HMEC

Highway Materials Engineering Course

PARTICIPANT WORKBOOK

Asphalt Materials and Paving Mixtures

MODULE

F

U.S.Department of Transportation
Federal Highway Administration

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About This Workbook

This workbook has been developed as a resource for participants. This workbook can be used during the training session to follow along with the instructor and take notes, as well as for reference after the module has ended.

Course Overview

The Federal Highway Administration (FHWA) Highway Materials Engineering Course (HMEC) is a comprehensive multi-week training event that consists of eight content “modules” that provide students with the knowledge to develop materials specifications and guidance, make effective acceptance decisions, and design, construct, and maintain assets with a long service life. Modules range in duration for the number of days they take to complete. The modules are:

- Module A: Quality Assurance
- Module B: Soils and Foundations
- Module C: Steel, Welding, and Coatings
- Module D: Aggregates for Transportation Construction Projects
- Module E: Mechanistic Empirical Pavement Design Guide
- Module F: Asphalt Materials and Paving Mixtures
- Module G: Portland Cement Concrete
- Module H: Evaluating Recycled Materials for Beneficial Uses in Transportation

Introduction

Module F: Asphalt Materials and Paving Mixtures is the sixth module in the FHWA HMEC.

This module includes detailed technical content. In addition to that content, it provides numerous opportunities to apply the content; experiences such as laboratory testing, analysis, mixture design, and other identified opportunities are included.

Module F Overview

Below is a visual overview of all of the lessons covered in this module:

Web-based Training (WBT)

- 1 Introduction and Overview

Instructor-led Training (ILT)

- 2 Asphalt Binders
- Laboratory Experience: Asphalt Binder PG Tests
- 3 Asphalt Binder Test Methods & Specifications
- 4 Weight-volume Relationships Used in Asphalt Concrete Mixtures
- 5 Hot Mix Asphalt Mixtures and Design Concepts
- Laboratory Experience: Mixture Design
- 6 Performance Tests for Asphalt Mixtures
- Laboratory Experience: Performance Testing
- 7 Production, Construction, and Acceptance of Asphalt Pavements
- 8 Preservation, Rehabilitation, and Recycling of HMA Pavements
- 9 Hot Topics
- **Review and Final Assessment**

Module Goals

The goals for this module are as follows:

- Describe the characteristics and engineering properties of asphalt mixtures and their effects when utilized in highway applications
- Identify engineering properties for related mix performance
- Explain the significance of common field and laboratory testing of asphalt mixtures
- Interpret test results to predict ultimate performance of asphalt mixtures
- Evaluate a series of field and laboratory test results to determine whether an asphalt mixture or process is within acceptable tolerances
- Determine appropriate test methods and frequency for asphalt mixture materials sampling, quality control testing, and quality assurance testing to support the development of an effective acceptance program
- Explain how construction operations can affect the ultimate performance of asphalt mixture pavements
- Discuss current best practices, potential issues, technology, and trends that may affect traditional asphalt pavements and pavement preservation methods

Learning Outcomes

Lesson 1: Introduction and Overview

- LO 1.1: Recall some of the more important technologies, equipment, and procedures that have been implemented regarding asphalt pavements
- LO 1.2: Explain why specific tests, equipment, and procedures have changed over time
- LO 1.3: Describe asphalt mixture types and uses
- LO 1.4: Identify types of asphalt pavement distress
- LO 1.5: Differentiate between structural and functional pavement performance
- LO 1.6: Explain the importance of tying mixture design to structural design and quality assurance

Lesson 2: Asphalt Binders

- LO 2.1: Describe asphalt binders and how the material response changes with temperature and loading frequency during service life
- LO 2.2: Describe why additives, modifiers, and extenders are used in asphalt binders
- LO 2.3: Determine the appropriate choice of binder based on traffic loads, highway application use, and service climate

Lesson 3: Asphalt Binder Test Methods and Specifications

- LO 3.1: Explain how laboratory aging should correlate to performance
- LO 3.2: Describe the rheological properties of asphalt binders
- LO 3.3 Describe non-rheological asphalt binder tests and their importance
- LO 3.4: Use laboratory data to grade an asphalt binder according to the Superpave system
- LO 3.5: Understand the chemical and physical hazards associated with asphalt binder laboratory testing
- LO 3.6: Recognize the equipment and materials required to run each test and learn how to perform each test in a laboratory environment

