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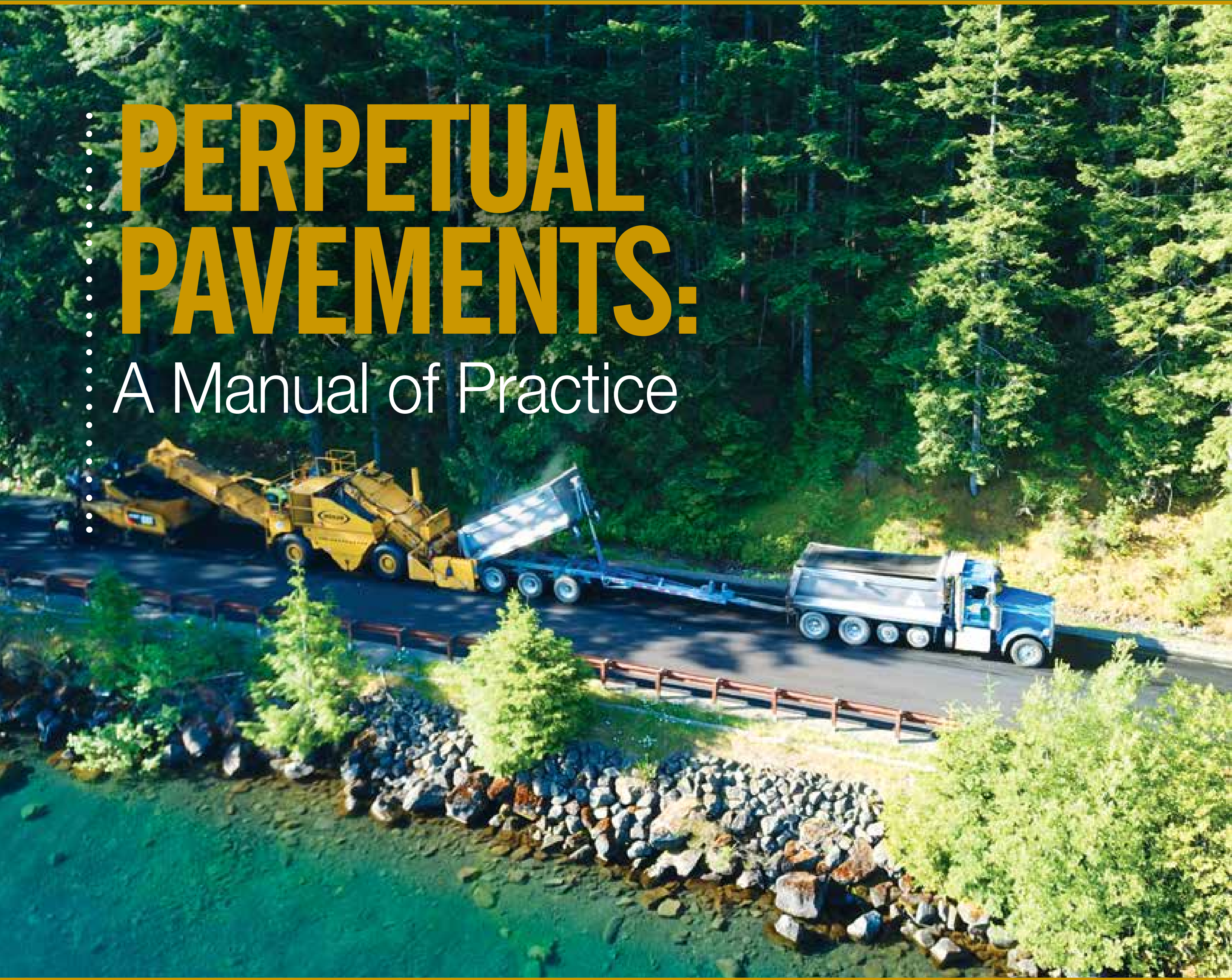

PERPETUAL PAVEMENTS:

A Manual of Practice

- **David E. Newcomb, Ph.D., P.E.** Texas A&M Transportation Institute
- **David H. Timm, Ph.D., P.E.** Auburn University
- **J. Richard Willis, Ph.D.** National Asphalt Pavement Association

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6406 Ivy Lane, Suite 350, Greenbelt, MD 20770-1441
Tel: 301-731-4748 | Fax: 301-731-4621 | Toll free 1-888-468-6499
www.AsphaltPavement.org

Audrey Copeland, Ph.D., P.E., *President & CEO*
Monica Dutcher, *Editorial Director*
PJB Marketing, *Design / Layout*

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1 INTRODUCTION

Pavement engineers have been producing long-lasting asphalt pavements since the 1930s. Research has shown that well-constructed and adequately designed flexible pavements can perform well for extended periods of time (Mahoney, 2001). Many of these pavements in the were the products of full-depth or deep strength asphalt pavement designs, and both design philosophies have been shown to provide adequate strength over extended life cycles (Newcomb et al, 2010).

Full-depth pavements are constructed by placing asphalt mixtures on modified or unmodified soil or subgrade material. Deep-strength pavements consist of layers of asphalt pavement on top of a thin granular base. Both design scenarios allow pavement engineers to design thinner pavements than if a thick granular base were used. By reducing the potential for fatigue cracking and containing cracking to the upper removable/replaceable layers, many of these pavements have far exceeded their design life of 20 years, with minimal rehabilitation (Newcomb et al., 2010).

Some pavements 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 result of designing and constructing pavements to resist distresses that impact a pavement's structural capacity. In recent years, pavement engineers have begun to introduce the methodology for designing pavements to resist the two main pavement distresses seen on roadways — fatigue cracking and rutting. This change in thinking has fostered the idea of Perpetual Pavements or long-lasting pavements.

The Asphalt Pavement Alliance (APA) has 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" (Newcomb et al, 2010). Most pavement engineers in the U.S. approach the idea of Perpetual Pavements with a 50-year structural design life in mind. While the structural integrity of the pavement should remain intact during the entirety of the pavement's life, periodic resurfacing often needs to occur after 20 years to improve friction, reduce noise, and mitigate surface cracking (Newcomb et al., 2001). While it is crucial to realize the importance of proper design for a long-lasting pavement, one must also understand that design life is a function of the design requirements, material characteristics, layer thicknesses, maintenance activities, and the failure criterion. In many cases, engineers define pavement failure as either 10% fatigue cracking in the wheel path or 0.5 inches of rutting (Von Quintus, 2001).

Though the APA defined a Perpetual Pavement through its design life, 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, 2001; Walubita et al., 2008).

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

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

While one might think pavements designed to last longer would incur more costs than pavement with shorter life-cycles, research has shown that Perpetual Pavements have the following benefits (Timm & Newcomb, 2006):

1. Perpetual Pavements eliminate reconstruction costs at the end of a pavement's structural capacity.
2. Perpetual Pavements lower rehabilitation-induced user-delay costs.
3. Perpetual Pavements reduce excessive use of non- renewable resources, such as aggregates and asphalt binder.
4. Perpetual Pavements diminish energy costs while the pavement is in service.
5. Perpetual Pavements reduce the life-cycle costs of the pavement network.

OBJECTIVES

This document aims to:

1. Provide guidance on material selection and mixture design to optimize Perpetual Pavement performance.
2. Explore current methodologies that can be used to design Perpetual Pavements.
3. Present best practices for constructing high quality, high performance pavements.

2 MATERIALS

Unlike strictly empirical pavement design procedures, mechanistic-empirical design incorporates the properties of the pavement layer materials directly as inputs. This requires methods to determine these properties and the means to understand how they fluctuate with environmental conditions. Most of the damage occurs when the pavement structure is weakest and/or the loads are the highest, and it is the goal of pavement design 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 compactive effort so that the asphalt layers are well densified. Throughout the performance period, the foundation provides support to the traffic loads and reduces the variability of seasonal pavement responses 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 soft clays and freeze-thaw cycles in frost-susceptible soils.

A pavement foundation may be comprised of compacted subgrade, chemically stabilized subgrade, and/or stabilized or unstabilized granular material. 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 et al. (1979) provided 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; guidance on subsurface drainage may be found in the FHWA *Highway Subgrade Design Manual* (Moulton, 1980).

The Illinois Department of Transportation (IDOT) published guidance for determining an appropriate subgrade in its *Subgrade Stability Manual* (IDOT, 1982). For constructability, IDOT 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 2-1 shows that in Illinois remedial action is required if the soil CBR is less than 6, optional between a CBR of 6 and 8, and 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 lime modification of the fine-grained subgrade soils that predominate in Illinois (IDOT, 2002). Undercutting and backfilling with granular material is also a commonly used procedure, along with occasional application of geofabrics. The required thickness above the subgrade is typically 300 mm. For subgrade strengths less than a CBR of 4, the thickness is increased as per Figure 2-1.

Seasonal modulus adjustment factors are used in Washington state 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 2-1 (Pierce & Mahoney, 1996). The seasons in Washington state 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 2-1 reflect backcalculated modulus values under pavements with asphalt thicknesses

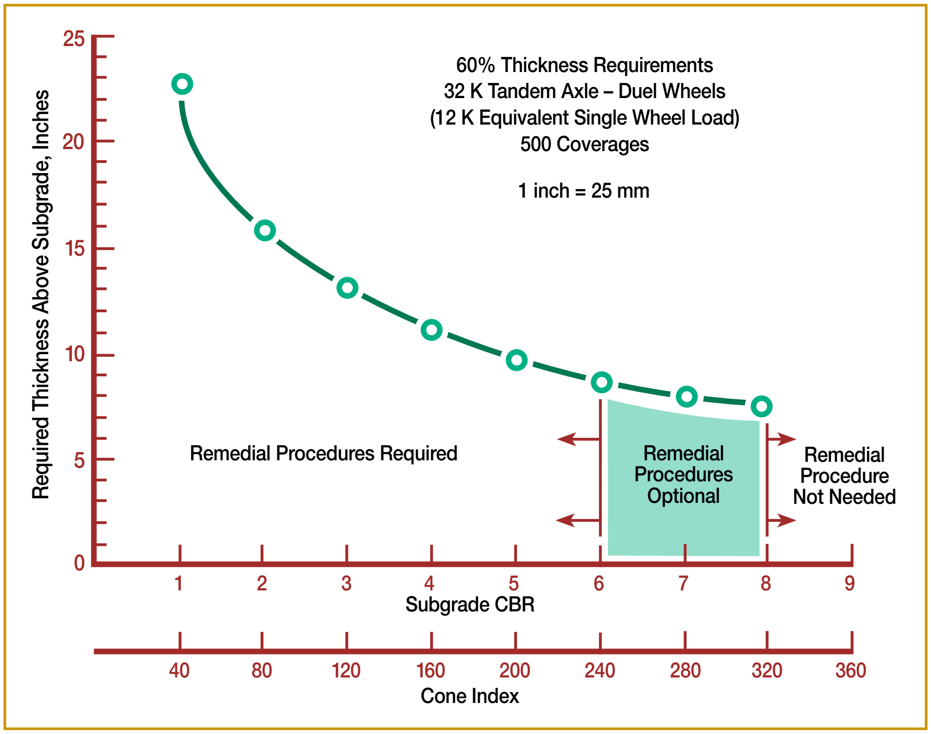

Figure 2-1. Illinois Granular Thickness Requirement for Foundation (IDOT, 1982)

season is in autumn. Because the progression of thawing results in different behavior in the upper and lower layers of the pavement base and subgrade, 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 (Mn/ROAD). 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.

ranging from thin to thick. A slightly different approach is taken in Minnesota, where the seasons are of unequal lengths as shown in Table 2-2, and the reference

In other words, it may be necessary to consider the worst condition in order to preclude undue damage during a given season.

Table 2-1. Seasonal Adjustments Factors for Unbound Materials Used in Washington state (Pierce & Mahoney, 1996).

LOCATION	MATERIAL	SEASON			
		Spring	Summer	Fall	Winter
Eastern WA	Base	0.65	1.00	0.90	1.10
	Subgrade	0.90	1.00	0.90	1.10
Western WA	Base	0.85	1.00	0.90	0.75
	Subgrade	0.85	1.00	0.90	0.85

Table 2-2. Seasonal Adjustment Factors for Mn/ROAD (After Ovik et al., 1999)

Month	Late November – February	March	April, May	June, July, August	September – early November
HMA (129/150 pen)	2.5	2.1	1.3	0.37	1.0
Granular Base	28	0.65	0.80	1.00	1.00
Subgrade	22	2.4	0.75	0.75	1.00

In the AASHTO Pavement ME Design, the Enhanced Integrated Climate Model (EICM) is used to predict the environmental conditions of temperature and moisture that affect pavement responses (AASHTO, 2008). Using past weather station data, the air temperature, wind velocity, percent sunshine, relative humidity, and precipitation at hourly intervals, unbound soil properties are predicted for the design life of the pavement. The information is used with the characterization of the unbound and asphalt mixture material properties to predict the modulus of the pavement layers in order to assess the response to traffic loads under these conditions. This information is used, along with the input of layer thicknesses and pavement performance models, to assess the extent of pavement distresses during the pavement's life.

Nunn et al. (1997) used field testing for pavement foundation materials, and several devices for accomplishing this are reviewed by Thomas et al. (2004). The British (Nunn et al., 1997) formulated an end-result specification based on nuclear density tests and surface stiffness measured by a portable dynamic plate-bearing test. The foundation design practice in the U.K. is shown in Table 2-3. 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 6-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. Transport Research Lab (TRL) set end-result requirements for the pavement foundation, both during and after its construction. Under a falling-weight deflectometer (FWD) load of 40 kN, a stiffness of 40 MPa was required on top of the subgrade and 65 MPa was required at the top of the subbase.

Table 2-3. 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 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 a specific function. For instance, the lowest layer must provide excellent durability and 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 provide rutting resistance. In an effort to provide guidance on the best application for various types of mixtures according to traffic level and the lift thickness, Newcomb & Hansen (2006) provided the information in Table 2-4.

The design of long-lasting asphalt mixtures requires attention to the selection of the component materials. The asphalt binder, aggregate, recycled materials, and any additives need to be combined to optimize the properties and behavior of the materials to fulfill their purposes in the pavement structure. The approach needs to rely on performance testing as well as volumetric balance to prevent premature aging as well as ensure resistance to cracking and rutting. This approach to asphalt mixture composition is known as a balanced mixture design (Zhou et al., 2007).

