be found in Kandhal *(1990)* and Mahboub and Williams *(1990)*. For mixtures with a nominal maximum aggregate size up to 37.5 mm, the Superpave mix design approach may be used (AI, 1996b). However, it should be noted that the large NMAS can lead to segregation and higher-than-desirable air voids which can lead to the intrusion of water. In such instances, it would be wise to require a lower void content in mix design, and to ensure a high level of compaction in the field. The same effect could be achieved with smaller aggregate sizes so long as stone-on-stone contact is maintained. One test for evaluating whether this type of interlock exists is the Bailey method *(Vavrik et al., 2001).*

The Performance Graded (PG) binder system is used to classify the asphalt according to high and low service temperatures *(AI, 1996a)*. The high-temperature grade of the asphalt should be the same as the surface to resist rutting. However, the low temperature requirement could probably be relaxed one grade, since the temperature gradient in the pavement is relatively steep and the low temperature in this layer would not be as severe as the surface layer (Figure 7). For instance, if a PG 70-28 is specified for the surface layer, a PG 70-22 might be used in the intermediate layer. The LTTBind Software can be used to determine the proper asphalt binder grade for each layer *(LTPP, 2010)*.

The mix design should be a standard Superpave approach *(AI, 1996b)* with a materials selection process and a design air voids level that will guard against permeability.

Figure 7. Impact of Temperature Gradient on Asphalt Grade



Performance testing should include rut testing and moisture susceptibility, at a minimum. Currently, the asphalt mixture performance tester (AMPT) is configured to provide a measure of rutting resistance known as the flow number. This repeated-load test relies on the development of tertiary flow to identify the point at which the material becomes unstable and thus susceptible to rutting. The procedure is still in the development stage as issues with repeatability of results are being addressed *(Dongre et al., 2009).*

A report on performance testing is available from the National Center for Asphalt Technology *(Brown, et al., 2001)*. They suggest the conditions of rut testing need to be selected considering the high temperature grade of the PG binder or criteria for the particular device. Another option for performance testing is the simple shear test (SST) *(Sousa et al., 1994)*, which was used in the California I-710 freeway project *(Harvey et al., 2004).*

Determination of asphalt modulus for design purposes may be done either in the laboratory or from field deflection testing. Currently the MEPDG calls for the use of the AMPT in the laboratory testing of asphalt mixtures to determine the dynamic modulus in accordance with AASHTO TP62-07. It appears at this time that this method of testing will become the standard for asphalt modulus going into the future although adjustments are being made to improve the precision of the test *(Bennert and Williams, 2009)*.

Backcalculation procedures for estimating pavement layer moduli from non-destructive deflection testing have been in use for almost three decades. Recently, Gefada et al. *(2008)* have found that a number of backcalculation methods produce general agreement in the values they determined. Scullion *(2006)* used backcalculation in determining the design modulus values for Perpetual Pavement asphalt mixtures used in Texas. In adjusting layer moduli for seasonal variations, the Washington DOT *(Pierce and Mahoney, 1996)* and the Minnesota DOT *(Ovik et al., 1999)* use modulus-temperature relationships for asphalt concrete and seasonal multiplication factors based on estimated pavement temperatures. Data available from the Long Term Pavement Performance (LTPP) database were used in the design of the Bradford Bypass in Pennsylvania *(Rosenberger, 2006)*. For structural design purposes, the asphalt mix modulus corresponding to the mean monthly pavement temperature is used.

## Wearing Surface

The wearing surface requirements would depend on traffic conditions, environment, local experience, and economics. Performance requirements include resistance to rutting and surface cracking, good friction, mitigation of splash and spray, and minimization of tire-pavement noise. These considerations could lead to the selection of stone matrix asphalt (SMA), an appropriate Superpave dense-graded mixture, or open-graded friction course. Guidance on mix type selection can be found in Newcomb and Hansen (2006) as listed in Table 4. It should be noted that small NMAS surface mixtures may benefit from the inclusion of fine RAP as a part of the sand fraction in the mix.

In some cases, the need for rutting resistance, durability, impermeability, and wear resistance would dictate the use of SMA. This might be especially true in urban areas with high truck traffic volumes. Properly designed and constructed, an SMA will provide a stone skeleton for the primary load carrying capacity and the matrix (combination of binder and filler) gives the mix additional stiffness. Methods for performing an SMA mix design are given in NCHRP Report No. 425 (Brown and Cooley, 1999).