Lesson 4: Weight-volume Relationships Used in Asphalt Concrete Mixtures

- LO 4.1: Understand the relationship between the weight and volume of an asphalt mixture
- LO 4.2: Calculate volumetric properties

Lesson 5: Hot Mix Asphalt Mixtures and Design Concepts

- LO 5.1: Identify the mixture characteristics required to produce a long-lasting asphalt mixture
- LO 5.2: Define the relationship between aggregate properties and properties of an asphalt mixture
- LO 5.3: Understand the philosophy behind the mix design process
- LO 5.4: Identify the major steps of the Superpave mixture design procedure
- LO 5.5: Compare the mixture designs for a dense-graded mixture, an open-graded friction course, and a gap-graded mixture
- LO 5.6: Identify reclaimed and recycled components that can be added to an asphalt mixture and their effects on the mixture's engineering properties
- LO 5.7: Identify specification limits for the types of asphalt mixtures
- LO 5.8: Recognize the equipment and materials required to run each test and learn how to perform each test in a laboratory environment

Lesson 6: Performance Tests for Asphalt Mixtures

- LO 6.1: Differentiate between fundamental and empirical tests
- LO 6.2: Explain stiffness or modulus testing and how the results are used.
- LO 6.3: Describe plastic deformation testing and how the results are used.
- LO 6.4: Describe durability and moisture damage testing and how the results are used.
- LO 6.5: Explain load related fracture testing and how the results are used.
- LO 6.6: Explain non-load related fracture testing and how the results are used.
- LO 6.7: Recognize the equipment and materials required to run each test and learn how to perform each test in a laboratory environment
- LO 6.8: Identify potential adjustments to meet deficiencies identified by performance testing

Lesson 7: Production, Construction, and Acceptance of Asphalt Pavements

- LO 7.1: Describe basic asphalt mixture plants and their components
- LO 7.2: Analyze production testing results to determine needed corrective measures during production
- LO 7.3: Recognize different asphalt mixture transportation methods and proper handling procedures
- LO 7.4: Discuss specifications for asphalt mixtures and mix designs and their impact on production facilities
- LO 7.5: Recognize proper handling procedures for incorporating recycled asphalt pavement (RAP) into an asphalt mixture
- LO 7.6: Recognize proper handling procedures for incorporating recycled asphalt shingle (RAS) into an asphalt mixture
- LO 7.7: Explain the consequences of changes made to the operation of an asphalt plant during production
- LO 7.8: Describe best operational practices for conventional asphalt mixture laydown, compaction equipment, and procedures
- LO 7.9: Describe the quality assurance elements
- LO 7.10: Describe methods of reducing variability in construction processes
- LO 7.11: Identify quality characteristics that are used for acceptance and demonstrate their use and application

Lesson 8: Preservation, Rehabilitation, and Recycling of Asphalt Mixture Pavements

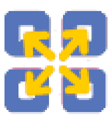
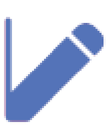
- LO 8.1: Identify appropriate repair strategies, based on the condition of the existing pavement.
- LO 8.2: Identify the different types of preservation and repair strategies for functional improvements and the conditions under which they should be applied.
- LO 8.3: Identify the different types of repair strategies for structural improvements and the conditions under which they should be applied.
- LO 8.4: Identify the different types of in-place recycling methods and the conditions for which in-place recycling methods should be applied.

Lesson 9: Hot Topics

- LO 9.1: Summarize some of major issues facing the asphalt industry currently and in the future
- LO 9.2: Describe some of the current research topics in areas of asphalt concrete mixture design and pavement performance areas
- LO 9.3: Describe some of the research products an agency will need to consider and evaluate for future implementation to improve performance and reduce overall costs of an asphalt pavement
- LO 9.4: Identify some of the future changes that are expected in asphalt concrete design, production, and construction equipment

ILT Instruction Icons

The following icons are used on the slides as a cue to the instructor and participants:

Icon	Icon Name	Typical Use
	Timer	▪ Call out the estimated time for the lesson
	Important Information	▪ Call out important information.
	Q & A	▪ Check for understanding or agreement. ▪ Survey participants. ▪ Solicit feedback.
	Breakout/Small Group Exercise	▪ Break participants into groups. ▪ Provide directions for exercise.
	Video/Sound	▪ Show a video.
	Reference	▪ Reference another document or resource.
	Links	▪ Share a Web link for additional resources.
	Whiteboard	▪ Draw or document something on a whiteboard or easel pad.