Asphalt binders must be selected based on the high and low temperatures in a given region as well as the expected traffic level (AI, 1996a). The asphalt industry has begun using modifiers and additives in binders in order to meet the needs of their customers. The most common of these are polymer modifiers, which are primarily employed to improve the high-temperature stiffness of asphalt binders (Stroup-Gardiner & Newcomb, 1995) to help rutting resistance. Sometimes, poly-phosphoric acid (PPA) is used as an inexpensive means to achieve the same end. While PPA has been used for several years, recently some practitioners and researchers have expressed concern that an excess amount may lead to premature cracking if too much is incorporated into the binder (Tam, 2012). Re-refined engine oil bottoms (REOB), also referred to as vacuum tower asphalt extenders (VTAB), have been used for improving the low-temperature behavior of asphalt binder. Again, there are years of experience in using REOB, but recent research suggests that the concentration of REOB in binders needs to be limited in order to avoid cracking problems (You et al., 2018; Karki & Zhou, 2018; Li et al., 2017). Modifiers have also been used for warm-mix asphalt (WMA) applications as well. These modifiers include long-chain waxes, water-bearing minerals, and other technologies that could reduce the production and placement temperatures of asphalt mixtures (McCarthy, 2018).

Aggregates used in asphalt mixtures in a Perpetual Pavement must have the qualities that match their function in the pavement structure. They all need to meet the quality standards required by the asphalt mixture's function in the pavement. All the aggregates need to be resistant to the effects of freeze–thaw and should have low asphalt absorption. Those in the surface layer should be non-polishing aggregates with an angular shape and rough surface texture to provide skid resistance.

Simply increasing pavement thickness does not guarantee that a pavement will have a long service life. Washington state's study of long-lasting pavements showed in many cases that pavements with shorter life-cycles in the state 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 HMA layer, the magnitude by which this reduction occurs is mix dependent (Romanoschi et al., 2008).

Table 2-4. Mix Type Selection Guide for Perpetual Pavements (Newcomb & 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 (1½)	110-150 (4.5-6)	✓✓	✓✓	✓✓
		25 (1)	75-100 (3-4)	✓✓	✓✓	✓✓
		19 (¾)	60-75 (2.5-3)	✓✓	✓✓	✓✓
	Dense, Course	37.5 (1½)	150-190 (6-7.5)	✓✓	✓✓	✓✓
		25 (1)	100-125 (4-5)	✓✓	✓✓	✓✓
		19 (¾)	75-100 (3-4)	✓✓	✓✓	✓✓
	Asphalt Treated Permeable Base	37.5 (1½)	75-100 (3-4)			✓✓
		25 (1)	50-100 (2-4)			✓✓
		19 (¾)	40-75 (1.5-3)			✓✓
Intermediate	Dense, Fine	25 (1)	75-100 (3-4)	✓✓	✓✓	✓✓
		19 (¾)	60-75 (2.5-3)	✓✓	✓✓	✓✓
	Dense, Course	25 (1)	100-125 (4-5)	✓✓	✓✓	✓✓
		19 (¾)	75-100 (3-4)	✓✓	✓✓	✓✓
Surface	Dense, Fine	19 (¾)	60-75 (2.5-3)	✓✓	✓✓	✓
		12.5 (½)	40-60 (1.5-2.5)	✓✓	✓✓	✓
		9.5 (⅜)	25-40 (1-1.5)	✓✓	✓✓	✓
		4.75 (¼)	15-20 (0.5-0.75)	✓✓	✓✓	✓
	Dense, Course	19 (¾)	75-100 (3-4)			✓✓
		12.5 (½)	50-60 (2-2.5)			✓✓
		9.5 (⅜)	40-50 (1.5-2)			✓✓
	Stone Matrix Asphalt	19 (¾)	50-60 (2-2.5)		✓	✓✓
		12.5 (½)	40-50 (1.5-2)		✓	✓✓
		9.5 (⅜)	25-40 (1-1.5)		✓	✓✓
	Open-graded Friction Course	12.5 (½)	25-40 (1-1.5)			✓✓
		9.5 (⅜)	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 "Adequate", (✓✓) indicates "Recommended".

ASPHALT BASE LAYER

The asphalt base layer must resist the tendency to fatigue crack from bending under repeated traffic loads. Since 2001, several 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 09-44 in order to develop a plan to validate the Fatigue Endurance Limit (FEL) (Bonaquist, 2009). As a part of this workshop, the FEL 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, the effects of temperature, aging, healing, and mixture composition must be considered.

One mixture design strategy that can help guard against fatigue cracking is designing a mixture with a higher asphalt content (Figure 2-2) that which accomplishes two important goals. An increased asphalt content 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 & 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 2-2). The concept of a high asphalt content base has been employed in California (Monismith & Long, 1999), but it is important to note that it is not merely additional asphalt binder that improves fatigue performance, but increased density (Crovetti et al., 2008; Crovetti, 2009). Many states have modified their mix design procedures by requiring compaction conditions that encourage higher asphalt content in the base layer.

Numerous laboratory studies have sought to define the FEL (Peterson et al., 2004; Prowell & Brown, 2006), and some of the most extensive investigations have been done by the University of Illinois (Carpenter et al., 2003; Ghuzlan & Carpenter, 2001; Thompson & Carpenter, 2004). More than 20 mixtures were tested in the laboratory, and this work demonstrated the existence of the fatigue limit in all the tested mixtures. In this work, Carpenter et al. (2003) showed that overloading for a few cycles did not destroy FEL, and that a value of –70 $\mu\varepsilon$ was a lower limit for the mixtures tested. In later work, Carpenter & Shen (2006) found that binder grade was a more important factor in establishing the FEL than binder content. The impact of binder grade and content will be discussed later.

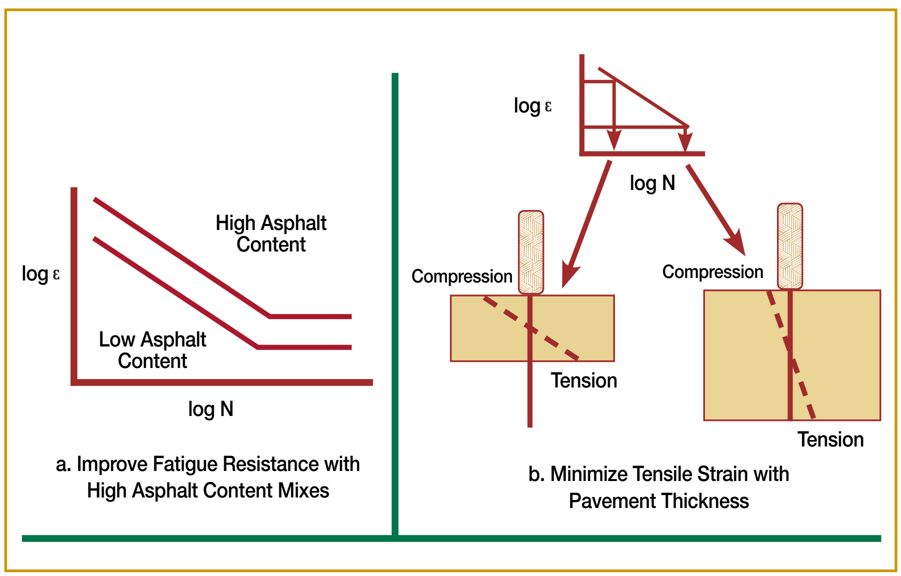

Figure 2-2. Fatigue Resistant Asphalt Base

More advanced concepts in identifying the fatigue endurance limit have been introduced by Underwood & Kim (2009) and Bhattacharjee et al. (2009) by using concepts of viscoelasticity. Underwood & Kim (2009) used viscoelastic continuum damage modeling to incorporate the effects of healing and, ultimately, reducing the need for lengthy testing protocols. Bhattacharjee et al. (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 a reduction in fatigue life. Fine-graded asphalt mixtures have also been noted to have improved fatigue life (Epps & 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 et al. (1997), as well as in the French approach for high modulus mixture design (EAPA, 2009; Corté, 2001; Levia-Villacorta et al. 2017). Molenaar et al. (2009) suggested that using a stiff base material could reduce the asphalt thickness by up to 40%. Their approach was to use a heavily modified asphalt binder with 6 to 7.5% SBS polymer. As opposed to Molenaar et al. (2009), 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 too. 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, which 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 4 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 aggregate size (NMAS), which could reduce cost due to a lower asphalt content; guidance for the design of large-stone mixtures can be found in Kandhal (1990) and Mahboub & Williams (1990). For mixtures with an NMAS 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 (Vavrick et al., 2002).

The Performance Graded (PG) binder system is used to classify the asphalt binder 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, as 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 2-3). 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 FHWA InfoPave LTPPBind software tool can be used to determine the proper asphalt binder grade for each layer.

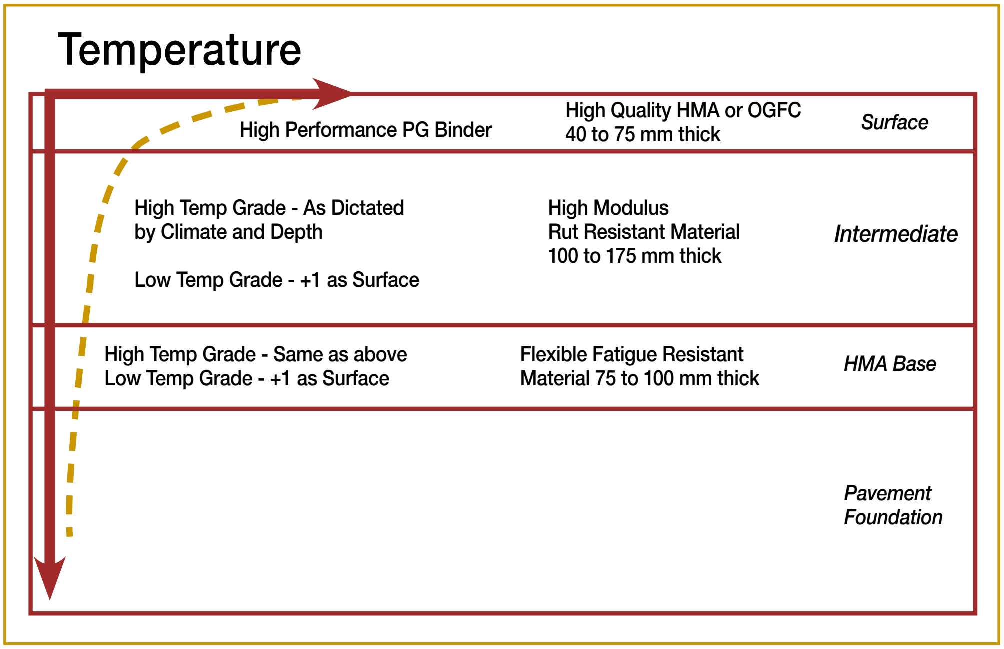

Figure 2-3. Impact of Temperature Gradient on Asphalt Grade

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 segregation and permeability. Performance testing should include rut testing, crack 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.

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).

Cracking tests for assessing the brittleness or ductility of a mixture come in several forms. Repeated loading tests are the most time consuming to perform and generally have the highest variability (Zhou et al., 2017). They are best suited for research and mixture design testing. Tests performed at cold temperatures, whether they are repeated load or monotonic, are best performed as mixture design inputs. Monotonic tests conducted at intermediate temperatures are usually best for quality control/quality assurance testing due to their simplicity. Zhou et al. (2017) describe several different cracking tests and highlight their advantages and disadvantages.

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 T 342. 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 & Williams, 2009).

Backcalculation procedures for estimating pavement layer moduli from non-destructive deflection testing have been in use for more than three decades. Gedafa et al. (2010) 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 state DOT (Pierce & 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 et al., 2006). For structural design purposes, the asphalt modulus corresponding to the mean monthly pavement temperature is used.

WEARING SURFACE

The wearing surface requirements 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 & Hansen (2006) as listed in Table 2-4. 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; 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 & Cooley Jr., 1999).

The matrix in an SMA can be obtained by using polymer-modified asphalt, with fibers, or in conjunction with specific mineral fillers. Brown & Cooley Jr. (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 No. 200 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 (AI, 1996b) provides guidance on the volumetric proportioning of Superpave mixtures. It is recommended that performance testing 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 (Dongré 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. Mixtures should be designed to have about 18–22% 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 & Mallick (1999) using the Superpave Gyratory Compactor. Guidance regarding the construction and maintenance of OGFC surfaces is found in Kandhal (2001).

SUMMARY

Engineers have compiled knowledge and research to create a composite pavement structure that can be utilized to increase the chances of a flexible pavement achieving long life. This pavement structure (Figure 2-3) includes a rut- and wear-resistant impermeable upper layer of asphalt. 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 should design a rut-resistant and durable intermediate layer. Finally, the base layer of the HMA needs to be a fatigue-resistant, durable layer that is easy to compact. This lift is designed many times at an increased asphalt content and reduced air voids (Newcomb et al., 2001).

3 PERPETUAL PAVEMENT STRUCTURAL DESIGN

OVERVIEW OF STRUCTURAL DESIGN

Asphalt pavement thickness design procedures have evolved from experience-based design in the 1950s through the development of empirical procedures in the 1960s to more modern mechanistic-empirical (M-E) procedures in the 1990s. The current AASHTO design standard is the Mechanistic-Empirical Design Guide (MEPDG) and accompanying design software, AASHTOWare Pavement ME Design.

Each design methodology aims to consider the prevailing design conditions to make a performance prediction. The design conditions include expected traffic, environmental conditions, available materials and costs. The performance predictions usually focus on bottom-up fatigue cracking of the asphalt concrete (AC) and rutting of the pavement layers. The means of making the performance prediction depends on the type of design approach. Experience-based design relies on local knowledge to make a performance prediction from experience in similar conditions. Empirical design makes the performance prediction based on rigorous testing and statistical equations linking performance to the design factors. M-E design relies on mechanistic models that compute pavement responses to external loadings which are used in empirical transfer functions to predict expected performance.

Regardless of method, the quality of any design approach is in its ability to accurately predict performance from the known design conditions. The early experienced-based design approaches were limited by lack of experience in higher volume (e.g., interstate pavements) scenarios. Therefore, experience-based methods transitioned to empirical approaches through road tests that established statistical relationships between the design conditions and performance. While initially effective, used widely across the U.S. for the past 50+ years, and still used in many states today, they have become less accurate as design conditions have drastically changed since the original testing done to establish the empirical design equations. Extrapolation has become the norm, which has the potential for inefficient structural cross-sections. Given this, M-E design has become the modern standard primarily because it can, through mechanistic modeling, more readily adapt to changing design conditions. However, M-E design still has an empirical component that requires data from calibration studies to improve its predictive accuracy.

While predicting pavement performance over time has been the goal of prevailing design methods for many years, a subset of M-E design has been aimed at eliminating traditional structural distresses (i.e., bottom-up fatigue cracking and deep structural rutting) by considering the endurance limits of the materials in the pavement cross-section. This framework, termed Perpetual Pavement design or long-life pavement design, recognizes that all materials have an endurance limit below which the material will not experience cumulative damage. For example, designing the AC thickness such that the tensile stress or strain is below the endurance limit will prevent it from experiencing bottom-up cracking. The goal of Perpetual Pavement design is therefore to predict *if* cracking will occur rather than *when* it will occur. Designing such that bottom-up cracking or deep structural rutting will not occur creates a Perpetual Pavement foundation that only requires periodic surface treatments, such as mill-and-inlay, to restore ride quality as it ages and perhaps cracks from the top down.