The matrix in an SMA can be obtained by using polymer-modified asphalt, with fibers, or in conjunction with specific mineral fillers. Brown and Cooley (1999) concluded that the use of fibers is beneficial to preclude drain-down in SMA mixtures. They also point out the need to carefully control the aggregate gradation, especially on the 4.75 mm and 0.75 mm sieves. In instances where the overall traffic is not as high, or in cases where the truck traffic is lower, the use of a well designed, dense-graded Superpave mixture might be more appropriate. As with the SMA, it will be necessary to design against rutting, permeability, weathering, and wear. The Asphalt Institute *(1996b)* provides guidance on the volumetric proportioning of Superpave mixtures. It is recommended that a performance test of dense-graded mixtures, whether SMA or Superpave, be done during mixture design. At a minimum, this should consist of rut testing *(Brown et al., 2001)*, but other tests such as the flow number test from the AMPT *(Dongre et al., 2009)* or the Superpave shear tester *(Sousa et al., 1994)* could be employed to estimate the performance of the material.

Open-graded friction courses (OGFC) are designed to have voids that allow water to drain from the roadway surface. These are primarily used in western and southern regions of the United States to improve wet-weather friction, but may be found in northern states such as Massachusetts, New Jersey, and Wyoming also. Mixtures should be designed to have about 18 to 22% voids to provide good long-term performance *(Huber, 2000)*. Fibers are sometimes used to help resist draindown of the asphalt during construction. Huber *(2000)* also reports that the use of a polymer-modified asphalt will help in providing long-term performance. A mix design method for OGFC has been developed by Kandhal and Mallick *(1999)* using the Superpave Gyratory Compactor. Guidance regarding the construction and maintenance of OGFC surfaces is found in Kandhal *(2001)*.

The PG grade used in the lift of asphalt should be appropriate for the climate and traffic in a given area, consistent with Superpave practice. The high temperature grade should be selected to resist rutting, especially if heavy, slow moving traffic is present in high volumes. To resist thermal cracking, the low-temperature grade should be that normally used for 95 percent or 99 percent reliability in the area, depending upon availability and cost. Again, the LTPPBind software should be used to provide guidance on the proper grade of asphalt if local guidance is not available *(LTPP, 2010)*.

## Summary

Engineers have compiled knowledge and research to create a composite pavement structure which can be utilized to increase the chances of a flexible pavement achieving long life. This pavement structure (Figure 3) includes a rut and wear resistant upper layer of asphalt surfacing. In many cases, a stone matrix asphalt (SMA), an open-graded friction course (OGFC), or a dense Superpave design is used for this lift. Below the wearing course, engineers design a rut resistant and durable intermediate layer. Finally, the base layer of the asphalt needs to be a fatigue resistant, durable layer that is easy to compact. This final lift is designed many times with an increased asphalt content and reduced air voids *(Newcomb et al., 2000)*.

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to create a composite pavement structure
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of a flexible pavement achieving long life.

# CONSTRUCTION

Construction of a Perpetual Pavement does not differ appreciably from conventional asphalt pavements, but it does require great attention to detail and a commitment to build it with quality from the bottom up. In the process of building the roadway or airfield, modern methods of testing should be employed to give continuous feedback on the quality of materials and construction.

The foundation must be able to support paving and compaction operations during construction. Materials for this layer may include sand or sandy-gravel subgrades, stabilized fine-grained subgrade, unstabilized or stabilized granular base materials, or rubblized concrete. Thus, this layer must be well-compacted, smooth and stiff enough to support construction traffic and provide resistance to compactors. Methods for achieving uniformity have been discussed in the Materials section of this publication. It is recommended that in-situ testing of the foundation as outlined in Thomas et al. *(2004)* or in Nunn *(1997)* be used to ascertain both the quality and consistency.

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When proper structural design and mix type selection processes are employed, good construction practices can ensure good performance. Issues that can surround the construction of the asphalt layers that can be detrimental to performance include lack of density, permeability to water, lack of interface bonding, and segregation. Although some of these are related to each other, they will be discussed separately as they may have different causes.