Icon	Icon Name	Typical Use
	Safety	▪ Call out important safety information.
	Common Error	▪ Call out a system or process that is often misused.

Slide 1


ILT
HMEC
Highway Materials Engineering Course
Lesson 2: Asphalt Binders
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Asphalt Materials and Paving Mixtures
U.S.Department of Transportation
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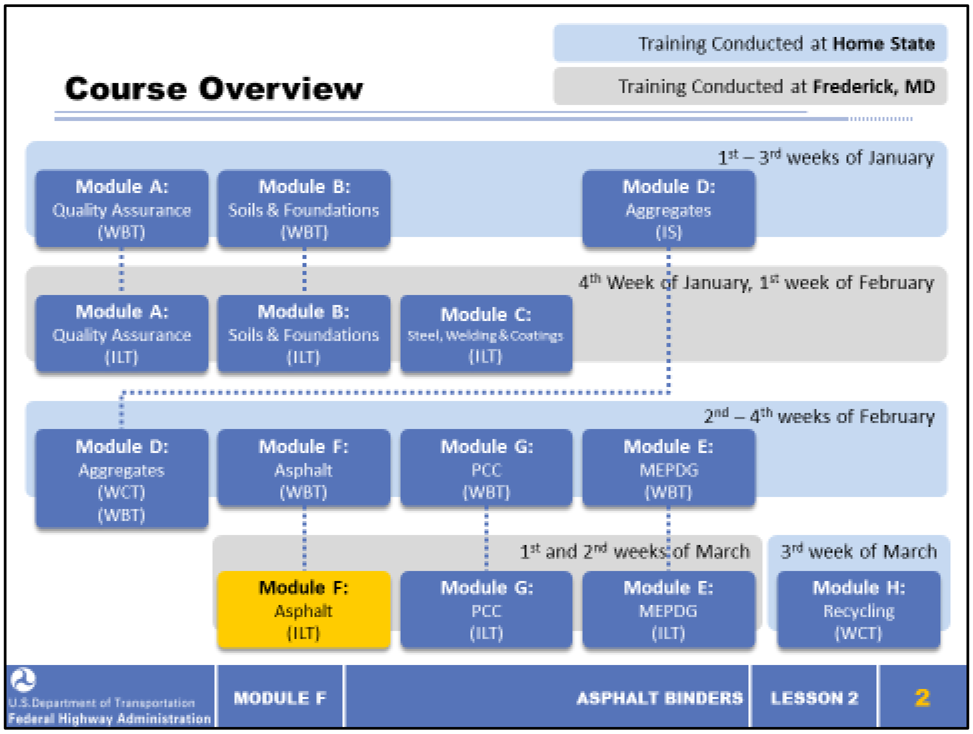
Slide 2