To fully understand Perpetual Pavement design, it's important to first begin with M-E design principles. The following sections detail M-E design, followed by Perpetual Pavement design, and finally discuss the various design tools available.

MECHANISTIC-EMPIRICAL DESIGN

As documented previously (Timm et al., 2014), researchers and practitioners have long recognized the limitations of empirical design, but M-E design did not become a viable option for pavement design on a routine basis until personal computers became commonly available in the 1990s. The main advantage of M-E design is its ability to adapt to changing conditions. When new materials, loads, or other conditions are introduced, they may simply be simulated to determine the effect on pavement thickness with more certainty than through empirical design.

As shown in Figure 3-1, M-E design has four major components that will be discussed in the following subsections:

- **Mechanistic modeling**
- **Empirical performance prediction**
- **Damage accumulation**
- **Design assessment**

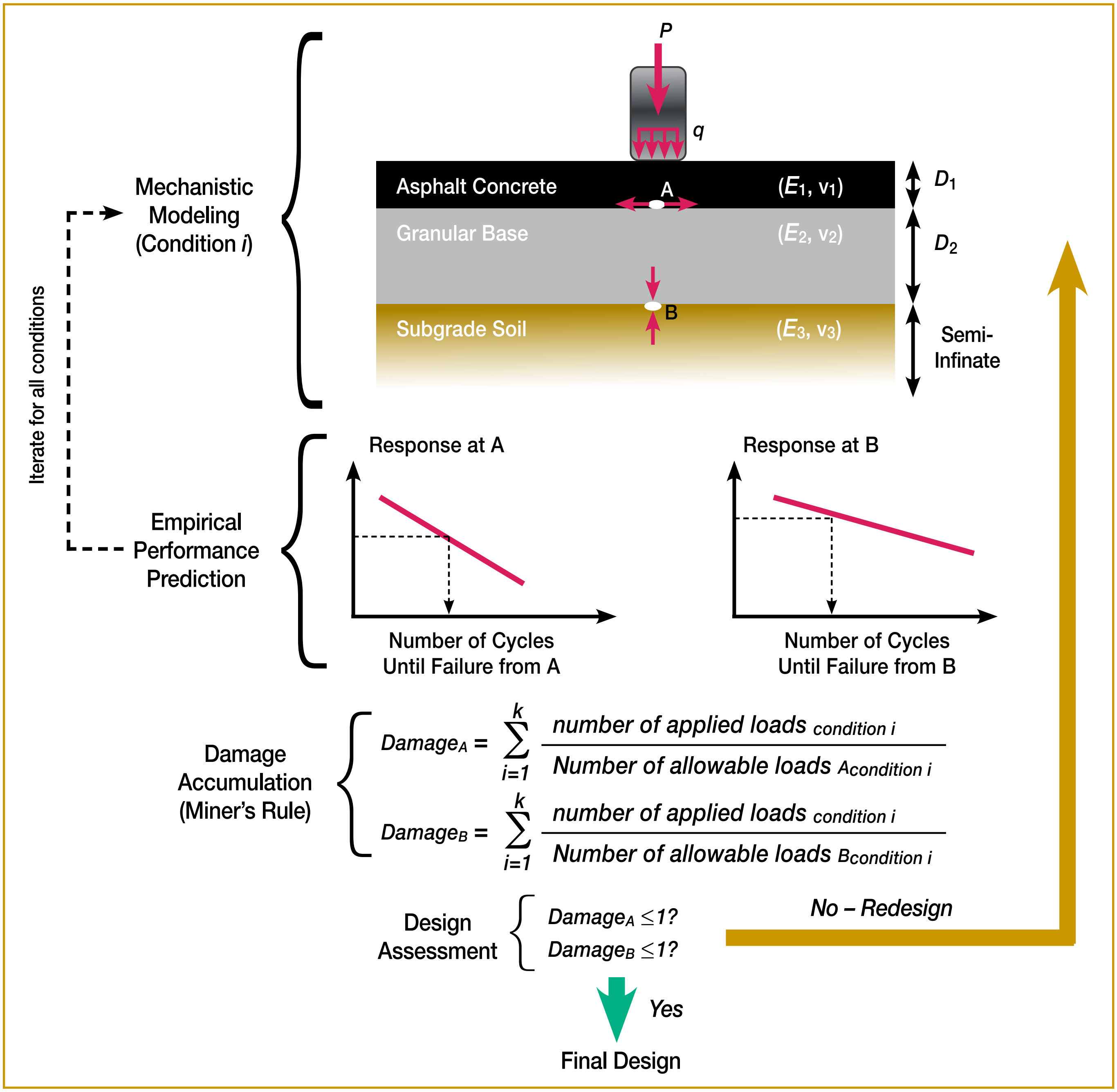

Figure 3-1. M-E Framework (Timm et al., 2014)

MECHANISTIC MODELING

M-E design begins with a trial cross-section that will be evaluated under the prevailing design conditions. The trial cross-section includes the material types, which dictates the material properties, in addition to the layer thicknesses. Figure 3-1 shows a simple three-layer pavement with AC over granular base over the subgrade soil. Each layer is defined by its elastic modulus (E_1, E_2, E_3), Poisson ratios (v_1, v_2, v_3) and thicknesses (D_1, D_2). A loading event is simulated at the pavement surface to cause pavement responses at critical locations in the structure. As shown in Figure 3-1, a single tire with known weight (P) and tire pressure (q) cause horizontal tension at the bottom of the asphalt concrete layer at Point A and vertical compression in the subgrade soil at Point B. These points are usually used to predict bottom-up fatigue cracking and structural rutting, respectively.

Note also from Figure 3-1 that the mechanistic modeling represents condition i with a loop for all conditions. This is an important consideration because pavements are in a nearly constant state of change. Daily and seasonal temperature fluctuations, changes in moisture conditions, and varying load configurations and axle weights all contribute to a range of pavement responses that must be accounted for in design. M-E design is perfectly suited to handle this complexity, as will be discussed in the Damage Accumulation subsection. More immediately, in the following subsections, are discussions of the mechanistic model, material properties and traffic characterization.

MECHANISTIC MODEL

There is a wide variety of available mechanistic models that compute pavement responses to external loadings. They range from relatively simple equations with overly simplistic assumptions about the pavement to extremely complex models requiring tremendous computational resources to execute a simulation. The current state-of-the-practice is to use a mechanistic model based on linear layered elastic theory. This theory relies on a formulation using Hooke's law and assumes that all materials behave elastically in response to external loadings. Other assumptions within a layered elastic model include:

- **All layers are infinite in the horizontal direction.**
- **All materials are homogeneous.**
- **All materials are isotropic.**
- **The bottom layer (i.e., the subgrade) is infinite in the downward direction.**

These assumptions and Hooke's law were first formulated by Burmister (1943; 1945; 1958) for analysis of layered soil systems. However, his formulation is also applicable to multi-layer flexible pavements and forms the basis of most modern linear layered elastic computer programs. These programs include JULEA, WESLEA, ELSYM5, PAVEXpress and CHEVNL, to name a few. Regardless of the program, the purpose of each is to take a given set of inputs (i.e., material properties and loading conditions) and predict pavement responses (i.e., stress, strain, and/or deflection). There are some differences between the programs, but they are too subtle for this discussion.

MATERIALS CHARACTERIZATION

The material properties required for pavement design are largely governed by the selected pavement model. In the case of linear layered elastic theory, the two primary material properties are the elastic modulus (E) and the Poisson's ratio (v), as indicated in Figure 3-1. To explain these concepts, consider Figure 3-2, where a solid cylinder with initial length (L_0) and initial diameter (d_0) is subjected to a vertical stress (σ) resulting in a deformed shape (L_f, d_0). When the stress is removed, the cylinder returns to its original shape.

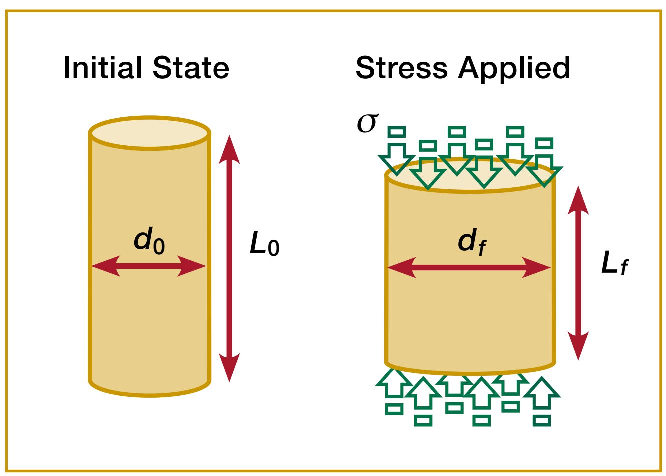

Figure 3-2. Material Deformation Under Stress

Using the dimensions in Figure 3-2, the strain in the cylinder is defined in the axial (ε_L) and transverse (ε_d) directions according to the change in dimension divided by the original dimension:

$$\varepsilon_L = \frac{L_f - L_0}{L_0} \qquad (3\text{-}1) \qquad\qquad \varepsilon_d = \frac{d_f - d_0}{d_0} \qquad (3\text{-}2)$$

Using Equations 3-1 and 3-2, Poisson's ratio is simply the transverse strain divided by the axial strain, multiplied by negative one to produce a positive ratio, or:

$$\nu = -\frac{\varepsilon_d}{\varepsilon_L} \qquad (3\text{-}3)$$

Finally, the elastic modulus, or Young's modulus, represents the slope of the stress-strain curve when a material is loaded below its elastic limit. The elastic modulus is:

$$E = \frac{\sigma}{\varepsilon_L} \qquad (3\text{-}4)$$

where:
E = elastic modulus, psi
σ = applied stress, psi
ε_L = resulting axial strain, in./in.

It is important to recognize that the modulus of AC will depend on the temperature of the material and speed of loading (i.e., traffic speed). The modulus of unbound materials will depend on the state of stress and moisture conditions. Therefore, it is critical to characterize the materials under a variety of conditions to capture the range of properties the pavement will experience and model the conditions within the M-E framework.

Figure 3-3 illustrates the impact seasonal temperature changes have on the AC modulus. The data were obtained from falling weight deflectometer testing at the National Center for Asphalt Technology (NCAT) Test Track, which clearly show the annual cycling of modulus as temperatures warm and cool throughout the two-year time period. Figure 3-4 plots the same moduli data from Figure 3-3, but versus temperature rather than date. Again, the influence of temperature is readily evident and should be incorporated in the mechanistic modeling by subdividing time into short increments to represent the range of in-place conditions.