One issue that can affect the density of the asphalt base layer is its interlayer friction with the pavement foundation. If there is insufficient friction between these two layers, compaction of the base layer will be problematic as it will tend to shove out from under the rollers. This condition can occur when there is too much dust on the foundation surface generated by construction traffic or if the foundation has been stabilized with a dust palliative and it has recently rained. The slippage of the mat from under the compactive effort will not allow the proper vertical force to be applied to consolidate the mixture. Remedial action for such a condition may include waiting for the material to dry to a lower moisture content, excavating the top few inches of the foundation to remove the dust, adding granular material to the top of the foundation, or using a thicker lift for the bottom of the base course. In an extreme case, a chip seal could be placed on the foundation to provide the friction needed to hold the asphalt mix in place during compaction.

The lack of density is detrimental to the cracking performance of the lower asphalt layers and rutting in the upper layers. As described in the Materials section, lower density equates to a lower fatigue life and a lower fatigue endurance limit. If low density results in a fatigue limit below that used in design, the pavement could crack deep within its structure. It is interesting to note that in the I-710 freeway project in California, the primary purpose of the additional asphalt in the base layer was to provide for ease of construction and compaction in this portion of the pavement. The higher density achieved in this layer helped obtain the goal of providing improved fatigue resistance *(Harvey et al., 2004)*. A low asphalt content coupled with a large nominal maximum size aggregate (one inch) led Scullion *(2006)* to express concern about the moisture susceptibility in Perpetual Pavement sections in Texas. One of the primary means of addressing the inability to compact an asphalt mixture is to make sure that the lift thickness corresponds appropriately to the nominal maximum aggregate size in the mixture as provided by Newcomb and Hansen *(2006)* in Table 4. In general, the lift thickness should be three to four times the NMAS for fine-graded mixtures and four to five times for coarse-graded mixtures *(Brown et al., 2004)*.

The lack of density in the asphalt layers may be caused by overly stiff mixes being difficult to work and compact resulting from binders that have been oxidized by overheating in the mixing process. This problem is sometimes exacerbated when polymer modified asphalt binders are used. Industry guidelines provided by APEC *(2001)* may be used to ensure the proper temperature is used in the handling and application of liquid asphalt binders. The workability of asphalt mixtures may be considerably improved with Warm Mix Asphalt technology which allows the material to be placed and compacted at temperatures anywhere from 35 to 100°F lower than conventional asphalt mixtures *(Prowell and Hurley, 2007)*.

Segregation can either be the result of a separation of fine and coarse aggregate during production, transport, and placement *(AASHTO, 1997)* or the result of temperature differentials that occur during transport and paving operations *(Willoughby et al., 2002)*. Coarse aggregate mixtures are usually the most problematic, as there is less opportunity for segregation to occur in finer graded mixtures. The danger with segregation in large aggregate, coarsely graded mixtures is that the mix may become permeable in coarse pockets which could lead to the infiltration of water and subsequent moisture damage *(Scullion, 2006)*. Segregation may be measured with infrared temperature techniques and laser texture methods such as the Rosan procedure (Stroup-Gardiner and Brown, 2000). Proper handling of the material during manufacture, transport and laydown can do much to prevent the problem. The use of material transfer devices can aid in avoiding thermal segregation by remixing the asphalt prior to placement. Additionally, steps may be taken in the selection of materials and mix design to avoid many of the problems associated with segregation. For instance, in large stone asphalt base mixtures, it is possible to design the mix at a lower void content so that it is less susceptible to being permeable. A finer total gradation will also allow less opportunity for mix segregation. As a means to insure impermeability, using a fine surface mix will seal the surface of the pavement preventing moisture infiltration from the top.

Closely related to segregation's impact on pavement performance is the issue of longitudinal joint density. Because density tends to be lower at the edges of the asphalt mat, the mix may be more permeable at this point, and more susceptible to moisture infiltration and damage. Guidance exists on the best way to construct longitudinal joints *(NAPA, 2002)*. Although often times not possible due to space limitations, the use of echelon paving or full-width paving have the effect of essentially eliminating the longitudinal joint since the two paving lanes are placed at the same time. Brown *(2006)* discusses ways to improve longitudinal joint performance by using techniques such as wedge joints, joint heaters, and joint sealants. One of the most practical ways of protecting longitudinal joints in lower pavement layers is to use a fine-graded, impermeable mixture on the pavement surface. This has the effect of sealing the joint in addition to providing a quiet, smooth surface.