Training Conducted at Home State
Training Conducted at Frederick, MD
Course Overview
1st – 3rd weeks of January
Module A: Quality Assurance (WBT)
Module B: Soils & Foundations (WBT)
Module D: Aggregates (IS)
4th Week of January, 1st week of February
Module A: Quality Assurance (ILT)
Module B: Soils & Foundations (ILT)
Module C: Steel, Welding & Coatings (ILT)
2nd – 4th weeks of February
Module D: Aggregates (WCT) (WBT)
Module F: Asphalt (WBT)
Module G: PCC (WBT)
Module E: MEPDG (WBT)
1st and 2nd weeks of March
3rd week of March
Module F: Asphalt (ILT)
Module G: PCC (ILT)
Module E: MEPDG (ILT)
Module H: Recycling (WCT)
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ASPHALT BINDERS
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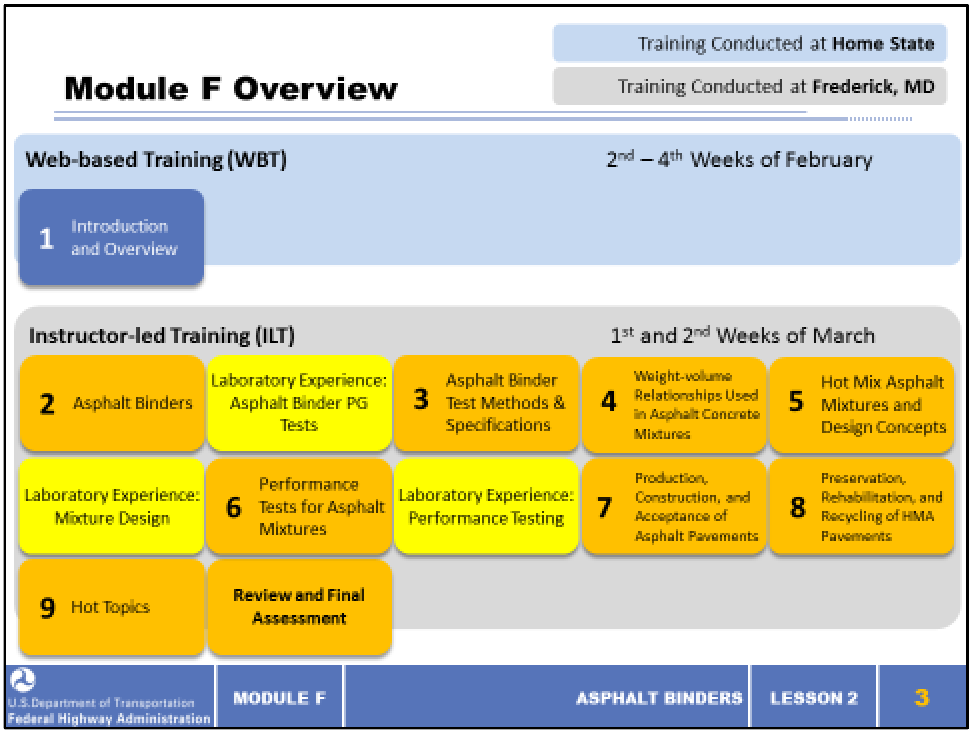
Slide 3