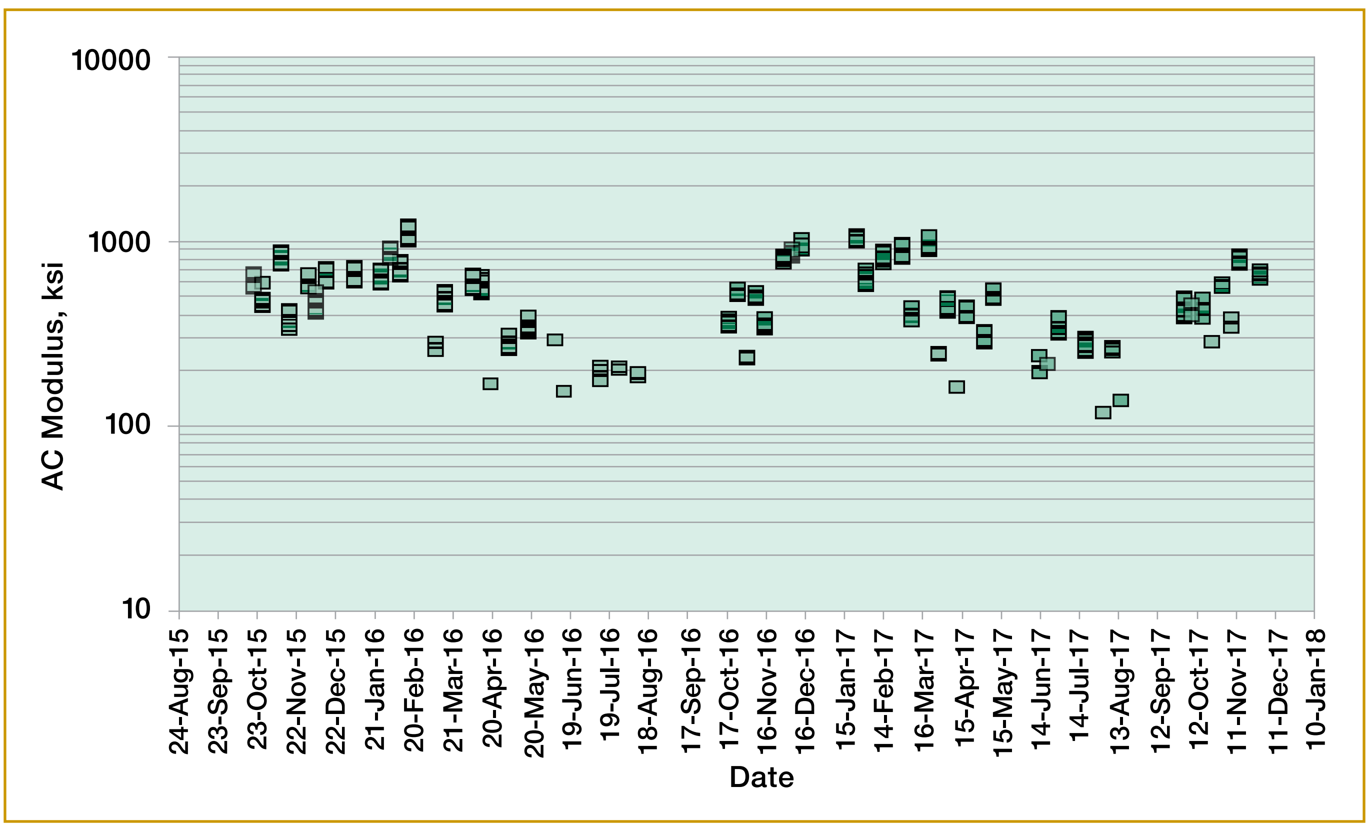

Figure 3-3. In-Place AC Modulus Versus Test Date at the NCAT Test Track

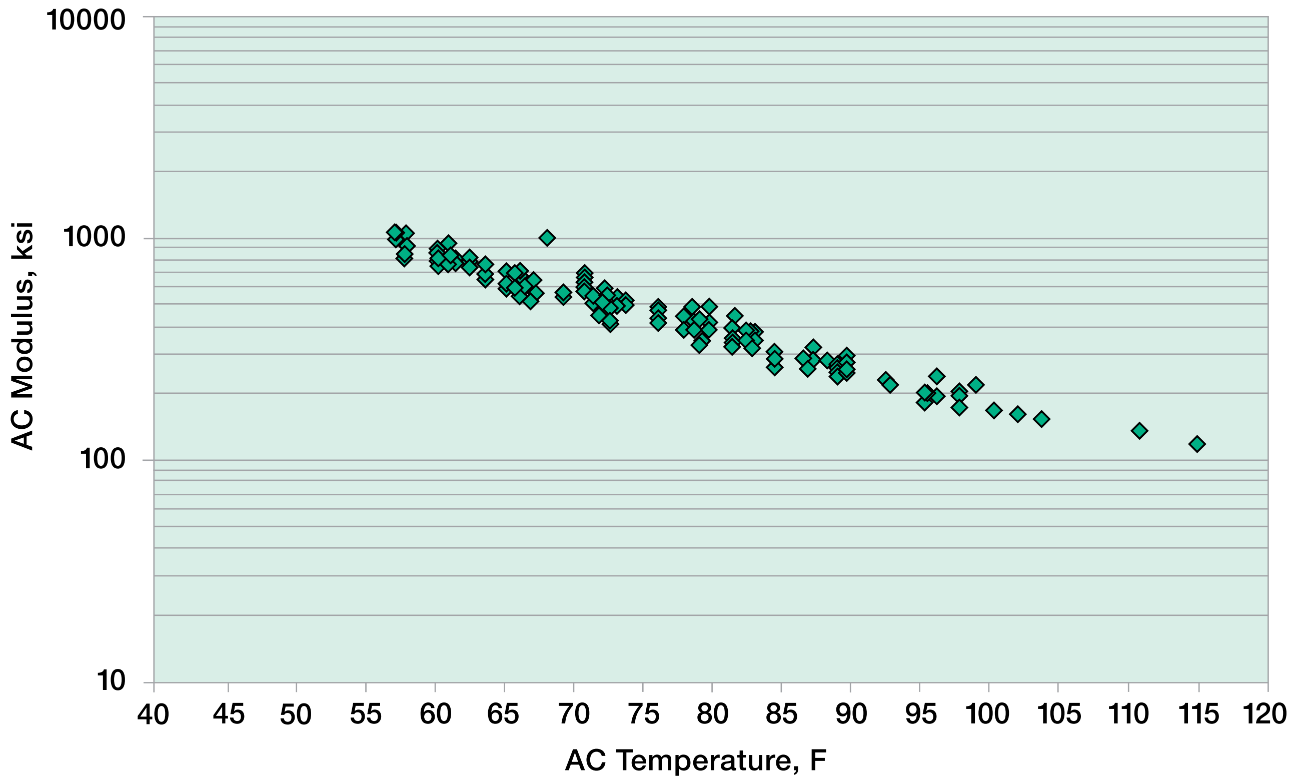

Figure 3-4. **In-Place AC Modulus Versus AC Temperature at the NCAT Test Track**

As mentioned above, the AC modulus also changes with traffic speed which, in turn, will affect the strain response of the pavement. Figure 3-5 illustrates this phenomenon with strain measurements at the NCAT Test Track from several test sections at various truck speeds. As the speed increases, the modulus increases, which results in lower measured strain.

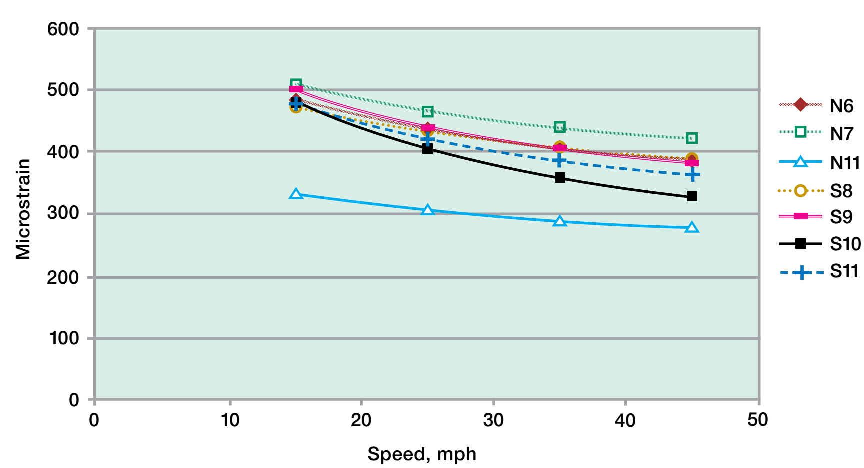

Figure 3-5. **Measured AC Strain Versus Truck Speed at the NCAT Test Track (Ellison & Timm, 2011)**

Figures 3-3 through 3-5 illustrate the various conditions that arise due to changes in AC temperature and traffic speed which should be modeled in M-E design. For design purposes, the influence of temperature and speed is often determined in the laboratory according to AASHTO TP 79, *Standard Method of Test for Determining the Dynamic Modulus and Flow Number for Asphalt Mixtures Using the Asphalt Mixture Performance Tester (AMPT)*. This test protocol determines the dynamic modulus ($|E^*|$) of asphalt mixtures at a range of temperature and loading frequencies to represent the real-world conditions the material will encounter while in service. The M-E design procedure then selects $|E^*|$ values corresponding to specific temperature and traffic speed combinations.

As mentioned above, unbound materials will exhibit a range of modulus values as a function of the state of stress and moisture conditions. Therefore, it is important to characterize the materials across the range of expected conditions and select representative values for design purposes. The testing is usually done according to AASHTO T 292, *Standard Method of Test for Resilient Modulus of Subgrade Soils and Untreated Base/ Subbase Materials.*

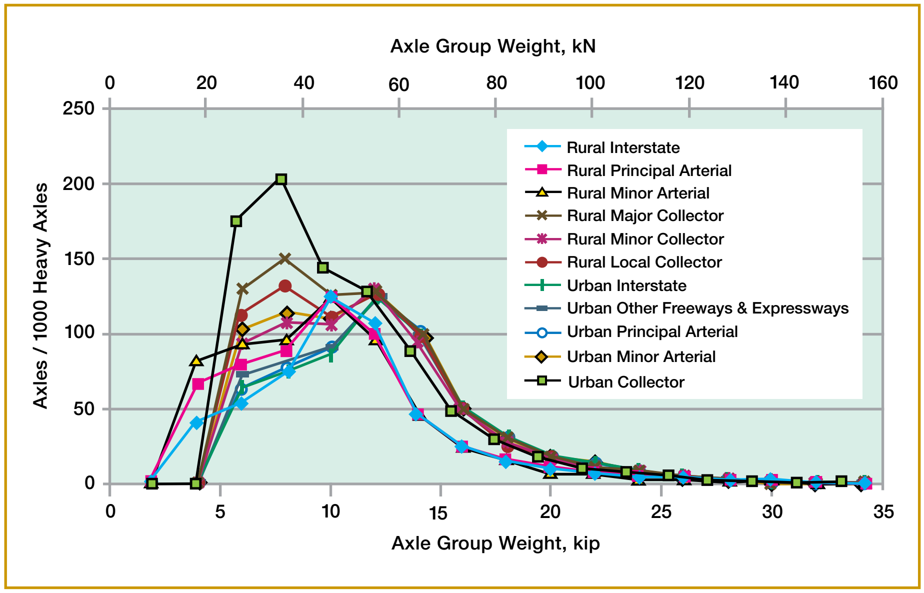

Figure 3-6. **Single Axle Load Spectra**

TRAFFIC CHARACTERIZATION

Most pavements experience a wide range of traffic loadings. From passenger cars to heavily loaded trucks, the pavement design process must consider the array of vehicles, axle configurations, tire pressure, and axle weights to arrive at an optimal pavement structure. Ultimately, M-E design requires all these factors to be arranged into so-called load spectra, which represents actual tire weights, pressures, and spacings to be simulated within the mechanistic model.

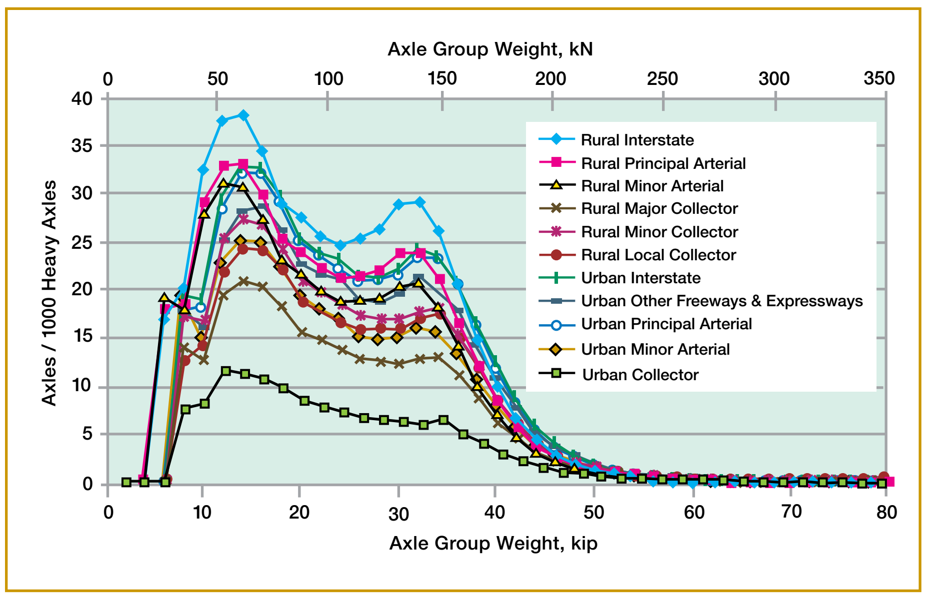

Figure 3-7. **Tandem Axle Load Spectra**

Figure 3-6 and Figure 3-7 illustrate typical load spectra for single and tandem axles, respectively. The graphs subdivide load spectra according to roadway classification to better represent conditions on specific facility types. For example, the urban collector and rural interstate single axle load distributions (Figure 3-6) represent dramatically different loading conditions, especially at the lighter axle weights. The same is true for tandem axle weights (Figure 3-7). As described above, within the M-E framework (Figure 3-1), the load spectra are used to generate specific loading conditions for simulation by the mechanistic model. Each loading condition yields a particular pavement response, which is then used to predict pavement damage accumulation. It is important to emphasize that load spectra should be as accurate as possible for a specific design to produce optimal pavement cross-sections.

EMPIRICAL PERFORMANCE PREDICTION

The second major component of M-E design is the empirical performance prediction. In this component, the mechanistic pavement responses are used with empirical transfer functions to predict the number of allowable load repetitions until pavement failure (N_f). These predictions are made on a distress-by-distress basis and bottom-up fatigue cracking and rutting are the two most common types of transfer functions. Though there are more complicated equations to predict N_f, the simplest is:

$$N_f = k_1 \left(\frac{1}{\varepsilon}\right)^{k_2} \qquad (3\text{-}5)$$

where:

N_f = number of cycles until failure

ε = strain response of pavement, in./in.

k_1, k_2 = empirical coefficients

Figure 3-1 illustrates Equation 3-5 as two separate graphs correlating responses at points A and B in the pavement to number of allowable cycles until failure at those points. These represent bottom-up fatigue cracking and rutting, respectively.

Many studies have developed calibration coefficients for M-E design and range from national to regional to state-level in scope. When considering using previously-developed transfer function coefficients, to ensure optimal pavement cross-sections, designers should consider implementing a three-step process of:

- **Evaluation**
- **Calibration**
- **Validation**

Evaluation consists of checking the predictive capability of an existing transfer function with local pavement performance data. Figure 3-8 shows an evaluation done with the MEPDG using default national calibration factors to predict rutting of pavements at the NCAT Test Track. There was clear over-prediction of rutting, which would produce over-conservative pavement designs. The coefficients of the rutting transfer function were then calibrated to minimize the error between measured and predicted rutting performance (Figure 3-8). Finally, an independent set of test sections was used to validate the locally calibrated transfer function (Figure 3-9). While the rutting predictions made with the locally calibrated transfer function were reasonably accurate for most of the sections, there were two sections (N1-06 and N2-06) that had notable over-predictions. This highlights one of the major deficiencies of M-E design; it still relies on empirical data and may not accurately predict future pavement performance.

Further details regarding the conduct of evaluation, calibration, and validation studies is fully described in the *Guide for the Local Calibration of the MEPDG* (AASHTO, 2010). AASHTO strongly recommends agencies perform local calibration before using the MEPDG for routine pavement design.

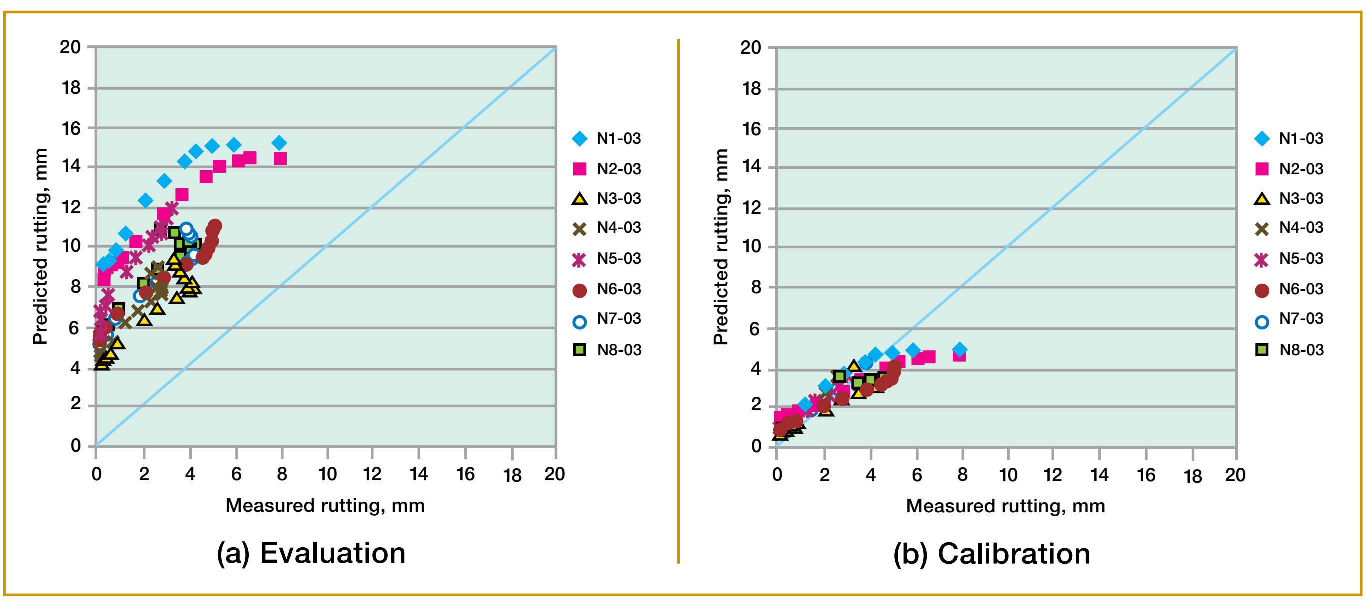

Figure 3-8. Evaluation and Calibration of Rutting Data at the NCAT Test Track (Guo & Timm, 2015)

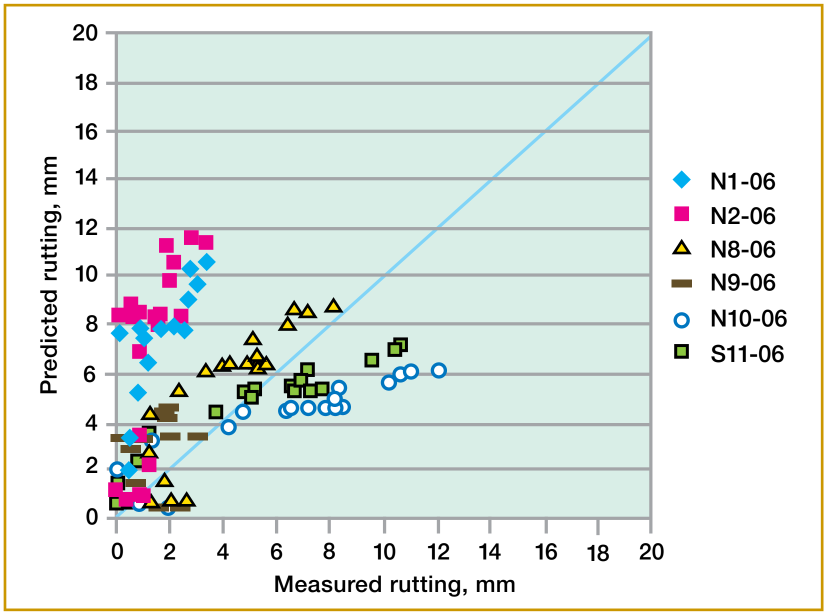

Figure 3-9. Validation of Locally Calibrated Rutting Transfer Function (Guo & Timm, 2015)

As noted in Figure 3-1, damage is determined on a distress-by-distress basis. Again, location A is used to predict fatigue damage and location B predicts rutting. One of the two distresses will control the design, which is dependent on the design inputs.