The importance of bonding of asphalt layers to each other was demonstrated at the NCAT test track *(Willis and Timm, 2007).* A test section that had been designed with a rich bottom layer (i.e., asphalt content 0.5 percent above optimum) showed fatigue cracking that initiated at an interface between a base asphalt layer and the layer above it. Forensic testing and modeling showed that the pavement layers had debonded, and that the resulting slip produced higher than anticipated tensile strains in the pavement leading to the cracking. In recent years, more research has been focused on the bonding of asphalt layers within the pavement system. Mohammed and his colleagues *(2009)* are involved in the development of field tests for the bond strength of tack coats in the field. West et al. *(2005)* found that both straight grade asphalt and asphalt emulsion can be used to produce quality tack coats, but that milling enhanced the bond in the case of asphalt overlays. Thus, for asphalt pavement rehabilitation, milling should be encouraged not only to remove surface defects but also to ensure the bonding of the overlay to the existing pavement surface.

As with any asphalt construction, volumetric control of the mixtures by the contractor will be the key to consistency and quality in the final product. The contractor should have access to a fully equipped and staffed quality control laboratory. Periodic testing and data analysis with good quality control and inspection techniques will ensure that the desired characteristics will be imparted to the pavement. Nuclear or dielectric methods of testing may be used for the assessment of in-place density, thickness can be continuously monitored with ground penetrating radar and smoothness can be evaluated with new lightweight profilometers.

While construction procedures for Perpetual Pavements do not differ from normal best practices, it is important that close attention be given to all aspects of the production and placement of the material. To help ensure the longevity of the pavement structure, it is important that:

- A strong and uniform foundation is prepared.
- Optimum density in the asphalt mixtures is achieved.
- The asphalt mix design, production, and placement lead to good uniformity.
- Bonding between all pavement layers is achieved.
- Normal quality control procedures are followed throughout the construction.

# PERFORMANCE

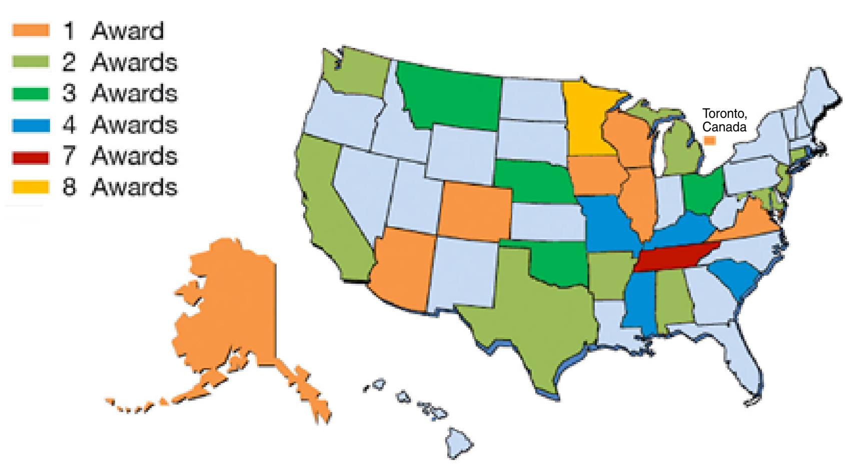
For Perpetual Pavements to be viable, they must perform from the perspectives of both engineering and economics. Designing against structural defects, proper materials selection, good construction practices, and scheduling resurfacing activities to maintain the functionality of the pavement are the primary engineering concerns for performance. Efficient design, low maintenance and rehabilitation costs, and long pavement life will ensure the economy of the pavement.

In the Perpetual Pavement concept, it is necessary to periodically monitor the pavement condition to track surface distresses and ensure they progress no further into the structure than the top few inches of the pavement. Thus, distresses such as top-down fatigue cracking, thermal cracking, rutting, and surface wear can be confined to the wearing course by timely resurfacing. There are a number of case histories that support the idea that thick, well-constructed asphalt pavements have distresses extending no deeper than their surfaces.

The Asphalt Pavement Alliance began a program in 2001 to recognize Perpetual Pavements that have been in service for 35 years or longer. As a part of the criteria for this award, no more than four inches of additional thickness could have been gained, and overlays had to have been constructed a minimum of 13 years apart. So far, over 56 pavement sections submitted by agencies throughout the U.S. have earned the Perpetual Pavement Award, and a map showing a distribution of the awards is shown in Figure 8. The pavements include interstate highways, major civilian and military airfields, and low and medium volume roads. No doubt there are many more Perpetual Pavements throughout the country.