Training Conducted at Home State
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Review and Final Assessment
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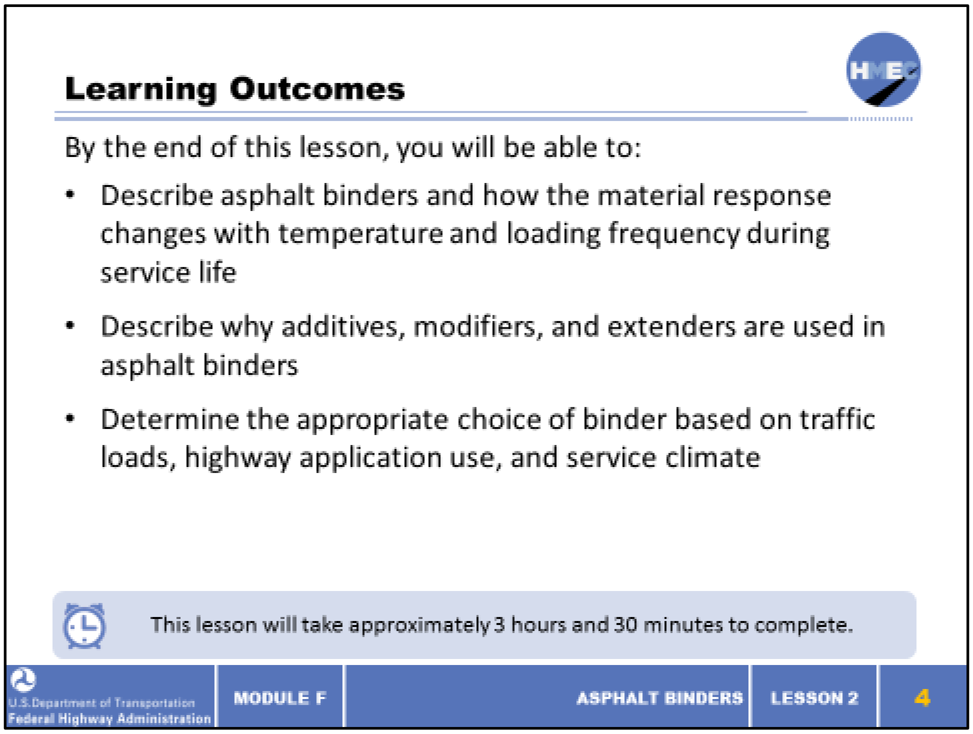
Learning Outcomes
HMEC
By the end of this lesson, you will be able to:
• Describe asphalt binders and how the material response changes with temperature and loading frequency during service life
• Describe why additives, modifiers, and extenders are used in asphalt binders
• Determine the appropriate choice of binder based on traffic loads, highway application use, and service climate
This lesson will take approximately 3 hours and 30 minutes to complete.
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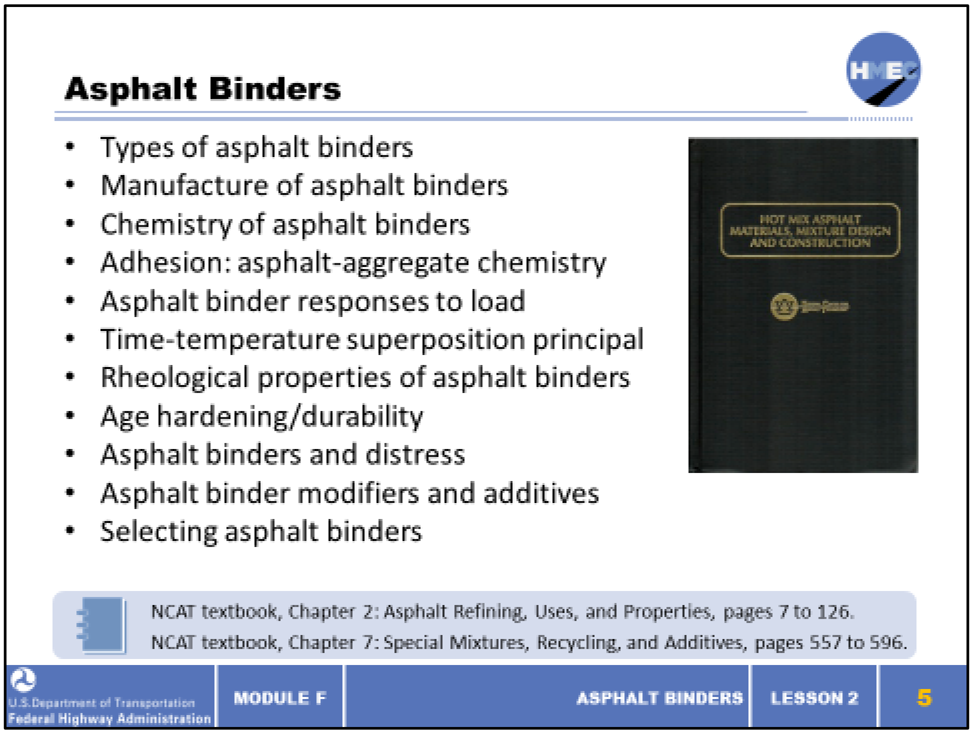
Asphalt Binders
HMEC
Types of asphalt binders
Manufacture of asphalt binders
Chemistry of asphalt binders
Adhesion: asphalt-aggregate chemistry
Asphalt binder responses to load
Time-temperature superposition principal
Rheological properties of asphalt binders
Age hardening/durability