DESIGN ASSESSMENT

After computing relative damage at each critical location, an assessment is made based on the relative damage values. If total damage of any distress exceeds 1.0 then the pavement is under-designed and the layer thicknesses should be increased. If total damage is well below 1.0 then the pavement is over-designed and the layer thicknesses should be decreased. The final, optimized, design should have damage values just below 1.0.

DAMAGE ACCUMULATION

Once the M-E design system has been properly calibrated and validated, it may be used to accurately predict pavement damage accumulation. As noted in the Figure 3-1 framework, there is an iterative loop to account for the various pavement conditions (i.e., material properties and loading conditions). Each condition produces a prediction of the number of allowable loads until failure (N_{fi}). This is combined with the number of applied loads in that condition (n_i) to compute the amount of relative damage (D). The equation that sums the damage across all the conditions is known as Miner's rule, and is expressed as:

$$D = \sum_{i=1}^{k} \frac{n_i}{N_{fi}} \qquad (3\text{-}6)$$

where:

D = relative pavement damage

n_i = number of applied loadings in condition i

N_{fi} = number of allowed loadings until failure in condition i

i = pavement materials and loading condition

k = all possible loading conditions

LIMITATIONS OF M-E DESIGN

M-E design represents a tremendous advancement over earlier design approaches. The ability to adapt more readily to new conditions and predict specific modes of distress through greater reliance on computational mechanics are significant improvements. However, there are two notable limitations of M-E design that require discussion.

First, M-E design has a significant empirical component. As discussed above, any agency implementing M-E design should execute evaluation, calibration, and validation of the transfer functions. These studies can be costly and take several years to complete. Also, depending on data availability, it may be cost- or time-prohibitive to execute these studies.

Second, M-E design, like the earlier empirical design approach, has no mechanism for arriving at a maximum pavement thickness. As additional traffic is added to the design, thicknesses will increase. This has the potential to arrive at unreasonably thick and cost-prohibitive pavement cross-sections. There should be some maximum thickness that can be determined from a given set of design inputs.

PERPETUAL PAVEMENT DESIGN

Perpetual or long-life pavements are structures that experience no deep distresses such as bottom-up fatigue cracking or structural rutting. These pavements, therefore, have a perpetual foundation that only require periodic (e.g., every 15–20 years) surface treatments to mitigate top-down cracking or other surface distresses that may develop. The structural design procedure for perpetual design may be considered a subset of the M-E approach described above with two notable modifications as described below.

Figure 3-10 illustrates a Perpetual Pavement framework that is nearly identical to the original M-E framework shown in Figure 3-1. The first major difference is the endurance limits in the Empirical Performance Prediction component. The endurance limits are material- and distress-specific pavement response values below which damage will not accumulate. For the tensile response at location A in Figure 3-10, the endurance limit corresponds to bottom-up fatigue cracking in the AC. For the response at location B, the endurance limit corresponds to structural rutting predictions made with strain in the subgrade layer. Traffic loadings causing pavement responses below either endurance limit will not contribute to pavement damage accumulation, resulting in infinite performance life prediction (e.g., horizontal dashed line in Figure 3-10). Traffic loadings causing pavement responses above the endurance limit, according to Figure 3-10, will cause damage to accumulate and transfer functions may be used to predict the amount of damage. The goal of Perpetual Pavement design, according to this framework, is to design the pavement thickness such that most of the applied traffic loads cause pavement responses below the respective thresholds (i.e., no damage accumulation).

The second major difference, depicted in the Design Assessment component of Figure 3-10, is that damage must be much less than 1.0 from Miner's rule. Since 1.0 would represent failure of the pavement in a specific mode, a Perpetual Pavement should be designed such that the damage is much less than 1.0, or even approach 0.

Based on these two major differences, it logically follows that determination of the endurance limit is critical to successful Perpetual Pavement design. What follows is a short history of endurance limits for bottom-up fatigue cracking which leads to a relatively new concept in Perpetual Pavement Design that features strain distributions rather than single value endurance limits to prevent this distress. Rutting, on the other hand, has been documented to be well-controlled through a single value, which will also be described below.

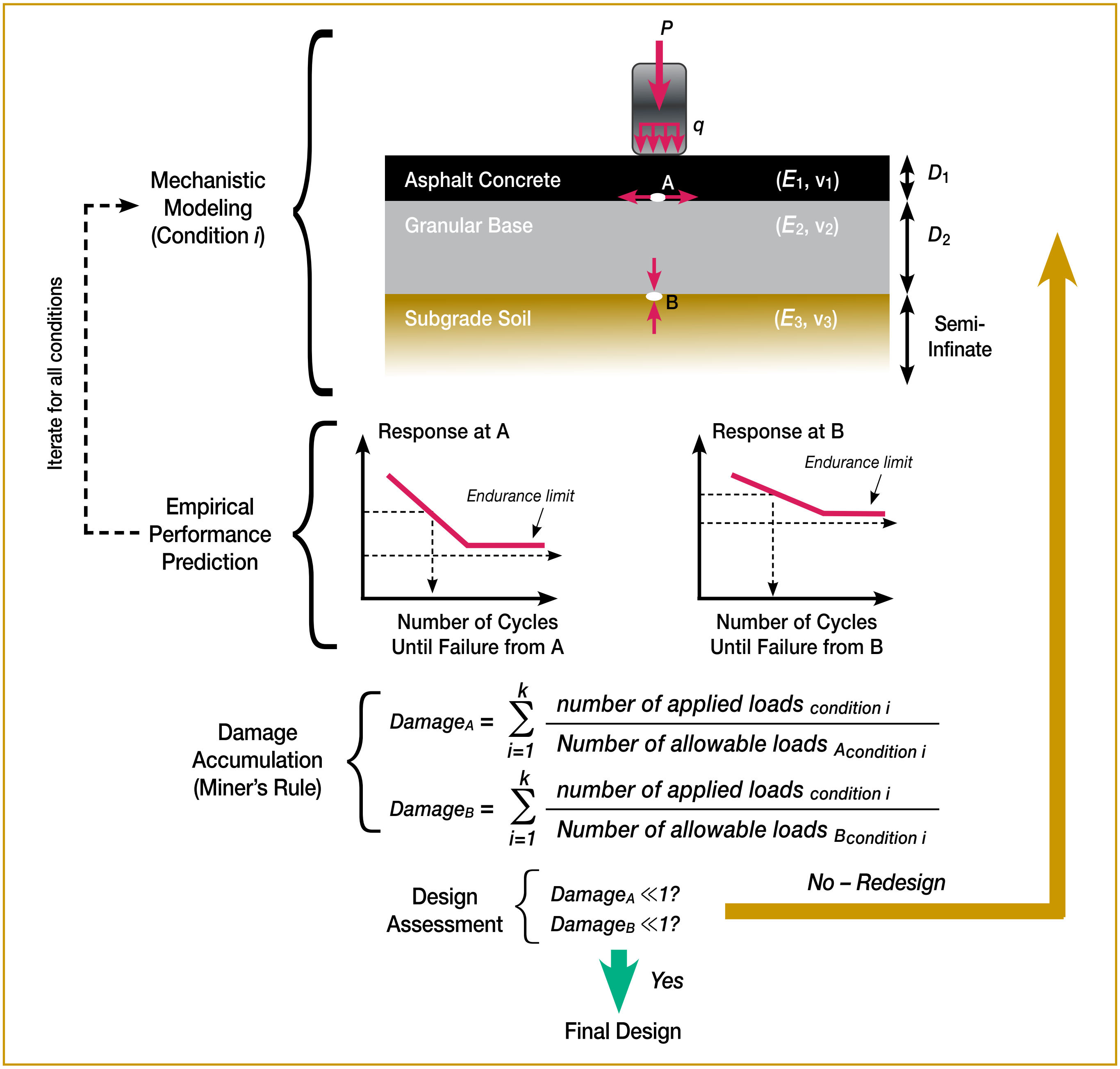

Figure 3-10. **Perpetual Pavement Design Framework (Timm et al., 2014)**

BOTTOM-UP FATIGUE CRACKING ENDURANCE LIMIT

Monismith & McLean (1972) first reported a fatigue endurance limit (FEL) of –70 $\mu\varepsilon$ based on laboratory bending beam fatigue testing. Many years later, Thompson & Carpenter (2006) stated that –70 $\mu\varepsilon$ represents a minimum FEL as no lab data were found below this value. They also recommended a practical range of –70 to –100 $\mu\varepsilon$. Four years later, Prowell et al. (2010) published laboratory FEL data ranging from –75 to –200 $\mu\varepsilon$. Clearly, the first estimate of –70 $\mu\varepsilon$ was a sound conservative lower bound for the FEL.

Even higher laboratory FELs, ranging from –90 to –300 $\mu\varepsilon$, were found from testing 120 different mixes by Carpenter & Shen (2009). Their study found that the mixture gradation had relatively little effect on the FEL while the mixture volumetric properties and binder type were more influential.

A study of in-service pavements in Japan found FELs around –200 με (Nishizawa et al., 1997) while a study of the Kansas Turnpike computed strain levels ranging from –96 to –158 με in long-life pavements (Wu et al., 2004). A Perpetual Pavement experiment in China, under extremely heavy traffic loadings, showed –125 με to be effective in designing Perpetual Pavements (Yang et al., 2006).

Each of the above studies largely focused on determining a single strain value for design and analysis purposes. However, pavements experience a wide range of strain responses due to variations in traffic load levels, environmental conditions, and aging. Therefore, it makes sense that one should consider a range of strain values in the context of Perpetual Pavements. Using that underlying concept, Willis & Timm (2009) investigated a range of Perpetual and non-Perpetual Pavements at the NCAT Test Track. They found that a control strain distribution could be used to distinguish between pavements that do not (Perpetual) and do (non-Perpetual) experience bottom-up fatigue cracking. Their original strain distributions were validated against Perpetual Pavement sections around the U.S. from which a recommended Perpetual Pavement strain control distribution was developed (Tran et al., 2015).

By way of example, Figure 3-11 shows the recommended strain distribution (labeled "Target") and two other distributions generated by trial pavement cross-sections with a fixed set of design conditions. The first trial failed because it had strain levels exceeding the target at the lower part of the curve. The second trial was successful as all points were above and to the right of the Target distribution (i.e., lower strain levels). Table 3-1 also lists the control points of the design (Target) distribution. It is important to note that generating a strain distribution, as depicted in Figure 3-11, requires a design tool such as PerRoad, which will be discussed in the next section.

Table 3-1. Recommended Control Strain Distribution and Strain Ratios (Tran et al., 2015)

Percentile	Microstrain	Strain Ratio
95	-257	1.73
85	-194	1.31
75	-158	1.06
65	-131	0.88
55	-110	0.74

The strain values in Table 3-1 were developed from mixtures that had FELs of around –150 με in the laboratory. Willis & Timm (2009) extended their findings to a wider range of mixtures through the use of strain ratios whereby a designer multiplies a laboratory FEL by the series of strain ratios listed in Table 3-1 to find a mixture-specific control distribution (Tran et al., 2015). For example, if laboratory testing determined a FEL of –200 με, then Table 3-2 would be the recommended control strain distribution found by multiplying –200 by the corresponding strain ratios from Table 3-1. This approach allows for designs using mixtures outside the scope of the NCAT Test Track.

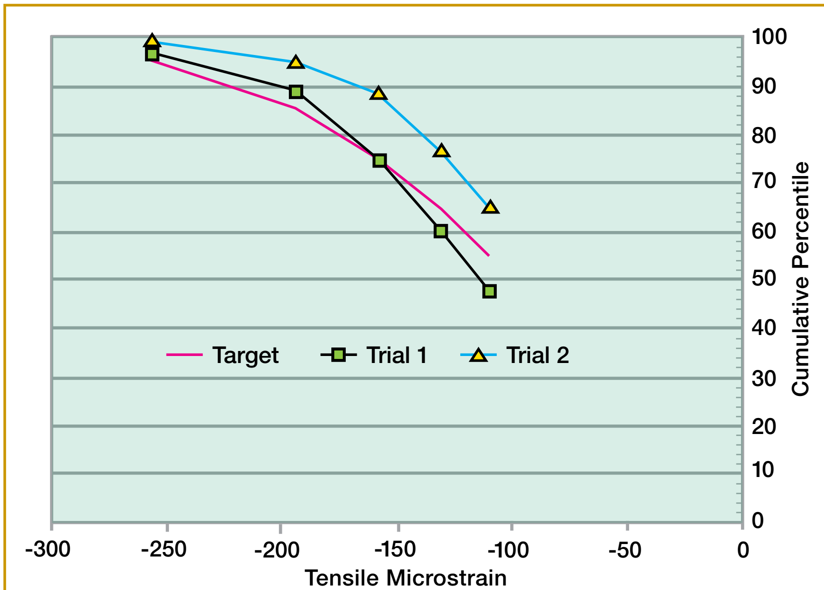

Figure 3-11. Example Design with NCAT Test Track Strain Distribution

Table 3-2. Example Control Strain Distribution for –200 με FEL

Percentile	Microstrain
95	-346
85	-262
75	-212
65	-176
55	-148

RUTTING ENDURANCE LIMIT

Rutting deep in the structure, in the aggregate base or subgrade or both has traditionally been controlled by the vertical compressive strain in the subgrade. For Perpetual Pavement design, there has been some consensus of limiting this strain to less than –200 με (e.g., Monismith et al., 2004; Walubita et al., 2008). A recently completed investigation that evaluated Perpetual Pavements in various states confirmed that limiting the 50th percentile compressive strain in the subgrade to less than –200 με was a conservative and viable design criterion (Castro et al., 2017).