A Dutch study *(Schmorak and Van Dommelen, 1995)* of 176 pavement sections showed that surface cracking occurred in asphalt structures thicker than 6 inches, with cracks

Figure 8. Distribution of Perpetual Pavement Awards



In the Perpetual Pavement concept, it is necessary to periodically monitor the pavement condition to track surface distresses and ensure they progress no further into the structure than the top few inches of the pavement.

extending about 4 inches down into the asphalt layer. They concluded that conventional fatigue failure was very improbable and that surface cracking would be the main form of distress in thick asphalt pavements. A 1997 report from the U.K. *(Lesch and Nunn)* showed that pavement deterioration in thick asphalt structures was much more likely to occur in the wearing course than deep in the pavement. This paper also demonstrated that the structural layers become stronger with time, instead of weakening as is commonly assumed. Ferne and Nunn *(2004)* and Merrill et al. *(2006)* confirmed these observations with a review of information contained in European-wide studies of long-lived asphalt pavements on high-traffic routes.

In a case study representative of good performing pavements, a review of thick (between six and 19-inch) asphalt pavements on I-90 through the state of Washington revealed that none of these sections had ever been rebuilt for structural reasons *(Baker and Mahoney, 2000)*. The pavement ages ranged from 23 to 35 years, and thick asphalt pavements on this route comprise 40 percent of the length (about 140 out of 362 miles). West of the Cascade Mountains, near Seattle, the average age at resurfacing was 18.5 years. On the eastern side of the state, the average age at first resurfacing was 12.4 years and the time until second resurfacing was 12.2 years. Mahoney and his co-authors *(2007)* followed up on this study and included pavements from Oregon and California. In this work, interstate highway concrete pavements were also included. It was noted in the conclusions that there was essentially no difference in the age of flexible and rigid interstate pavements for Washington and Oregon. Data contained within the paper on pavement smoothness in Washington and Oregon clearly shows that the asphalt pavements are considerably smoother than the concrete pavements. California did not report pavement ages or pavement smoothness data.

The previously discussed New Jersey DOT investigation of a 26-year old pavement surface on I-287 is another example of a long-life asphalt pavement *(Fee, 2001)*. The 10-inch thick asphalt pavement had received a minimum of maintenance, and just the surface showed fatigue cracking, longitudinal cracking in the wheelpaths, and ruts deeper than one inch. None of the distresses extended more than three inches into the depth of the asphalt. As a result, the decision was made to mill off the top three inches and replace it with a total of four inches of asphalt surfacing. This is similar to the performance noted on Route 82 in Connecticut where a 2007 Perpetual Pavement Award winning section that was 10 inches of asphalt over 10 inches of granular materials was noted to have gone 24 years before resurfacing to correct top-down cracking *(Yut et al., 2009)*.

Monismith et al. *(2009)* presented a review of the construction of the I-710 Perpetual Pavement that was built in 2003 and presented monitoring data after five years of service. As discussed earlier, the design traffic amounted to 200,000,000 ESAL, the full-depth asphalt section was 12 inches thick, and the asphalt over cracked and seated concrete pavement was 8 inches thick. The deflection data from this pavement showed that the pavement sections were performing as expected. In a similar type of study in

Texas, Scullion *(2006)* found that the Perpetual Pavements constructed there had very low deflections indicating an overall pavement stiffness approaching that of concrete pavement.