Asphalt binders and distress
Asphalt binder modifiers and additives
Selecting asphalt binders
HOT MIX ASPHALT MATERIALS, MIXTURE DESIGN AND CONSTRUCTION
NCAT textbook, Chapter 2: Asphalt Refining, Uses, and Properties, pages 7 to 126.
NCAT textbook, Chapter 7: Special Mixtures, Recycling, and Additives, pages 557 to 596.
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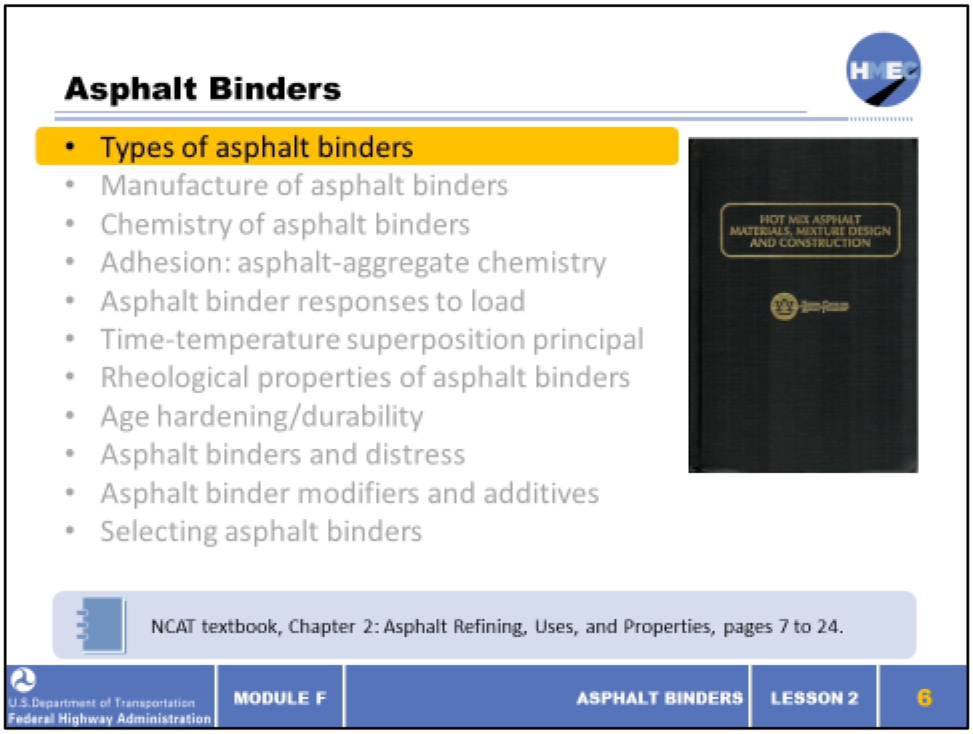
Asphalt Binders
HMEC
• Types of asphalt binders
• Manufacture of asphalt binders
• Chemistry of asphalt binders
• Adhesion: asphalt-aggregate chemistry
• Asphalt binder responses to load
• Time-temperature superposition principal
• Rheological properties of asphalt binders
• Age hardening/durability
• Asphalt binders and distress
• Asphalt binder modifiers and additives
• Selecting asphalt binders
HOT MIX ASPHALT MATERIALS, MIXTURE DESIGN AND CONSTRUCTION
NCAT textbook, Chapter 2: Asphalt Refining, Uses, and Properties, pages 7 to 24.
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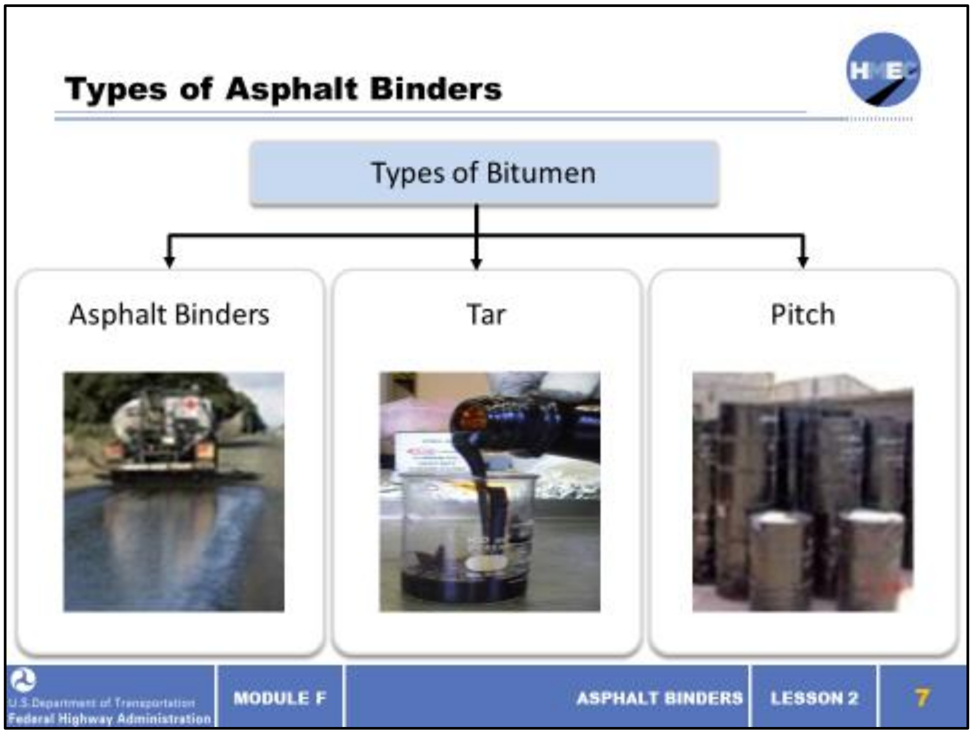