PERPETUAL PAVEMENT DESIGN TOOLS

Design tools for Perpetual Pavement follow one of two general approaches. The first approach uses single values for endurance limits. The designer selects layers thicknesses to produce pavement responses that are predominantly below the respective endurance limits (i.e., no damage accrual). Pavement responses that exceed the endurance limit will be used in transfer functions to estimate the amount of damage caused by the loading event and sum up the damage with Miner's rule over successive loading events. To achieve a Perpetual Pavement, the designer must then keep damage to an extremely low (or zero) level. This approach was described previously and depicted in Figure 3-10. The second approach considers ranges of strain responses and applies strain distributions, or control points at specific percentiles, to achieve a Perpetual Pavement. This approach was also described previously and is depicted in Figure 3-12. Both approaches are viable and will be discussed further in the context of the AASHTOWare Pavement ME Design software and the PerRoad and PerRoadXPress computer programs.

AASHTOWARE PAVEMENT ME DESIGN

As noted previously, the current AASHTO design standard is the Mechanistic-Empirical Design Guide (MEPDG) and accompanying design software, AASHTOWare Pavement ME Design. The primary purpose of the MEPDG is to design pavements according to a traditional M-E framework that predicts damage over time, as laid out in Figure 3-1. However, the Pavement ME Design software does have the capability to incorporate fatigue endurance limits following the first approach described above. Figure 3-12 shows where the designer may enable an endurance limit and specify the value in version 2.5 of the software.

Enabling the endurance limit feature, and specifying the single design threshold, will place a lower bound on the fatigue transfer function as depicted in Figure 3-10. Any strain values below the endurance limit will produce no damage while strain values in excess of the endurance limit will be used in the fatigue cracking transfer function and Miner's hypothesis to compute damage. Ideally, the designer should aim for 0% fatigue cracking to achieve a Perpetual Pavement under these conditions.

It should be noted that the Pavement ME Design software only considers an endurance limit for bottom-up fatigue cracking. There are no inputs or settings for structural rutting. However, a designer could achieve a Perpetual Pavement in this framework by selecting materials and thicknesses such that rutting is confined to the AC layers only. This would meet the definition of Perpetual, though may be overly conservative as an endurance limit for the subgrade was not utilized.

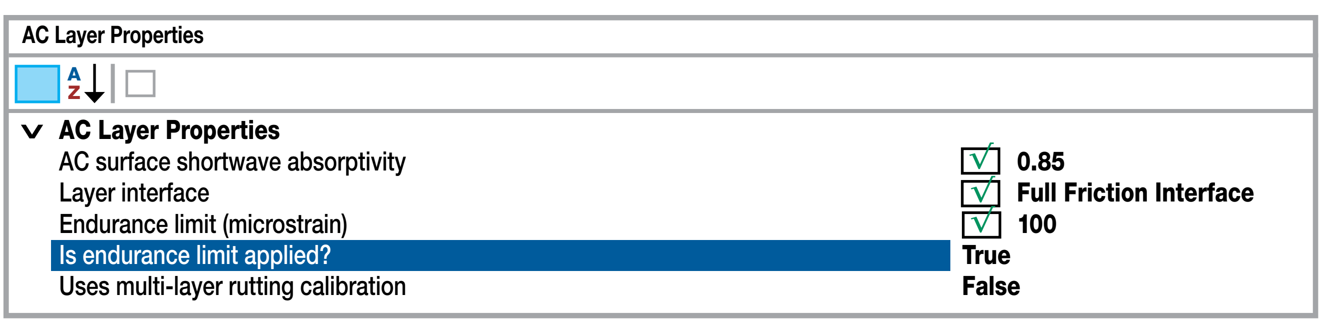

Figure 3-12. AASHTOWare Pavement ME Design Version 2.3.1 Endurance Limit Settings

PERROAD

PerRoad is a flexible pavement thickness design tool originally intended and developed for Perpetual Pavement design. It has the capability to conduct designs using either the first or second approach described above.

When following the first approach, PerRoad executes a procedure very similar to the AASHTOWare Pavement ME Design software. Figure 3-13 shows the settings and inputs for this type of analysis from PerRoad version 4.4. Reading from left to right in Figure 3-13, the designer selects the bottom of the AC layer, specifies horizontal strain as the control and specifies a single microstrain value. The designer then specifies appropriate calibration coefficients for the transfer function. The values in Figure 3-13 are for illustrative purposes and should not be taken as global values. Like the AASHTOWare program, PerRoad then assigns zero damage to any responses below the endurance limit and computes damage for any responses above the limit. Unlike the AASHTOWare program, however, PerRoad estimates the amount of time until damage reaches 0.1 on the 0 to 1 Miner's rule scale. For a long-life pavement, the time should be at least 35 years.

When using the second approach, the designer can use either the default NCAT Test Track strain distribution or strain ratios (Table 3-1) to control fatigue cracking. Figure 3-14 shows where horizontal strain distribution at the bottom of the AC has been selected. The NCAT default distribution has been loaded into the software by clicking the "Load Default Distribution" button. Alternatively, default strain ratios could be applied to a single laboratory value by clicking the "Enter Endurance Limit" button. In either case, this executes a design as depicted in Figure 3-12 where the control distribution is compared against the actual distribution to evaluate the quality of the design. A simple pass/fail assessment is given in the output window of the software. A complete help file explains the software in much greater detail at: http://www.eng.auburn.edu/users/timmdav/PerRoad_43.html

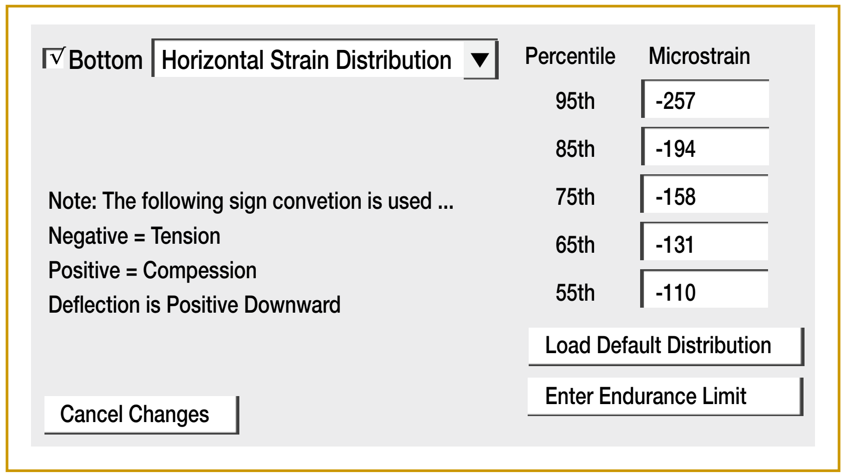

Figure 3-14. PerRoad Version 4.4. Strain Distribution for Bottom-Up Fatigue

It should be mentioned that PerRoad can also consider a single strain value and corresponding percentile. Recall that –200 με at the 50th percentile was recommended to control structural rutting. Figure 3-15 illustrates this set of controls and inputs with each labeled, accordingly. When PerRoad executes the design simulation, a simple pass/fail is also reported based on whether the 50th percentile vertical strain exceeds the target (fail) or falls below (pass).

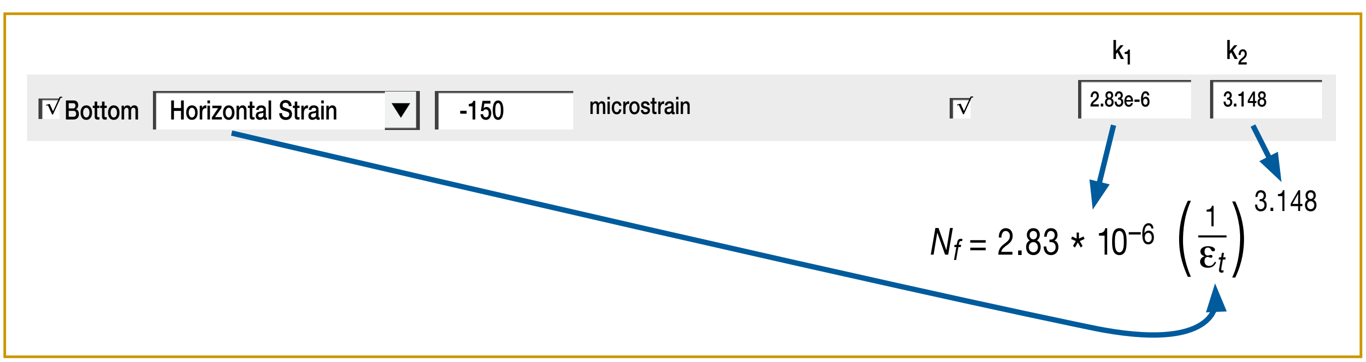

Figure 3-13. PerRoad Version 4.4. Single Value Endurance Limit and Transfer Function

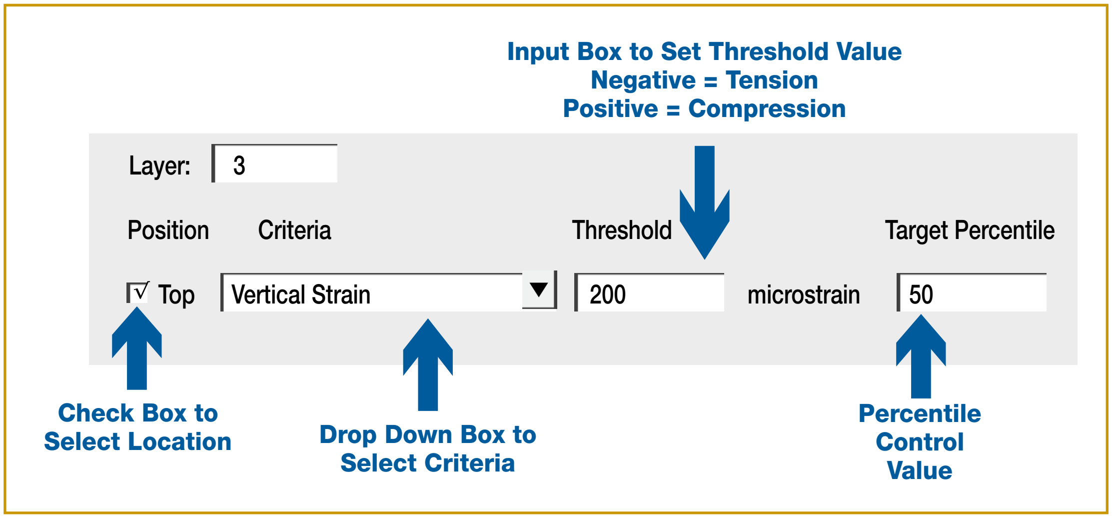

Figure 3-15. PerRoad Version 4.4 Strain Percentile for Rutting

PERROADXPRESS

The final design tool to discuss is PerRoadXPress, version 1.0 (Figure 3-16). This program was derived from the full PerRoad program but with a limited set of design conditions meant for lower volume roads where designers may lack the full-level of detail necessary to run PerRoad or AASHTOWare Pavement ME Design software. As shown in Figure 3-16, there are a limited set of inputs (with limited ranges) required to execute the design. PerRoadXPress follows the first approach described above wherein endurance limits and transfer functions were used in the perpetual framework to predict the required thickness needed to have 35 years until damage equals 0.1. It should be noted that the endurance limit for fatigue and rutting, and corresponding transfer functions, are hardcoded into the program. More details regarding PerRoadXPress are documented elsewhere (Timm et al., 2006).

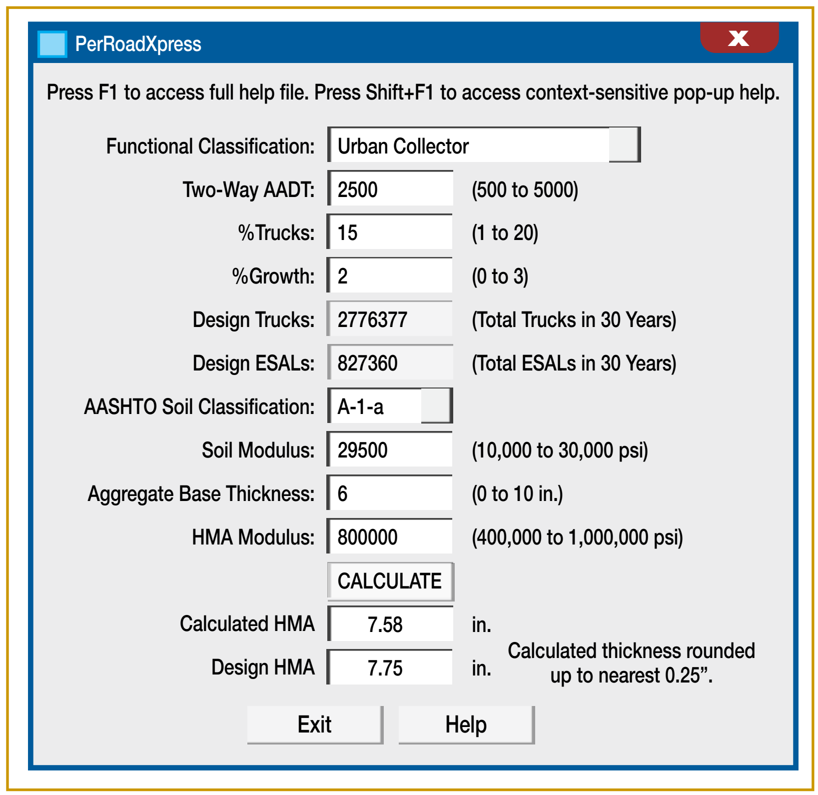

Figure 3-16. PerRoadXPress Version 1.0

SUMMARY

Thickness design of Perpetual Pavements is viable within an M-E framework. Key to the success of Perpetual Pavement design is recognizing, measuring, and using endurance limits to prevent bottom-up fatigue cracking and deep structural rutting. Existing methods and software, such as the AASHTOWare Pavement ME and PerRoad programs, can execute Perpetual Pavement design following one of two approaches. The first approach maintains the use of transfer functions, but with a lower limit below which damage does not accumulate. The second approach eliminates the need for transfer functions and relies on strain distributions for design purposes. The AASHTOWare Pavement ME software only uses the first approach and is limited to bottom-up fatigue cracking. PerRoad version 4.4 can handle either approach and design for both bottom-up fatigue cracking and deep structural rutting.