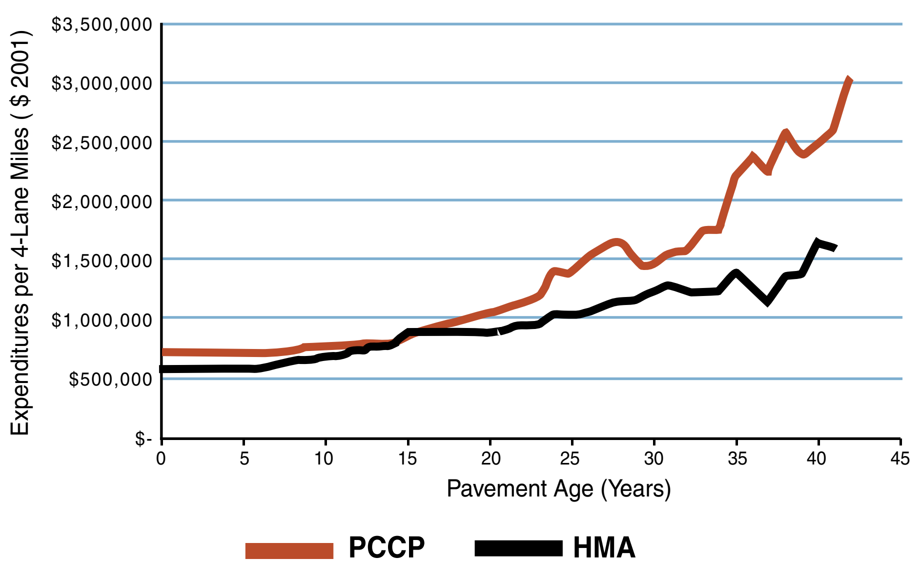
Romanoschi et al. *(2006, 2008, 2009)* have documented the testing and behavior of four flexible pavement sections on U.S. 75 in Kansas near Sabetha. These sections were designed as conventional asphalt pavements as well as a Perpetual Pavement and instrumented with strain gauges to monitor pavement reactions. They found that under an 18,000-lb axle load, even in hottest testing time in July, that the measured strains were very low (less than 70 $\mu\epsilon$). Their research suggests that the Perpetual Pavement with the rich base layer may have superior fatigue performance. In an actual pavement section on a street in the City of Eugene, Oregon, it was found in 2008 that a 10-inch section of asphalt pavement had lasted 55 years and was still in good condition, needing only surface repairs *(Huddleston, 2008)*.

The economics of thick asphalt pavements in high traffic situations have been well-documented. Cross and Parsons *(2002)* compared several interstate asphalt sections in Kansas to concrete sections. Their conclusion, as shown in Figure 9, was that on average, over a 40-year period, asphalt pavements were more economical than concrete pavements for the Kansas interstate system. This was primarily due to the extensive rehabilitation and reconstruction activities that were needed on some of the concrete sections later in their life. Gibboney *(1995)* noted this same trend for four interstate highways he studied in Ohio. Studies for Pakistan *(Kamal et al., 2006)* and Afghanistan *(Lande et al., 2006)* have shown that Perpetual Pavements were clearly more economical than comparable rigid pavements.

Perpetual Pavements are also more economical over the long term than conventionally designed asphalt pavements. El-Hakim et al. *(2009)* compared the design and performance of a conventional asphalt pavement and a Perpetual Pavement section in Ontario. The Perpetual Pavement cross-section was comprised of 13.5 inches of asphalt

Figure 9. Comparison of Average Kansas Interstate Costs

(Cross and Parsons, 2002)



mix over 4.5 inches of an asphalt stabilized subgrade, and the conventional pavement was 11 inches of asphalt over 13 inches of granular material. The Perpetual Pavement had better predicted performance according to the MEPDG and was 6.6 percent cheaper in life cycle cost over a period of 50 years. Muench et al. *(2004)* found that the thicker (perpetual) pavement on low-volume roads in Washington saved over $160,000 per 2-lane miles over a 50 year period. Cheneviere and Ramdas (2006) confirm the economic benefits of Perpetual Pavements over conventional pavements in the United Kingdom where the life cycle cost gives a clear indication of its sustainability.

The performance of Perpetual Pavements has been confirmed in a number of studies. The Asphalt Pavement Alliance's Perpetual Pavement Award program has shown many examples of long lasting asphalt pavements in applications ranging from major airports to low volume roads. Reviews of the performance of existing pavements in a number of states have shown the ability of well-designed and well-constructed asphalt pavements to serve under a variety of traffic conditions for the long term. Perpetual Pavement test sites have been monitored and tested, and to date, are performing as well or better than expected. The economics of long-life asphalt pavements have proven to be advantageous when compared to concrete pavements or conventionally designed pavements.

*The Minnesota Department of Transportation won a 2009 Perpetual Pavement Award for the eastbound lanes of US TH 10, east of Detroit Lakes to east of Perham.*

# SUMMARY

Perpetual asphalt pavements have been designed and constructed for decades as full-depth and deep-strength asphalt pavements. Recently, a number of design procedures have been developed to recognize the conditions under which asphalt pavements are not subject to damage and to allow for the efficient design of the pavement sections. Materials selection plays a key role in the design and construction of Perpetual Pavements, and the materials must be selected according to the role they play in enhancing pavement performance. Construction practices are of paramount importance to the performance of asphalt pavements. The performance of Perpetual Pavements has been documented in a number of studies both from engineering and economic points of view.

Going forward, the mechanistic-empirical design process will be the format for the design of Perpetual Pavements. Transfer functions describing pavement performance need to allow for limiting strains so that conditions resulting in no pavement damage can be accounted for. Existing design procedures for Perpetual Pavements encompass a variety of applications including high-volume and low-volume pavements, high-modulus pavements, and the rehabilitation of flexible and rigid pavements.

*The Mississippi Department of Transportation won a 2009 Perpetual Pavement Award for MS 613, Jackson County.*

The knowledge and research exist to create a pavement structure which can ensure the long life of a flexible pavement. The materials used for the various layers of the pavement structure must be selected with respect to the functions they must serve. This includes a rut and wear resistant upper layer of asphalt surfacing. In many cases, a stone matrix asphalt, an open-graded friction course, or a dense Superpave design may be used as the surface. In the case of dense-graded or SMA mixtures, the materials should be selected to keep the surface impermeable. Below the wearing course, a rut resistant and durable intermediate layer should be constructed from a dense-graded mix. Finally, the base layer of the asphalt pavement needs to be a fatigue resistant, durable layer that is easy to compact. This base lift is designed many times at an increased asphalt content and reduced air voids in order to increase density and improve fatigue resistance.

Construction procedures for Perpetual Pavements do not differ from normal best practices, but it is important that attention be given to all aspects of the production and placement of the material. The foundation layer must be strong and uniform to provide a sturdy working platform and to support traffic loads. Density and uniformity of asphalt mixtures are critical to the long-term health of the pavement, and this can be achieved through proper design of lift thicknesses, proper material selection and mix design, and appropriate construction practices. Bonding between pavement layers has been shown to be essential to the long-term performance of the pavement structure. Normal quality control procedures should be followed throughout the construction.

The long-term performance of well-designed and well-constructed asphalt pavements has been shown in a number of studies. The Asphalt Pavement Alliance's Perpetual Pavement Award program has nearly 70 examples of long lasting asphalt pavements ranging from major airports to low volume roads. The performance of existing pavements in many states has shown the ability of asphalt pavements to serve under a variety of traffic conditions for the long term. Research continues at Perpetual Pavement test sites and facilities throughout the world. The data from these have shown that Perpetual Pavements are performing as well or better than expected. Long-life asphalt pavements have been shown to have lower life cycle costs than concrete pavements or conventionally designed pavements.

The knowledge and research exist to create a pavement structure which can ensure the long life of a flexible pavement.

# RECOMMENDATIONS

Although Perpetual Pavements are becoming more accepted as their performance comes to light and design procedures are developed, there are areas of fruitful investigation which may lead to further understanding and refinements. It is recommended that the following topics be pursued through further research:

- Development of high-modulus asphalt pavements. Such pavements would reduce the overall section of material to be used, allowing for fewer vertical grade changes, reducing the raw materials to be consumed, and improving the sustainability of the pavement.
- Develop mix designs for high-modulus asphalt mixes, including the selection of binder, optimum asphalt content for low voids, and required stiffness.
- Refine the fatigue endurance limit. Although this is the subject of NCHRP Study 9-44A, it is important to identify factors that influence the fatigue limit in order to further improve Perpetual Pavement design.
- Develop an understanding of pavement layer bonding. From both a construction and a performance standpoint, it is crucial to understand how bonding occurs between pavement layers and its role in pavement responses to loads.
- Develop a unified approach to the Perpetual Pavement design. Currently, design criteria for high-volume Perpetual Pavements leads to over-designed low-volume pavements, and so a separate design procedure exists for low-volume roads. A statistically based procedure developed according to the distribution of pavement responses may be one approach.

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» Asphalt Pavement Alliance
5100 Forbes Boulevard
2nd Floor
Lanham, MD 20706
877.272.0077 Voice
301.731.4621 Fax
www.asphaltalliance.com
www.asphaltroads.org

» Asphalt Pavement Alliance Co-Chairs:

Gaylen Ghylin
Commercial Asphalt Inc., Minnesota

Mike O'Leary
MeadWestvaco, Tennessee

Brian Wood
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