4 CONSTRUCTION

The commitment to construct a Perpetual Pavement requires that all parties involved agree to provide the practices, materials, and oversight necessary to ensure a high level of performance. The decision to consider a long-life pavement was made during planning as a means of reducing life-cycle costs, including user costs. The design phase has provided the plans showing the layer compositions and thicknesses necessary to support the expected heaviest loads in the mix of traffic. Construction is where the meaning of the design becomes a reality, and it is where the greatest amount of attention must be paid in order to ensure performance.

The main premise in Perpetual Pavement design is to avoid all conditions that would result in failures deep within the structure. Thus, the design and the approach to construction must be “bottom-up” in that the soils, granular base, asphalt base, asphalt binder, and asphalt surface layers are given the appropriate attention to ensure long-term performance. In Finland, Väylä, the Finnish Transport Infrastructure Agency, uses the approach that subgrade materials should be designed to last at least 100 years, granular materials between the subgrade and paved layers should be designed to last 60 years, and the surface layers should be designed to last 30 years with periodic resurfacing for damage from studded tires. This is an example of the type of philosophy that leads to good long-term performance.

If the Perpetual Pavement design is a part of a rehabilitation or reconstruction project, then much of the investigation may need to be done non-destructively to provide a broader coverage of the area to be evaluated and help ensure the safety of engineering personnel. There are several tools that can be used to determine the structural condition of the pavement, along with layer thicknesses, the presence of moisture pockets, uniformity of materials, and the state of bond between the layers (Saarenketo & Scullion, 1994). This information is necessary for design and is important to construction as it might dictate the location and extent of areas to be patched or otherwise repaired and the depth of milling to avoid delamination problems. Tools available to conduct these surveys include ground-penetrating radar (GPR), the falling-weight deflectometer (FWD), and light detection and ranging (LIDAR) devices as shown in Figure 4-1.

FOUNDATION

The foundation of a Perpetual Pavement begins with the bottom of the treated or compacted subgrade and extends to the bottom of the asphalt layers. It is imperative that the foundation be properly designed and constructed. As layers are improved through compaction and/or chemical stabilization each of them serves not only as part of the pavement’s structural capacity but as the platform for constructing the layer

Figure 4-1. Examples of Nondestructive Methods for Pavement Evaluation

Figure 4-2. Lime Stabilization of Soil

above it. Each layer provides the support for resisting deformation under compaction so that the density may be optimized.

Subgrade stabilization is accomplished in several different ways, depending upon the nature of the soil and the proximity to water. For clay soils with high plasticity indexes, lime stabilization is often used. Lime stabilization, shown in Figure 4-2, helps to dry the clay as well as to flocculate clay particles to reduce the risk of volume change during service. Lime is not an appropriate treatment for clays bearing sulfate minerals. Subgrades with coarser particles may benefit from the use of portland cement, asphalt emulsion, fly ash, chemical admixtures, or foamed asphalt. Most subgrades can be adequately strengthened to a good state of load bearing by following best practices for compaction. Before deciding upon a plan for stabilizing soils, it is best to consult historical records for the area and to seek advice from geotechnical engineers.

The stability of granular layers is normally achieved through mechanical compaction of the material. However, in instances where roadway elevations may be restricted, chemical stabilization through the application of cement, asphalt emulsion, or foamed asphalt may be helpful. These stabilizers will provide strength and stability beyond what is possible with compaction alone. However, it is important to note that an excess of cement will probably lead to cracking in the base, which will propagate through the surface of pavement. As with the subgrade, it is important that the granular layers provide additional structural capacity as well as resistance to deformation during the compaction of the asphalt layers.

The unbound layers should receive frequent and wide-area testing. Techniques such as proof-rolling can readily point out weak areas that need improvement or replacement. Additionally, the dynamic cone penetrometer (Figure 4-3) can provide information

Figure 4-3. Dynamic Cone Penetrometer

that is layer specific. Weaker layers will show less resistance to penetration than stronger layers, and this has been shown to be directly related to the shear strength of the soil or base material.

The Perpetual Pavement foundation is the platform on which the asphalt layers are constructed. The foundation must provide the strength for the reaction to compaction for the upper layers. If the strength of the foundation is adequate, the rest of the pavement will not fail due to excessive deformation during construction.

ASPHALT MIX TYPE SELECTION

A guide to selecting asphalt mixtures for particular applications can be found in Hansen & Garcia (2001). The constructability of a Perpetual Pavement is highly dependent upon the selection of the right combination of mixtures to ensure optimum density, long-life performance, and economy. The goal for a Perpetual Pavement is to ensure the long-term integrity of the pavement by confining distresses to the top layer, which will be periodically milled and overlaid to maintain safety and durability.

The asphalt base layer is the bottom-most asphalt mixture. While it may be subjected to a minimal amount of traffic during construction and possibly during rehabilitation, that is not its prime purpose.

The base layer is intended to provide an economical thickness to the overall structure and durability against water intrusion. The asphalt binders used in base course applications are not generally polymer modified. Although larger nominal maximum aggregate size (NMAS) mixtures are often employed for base layers, they may be subject to segregation and permeability. The consequences are a lack of uniformity and weakening which result in early failures. Durability is best achieved through the use of base mixtures with an NMAS of 19 to 25 mm or less which have a fine gradation or by using a lower design air void mixture (e.g., 3% voids instead of 4%). These mixtures will compact easier than coarse, higher air void mixtures. They will also serve to minimize non-uniformity and permeability.

Between the surface and base layers is the intermediate asphalt layer or, as it is sometimes called, the binder layer. Although the intermediate layer is subject to greater stresses than the base layer, it is not intended to carry traffic nor to be directly exposed to environmental elements for extended periods of time. If this layer is thinner than the asphalt base course, the NMAS is less than the mixtures used in the base. Usually the NMAS is 12.5 to 19 mm and, depending upon the expected temperature in the area and the thickness of the surface, the asphalt binder may be polymer-modified. For the most part, a dense aggregate gradation is used, and it is recommended that a fine gradation be used to minimize segregation and permeability. In some instances, where a stone matrix asphalt (SMA) or open-graded friction course (OGFC) is used for the surface, the intermediate layer may be comprised of a SMA mixture. SMA mixtures have the construction advantage of being relatively easy to compact due to the binder-rich mortar, which binds the stone-on-stone aggregate matrix.

The surface layer is subject to harshest conditions in the pavement structure. It is in direct contact with traffic loads and has the greatest exposure to temperature extremes and precipitation. It is in this layer that cracking or rutting is likely to initiate in a Perpetual Pavement. Load-related cracking in a Perpetual Pavement most often happens starting at the top of the pavement and propagating downward. It is a function of the aging of the asphalt binder at the surface of the pavement and the application of traffic loads. As the asphalt binder ages, it becomes more brittle and less able to bend when traffic loads are applied. Surface cracking can also happen due to tensile stresses that are developed in the pavement as it tries to contract during periods of cold temperature, creating thermal cracks.

Rutting in the surface mixture happens when the aggregate structure and binder in the mixture permanently deforms under heavy loads at high temperatures. While dense-graded mixtures can be used successfully in surface layers on high volume pavements, SMA mixtures and OGFC mixtures have excellent performance histories for higher traffic roads due to their high binder content and stone-on-stone structure, as shown in Figure 4-4. For high-volume pavements, polymer-modified binders are recommended, and it is suggested that the aggregate gradation be designed to provide a firm skeleton with a strong mortar between the particles.

Figure 4-4. Stone-on-Stone Structure of SMA Mixture

In order to maximize the density of the mixtures used in the pavement layers, it is important to design the pavement lifts to allow adequate room for the aggregate particles to reorient themselves according to their strongest packing. Empirically, for dense aggregate gradations, the best range of the ratio of lift thickness to maximum particle size has been found to be 3:1 to 5:1. It is important to note that the lift thickness is not the same as layer thickness. A *lift* is the thickness of mixture placed in one pass of the asphalt paver on the roadway, and an individual *layer* may have one or more lifts within it. These guidelines will provide the best opportunity to achieve a high density.

Asphalt mixture field compaction targets and specifications vary greatly among agencies. Many current specifications have minimum density values of 92% of theoretical maximum density and maximum values of 98%. For specifications having a statistical component such as percent-within-limits (PWL), it is desirable from a contractor's pay perspective to obtain densities of 94–96%. This level of compaction will help ensure the greatest durability of the mixture and help to maximize the contractor's pay.

Providing a high-quality asphalt structure is key to the performance of a Perpetual Pavement. This requires attention to:

- The materials and quality of construction of the pavement foundation.
- The selection of the mix type and materials to be used in each asphalt layer of the pavement.
- The proper selection of lift thickness for each type of mixture.
- The use of in-place density specifications to encourage high levels of in-place density.

ASPHALT MIXTURE PRODUCTION

The goals for the mixtures produced for the construction of a Perpetual Pavement are established during the mixture design and according to the desired uniformity during production and construction. Performance in terms of durability, cracking resistance, and rutting resistance is the primary goal, as early failures will defeat the pavement's purpose and increase costs to the agency and to the road users delayed by rehabilitation activities. It is advisable to incorporate the use of performance testing during mixture design to help ensure that the pavement will perform as intended. Using the job mix formula (JMF) from the mix design as a starting point, the proportions of aggregate and binder may need to be adjusted to account for changes in materials that occur during production. However, these adjustments should be made only after testing the initial run at the plant. Once field adjustments have been made to the JMF,

Figure 4-5. Conical Aggregate Stockpile

the producer should strive to maintain consistency in the materials and plant operations for each type of mixture being manufactured for the project.

Aggregate stockpiles should be built to minimize the segregation of aggregate particles and the amount of moisture in the material going into the plant. Segregation in the final asphalt mixture will negatively affect the consistency of the volumetric parameters, as will fluctuations in the aggregate moisture content. Among practices that help avoid segregation is the separation of individual aggregate sizes into different stockpiles. Aggregates, if they are deposited by stacking conveyors, should be piled in flat lifts or in conical piles (shown in Figure 4-5) depending upon the amount of room available. If they are deposited by trucks they should be added to the wet side of the pile and taken from the dry side. This will give the aggregate time to partially dry before being fed into the plant. Some asphalt mixture plants have covered stockpiles to keep aggregates as dry as possible. For uncovered stockpiles the aggregate moisture content should be checked at least four times per day. As loaders retrieve aggregates from the stockpile to place into the cold feed bins, they should approach the stockpile face horizontally and then scoop up the material. The number of cold feed bins is determined by the range of sizes of the aggregate being used. It is not unusual to have six or more bins, especially for mixtures such as SMA. On the other hand, an OGFC mix may only have one bin. AASHTO and NAPA have jointly published best practices to avoid segregation (AASHTO, 1997).

Likewise, reclaimed asphalt pavement (RAP) stockpiles should be constructed with care to minimize the variability in the resulting asphalt mixture. In some instances, RAP is segregated by its source (e.g., high-volume roads, low-volume roads, etc.) This helps to maintain the consistency of the RAP binder and aggregate type. If the space at the plant does not allow for extra stockpiles, then the RAP should be crushed and blended to maximize its uniformity. Fractionation of RAP by dividing it into stockpiles of different sizes (Figure 4-6) can be useful in maximizing its usefulness. Fine RAP has a strong affinity for moisture, and this may have an impact on the variability and heating of the material. The moisture content of a RAP stockpile should be monitored throughout the day. In some cases, contractors have covered fine RAP materials to keep them as dry as possible. More information on RAP handling at asphalt mixture plants is given elsewhere (Young, 2007).

Figure 4-6. Fractionation of RAP

mixture usually begins hours ahead of paving. Silos are charged with the asphalt mixture so that relatively minor changes in production do not cause frequent stop–start cycles. In order to avoid segregation problems, silos should have baffles that prevent long free-fall distances for the mix. An asphalt mixture can be successfully stored for up to two days or more depending upon the silo design, insulation, cone heating system, and the amount of air in the head space. Because extended silo storage can affect mixture quality, after longer periods of storage, the first load discharged should be checked for temperature and the material tested to ensure it meets the volumetric criteria of the JMF.

As stated previously, the production of the asphalt mixtures should be focused on getting the right proportions consistently. The plant should begin manufacturing the asphalt mixtures hours ahead of paving so that the paving train has a constant amount of material flowing to the job site over the day's work hours. There is a temptation to fill the dump trucks rapidly in order to get them out of the gate at the beginning of production, but this is likely to result in the trucks arriving at the job site and having to wait a substantial amount of time until they can deposit their loads either on the roadway or in the paving hopper or material transfer device. Also, having trucks backed up at the construction site can result in work zone congestion, creating problems for road users and other construction activities. It is better to have trucks spaced evenly apart so that there are no more than about three trucks at the job site at a time. Balancing the plant production and paving operation is covered in NAPA Publication IS-120, *Balancing Production Rates in Hot Mix Asphalt Operations* (Warren, 1996). The key to achieving this balance is excellent communication between the plant and paving site concerning the number of trucks waiting to be filled at the plant, the number of trucks at the construction site, and the number in transit. As stoppages or other significant changes occur at either end, the other site should be contacted with information that will allow for a shift in operations.

In addition to balancing production rates, the use of best practices for storage and transporting asphalt mixtures is crucial to maintaining the consistency needed to avoid early distress. Production of the

When filling dump trucks, the asphalt mixture should be loaded in multiple drops to inhibit aggregate segregation. The techniques for accomplishing are outlined in AASHTO–NAPA publication QIP-110, *Segregation Causes and Cures for HMA* (AASHTO, 1997). From a safety perspective, it is always advisable to cover truck beds in transit with tarps; this will also help reduce the loss of heat in the material. If the material is not covered, mixture cooling will occur more rapidly, especially in colder weather and over longer distances. This may cause a crust to form over the top and sides of the mixture. During discharge, this crust will show up as cold spots in the mat (usually at the end of the load), which are difficult to compact. The use of a remixer will help alleviate this problem, sometimes referred to as "temperature segregation."

The quality and consistency of the produced material is determined by the results of testing in the quality assurance process. Quality control is a monitoring process used by the contractor to establish quality benchmarks as set by the specifications and JMF and to track the consistency of the produced mixture. Once the quality level has been established, the contractor then uses periodic testing to track the consistency of production and to make decisions about adjustments. Quality assurance (QA) is testing performed by the agency as either a check on the contractor's results for payment based on the QC results or as an independent assessment to determine payment based upon QA results (TRB, 2018). It is important to have a properly structured and statistically based QA system to provide the greatest confidence in both the quality and consistency of the pavement being constructed.

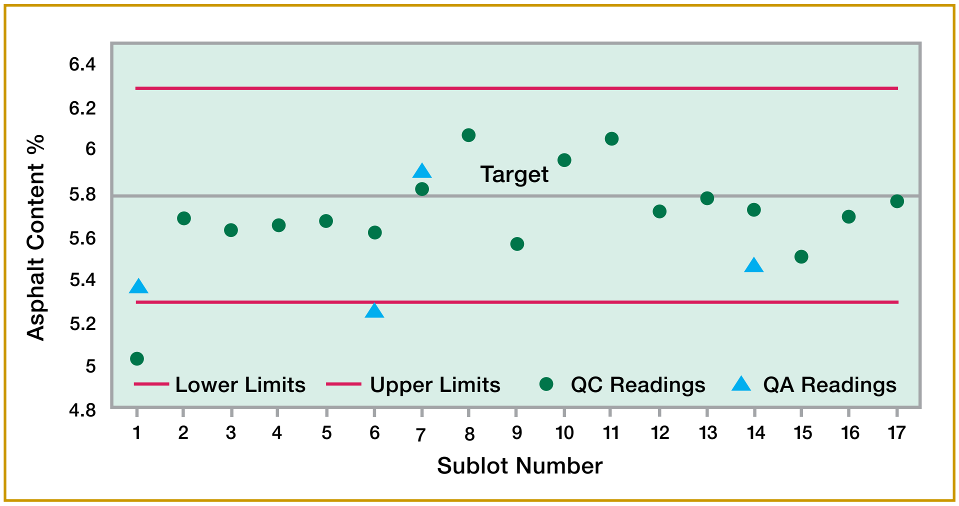

Figure 4-7. Example of a Control Chart Used in QC/QA

An example of a control chart showing how asphalt content on one project varied with time is presented in Figure 4-7. The black dots are the contractor's QC results and the blue triangles are the agency's QA results. It shows that the asphalt content at the beginning of the project started below the lower end of acceptable limits established by the JMF. This result for Lot 1 alerted the contractor to change the process and increase the asphalt content. As time progressed the asphalt content varied about the target while remaining within the upper and lower limits. The QA values show that the agency's testing was within a margin of error of the contractor's results in two of the four cases, which may indicate some problem at the beginning of the project.

QC testing at the asphalt plant normally includes the aggregate gradation and volumetric properties of the asphalt mixture after compaction in a laboratory. QA testing is normally conducted at the agency's or agency's representative's laboratory and usually within a specified time frame. The location for taking plant QC/QA samples is important to ensure the uniformity of the test results and to avoid possible bias. In most instances, these samples are taken from the bed of the dump trucks, placed in buckets, and then compacted at the plant laboratory.

The asphalt mixing plant operation is important to manufacturing a high-quality mixture that is consistent. Well-run plant operations ensure that all precautions are taken to avoid aggregate segregation and cold lumps of material from being produced and delivered to the paving operation. Material should be fed to the paving operation by balancing the rate of production to the rate of paving. QC/QA testing at the plant is crucial to ensuring the quality and consistency of the mixture. While good plant operations are important to the success of the Perpetual Pavement construction, the final, critical point is the asphalt placement and compaction.

ASPHALT PAVING AND COMPACTION

Paving and compaction operations provide the final opportunity to ensure that a Perpetual Pavement has the qualities needed to provide a long service life. Paving conditions, weather, traffic, mixture production, trucking, paver operations, compaction, and QC must all be integrated to ensure a high-quality roadway. While there is never a time when all these issues are ideal, understanding their impact and having contingency plans will help overcome problems.

Asphalt paving must begin with an understanding of the site conditions. Once all the personnel and equipment are in place, an assessment of potential problems, including the weather, should be made. The air temperature and the temperature of the surface to be paved should be measured. The wind speed should be obtained from a weather station or weather app. Using this information in a tool such as the MultiCool software or mobile app (available from www.asphaltpavement.org/MultiCool) will help alert

crews to potential problems with mixture cooling, especially in late season paving.

MultiCool uses user-friendly inputs such as air temperature, wind speed, mix temperature, lift thickness, and paving surface temperature to calculate the time for the mat to cool to a specified temperature. The output guides the paving crew to how closely the compactor should follow the paver. Figure 4-8(a) and 4-8(b) illustrate the importance of ambient temperature, wind speed, and lift thickness. From the graphs, thicker lifts of asphalt (≈2.5 in.) can be paved in a wide spectrum of conditions with plenty of margin to get the mat compacted. As lift thickness decreases from 1.0 to 1.5 inches. the compaction window shrinks considerably for both ambient temperature and wind speed. It should also be noted that wind speed can be just as important as cooling temperatures in terms of the amount of time to achieve compaction. For instance, in Figure 4-8(a), even when the ambient temperature is 50°F and the wind speed is minimal, there is only about a 10-minute period in which to compact a 1-inch lift.

It is also important to follow best practices regarding bonding between pavement layers. A successful bond between pavement layers is critical to long-term performance

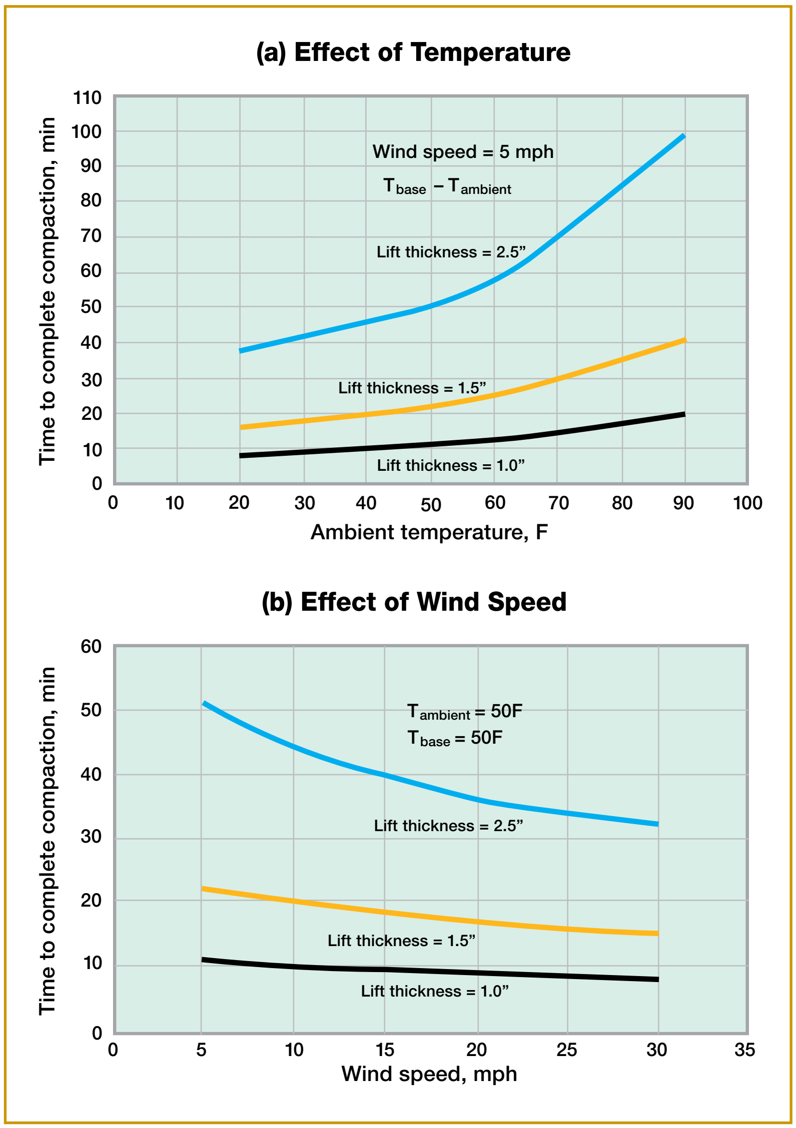

Figure 4-8. Impact of Environmental Factors on Asphalt Pavement Cooling

as insufficient bonding can lead to delamination and slippage cracking. Tack coats are commonly used to properly bond new mixes to the underlying pavement. The key factors that need to be considered to ensure successful tack coat application are (1) condition of the existing pavement, (2) tack coat application rate, (3) residual binder content, (4) proper distribution operation, and (5) emulsion break or set times (Decker, 2013).

As discussed previously, it is important to keep a smooth flow of material from the plant to the paving site. This is done by balancing the production to the requirements for material at the paving site. It is undesirable to have a paver stop and start and wait an excessive amount of time between loads. This can result in building roughness into the final mat due to the acceleration and deceleration of the paver. Cold spots can form under the paver as it sits between loads, and this can cause weak spots due to poor mat density. To the greatest extent possible, the paver speed should be slowed so that a continuous head of material flows to the augers and screed. This will also allow the compactors to keep up with the paver and provide more time for them to accomplish a full coverage of the pavement.

Avoiding segregation at the paving site is as important as it is at the plant. There are several opportunities to ensure the uniformity of the mixture during paving. For instance, if the mixture is being delivered to the job site in an end dump truck, the truck should be aligned with the paver, the bed is raised until the load “breaks,” and then the gate is opened to allow the material to flow into the paver hopper. This requires good coordination and skill on the part of the truck driver and the crew member guiding the truck. It is also important to note whether any type of crust developed on the surface of the mixture so that tools, such as heavy tarps, can be employed to help avoid this in the future.

Live bottom-dump trucks provide a smooth flow of material from the truck bed to the paving hopper, but if the material at the top of the load is cold, it could cause cold spots in the finished mat. Windrows formed from bottom-dump trucks are subject to slight remixing from the pickup machine feeding material into the paver, but crews need to ensure that the placement of the windrow does not get too far ahead of the paving operation, as this may result in the material cooling too much. By far, it is best to reblend the material in a remix machine ahead of feeding the material into the paver. This will improve the consistency of the mixture temperature and help mitigate any aggregate segregation that may have happened. However, if the remixing machine is run dry in between loads, it can also cause segregation.

The asphalt paver is the last point in the paving process where segregation may occur. Material is fed into the paver hopper, carried from there to the back of the paver by a conveyor or screw-feed system, which deposits the material just ahead of the augers. The augers move the material across the width of the paving pass, and from there the material is smoothed under the screed. As material is placed into the hopper, a bin with wings on each side, some of it builds up on the wings. Often, a paver crew will periodically fold the wings to clear any excess material. This dumping of the wings often causes a cold mass of material to move through the paver and create a cold spot in the pavement. Many contractors have elected to use a hopper insert or to stop dumping the wings in order to improve quality.

The next point of interest is the auger, as the mix is fed by the conveyor through gates that control the flow. Some paver augers have a gear box that sits in the middle of the paver. In the past, this gear box caused a streak of segregation in the middle of the paver pass that sometimes resulted in longitudinal cracking. Paving manufacturers have developed counter-measures, such as placing a reverse auger or paddle to more evenly distribute the material. The flow of material should be such that the augers should be about halfway covered during the paver movement. If not enough material is fed to the auger, segregated material may pass through and wind up in pockets or streaks across the mat. If too much material is fed to the augers, then it may not be evenly distributed under the screed and the surface will be rough. Also, in instances where a wider paving width is necessary, the auger gates are moved toward the outside of the machine to allow material to flow to the full width. However, whenever this is done, auger extensions must be added to avoid segregation and streaking on the outside of the paving pass.

Compaction is the final opportunity to achieve quality in the constructed roadway and is often the determining factor in whether the pavement will achieve a truly long life. Over the past 20 years, Intelligent Compaction technologies have been advanced for monitoring the conditions for compaction and tracking the progress in obtaining the desired density. Infrared sensors and cameras have been very useful in detecting temperature patterns that may be used to alert construction crews to potential compaction problems. As a result, crews have been able to resolve problems proactively rather than waiting to receive the results of QC testing.

To coincide with this development, more contractors are using GPS or other surveying techniques to track the coverage of compaction equipment to ensure that the full-width is being compacted evenly. The combination and order of compactors to be used in construction and the number of passes each makes on the mat depend upon the mixture type, lift thickness, and the compaction characteristics. In most cases, it is common to use a vibratory compactor for initial density, another vibratory or pneumatic tire roller in the intermediate position, and a static roller to finish the compaction. As stated above, the right combination and number of passes are very particular to a given lift and mix type. The on-site QC technicians should assist in determining the roller pattern that will provide the greatest possible density.

As with plant production, QC/QA testing is very important to the longevity of the finished pavement. Beyond helping to establish the roller pattern, the construction site QC team should monitor the in-place density of the mat through the use of nondestructive density gauges and/or cores, the temperature of the material at the beginning and ending of compaction, and the thickness of the compacted lift. Nondestructive measurements should be taken with a gauge that has been calibrated and is correlated to cores taken from that mixture. As the results are gathered, any excessive variability in results or trends should be noted and discussed with the paving superintendent. Some variability in results is expected, so it is suggested that decisions on mix or operational changes be made based on at least three readings.

Finally, trouble-shooting should begin before paving. While it is impossible to have a plan for every situation, a list of common problems encountered should be developed along with possible causes, methods of investigation, and corrective actions to be taken. While the design of Perpetual Pavements is important, the construction of the structure will often dictate whether it meets the goal of being “perpetual.”

5 CONCLUDING THOUGHTS

Long-life asphalt Perpetual Pavements have been designed and constructed for decades as full-depth and deep-strength asphalt pavements. Recently, design procedures, classified as Perpetual Pavement design, 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. These materials must be selected according to the role they play in enhancing pavement performance through their mechanistic properties. Good construction practices are of paramount importance to the performance of asphalt pavements.

Going forward, the mechanistic-empirical design process will be the format for the design of Perpetual Pavements. Transfer functions that describe pavement performance need to allow for the input of limiting strains so that pavements can be designed to account for instances where pavement responses are below the point where damage occurs. Existing design procedures for Perpetual Pavements encompass a variety of applications, including high- and low-volume pavements, high-modulus pavements, and the rehabilitation of flexible and rigid pavements.

The knowledge and research exist to create a pavement structure that 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 serve. This includes a rut- and wear-resistant upper layer of HMA. 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, durable intermediate layer should be constructed from a dense-graded asphalt pavement. Finally, the base layer of the asphalt pavement needs to be a fatigue resistant, durable layer that is easy to compact. This final 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 process.

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 more than 130 examples of long-lasting asphalt pavements ranging from major airports and interstate highways to low-volume roads and municipal streets. The performances of existing pavements in many states have 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 around 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 asphalt pavements.

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NATIONAL ASPHALT
PAVEMENT ASSOCIATION

6406 Ivy Lane, Suite 350, Greenbelt, MD 20770-1441

www.AsphaltPavement.org

NAPA@AsphaltPavement.org

Tel: 301-731-4748 | Fax: 301-731-4621 | Toll Free: 1-888-468-6499

@NAPATweets

at National Asphalt Pavement Association

at National Asphalt Pavement Association

at National Asphalt Pavement Association

@goasphalt