The American Society for Testing and Materials (ASTM) definitions for bituminous materials are given as follows:

• Asphalt – A dark brown to black cementitious material in which the predominant constituents are bitumens that occur in nature or are obtained in petroleum processing. Asphaltites are not defined by ASTM but are very hard bitumens usually found in vein deposits. The gilsonite deposits in Utah and Colorado are examples of asphaltites.
• Tar – Brown or black bituminous material, liquid or semi-liquid in consistency, in which the predominate constituents are bitumens obtained as condensates in the destructive distillation of coal, petroleum, oil-shale, wood, or other organic materials, and which yields substantial quantities of pitch when distilled. Tars are not used in road building anymore. Tar is especially used where jet fuel and gasoline resistance is required. Slurry seals are also sometimes constructed using tar. Both uses are diminishing. Tars are used in airport applications, and auto parking and fueling areas. Most often tars are used as binders in hot mixes and slurry seals. The cost of tar binders is about twice that of asphalt. Asphaltites have been diluted with petroleum cutter stock and used for sealers on a limited scale.
• Pitch – Black or dark-brown solid cementitious materials that gradually liquefy when heated and which are obtained as residue in the partial evaporation or fractional distillation of tar.

Asphalt binders are specifically those bituminous materials that are used predominantly in paving (and roofing) applications. Tars are sometimes used in paving where resistance to jet fuel or gasoline is desired. Pitch is never used in paving.

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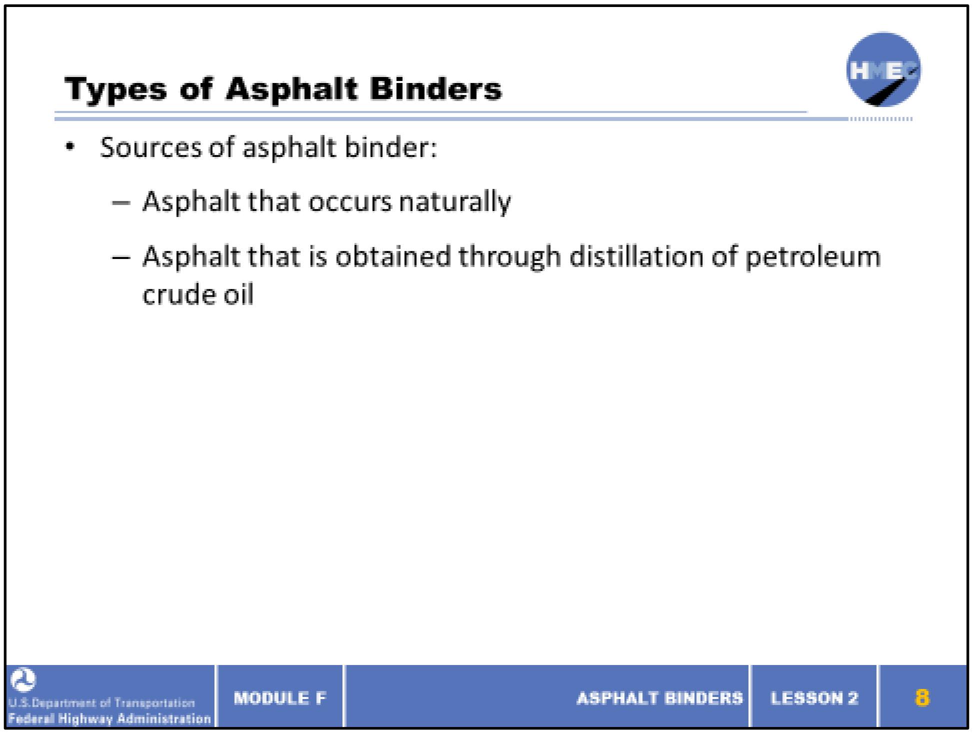

There are two sources of asphalt binders: 1. Those occurring naturally, and 2. Those obtained by the refining of petroleum. In both cases, the asphalt binder is the product of the fractional distillation of petroleum, whether over short periods as in the refinery process or longer periods as in nature.

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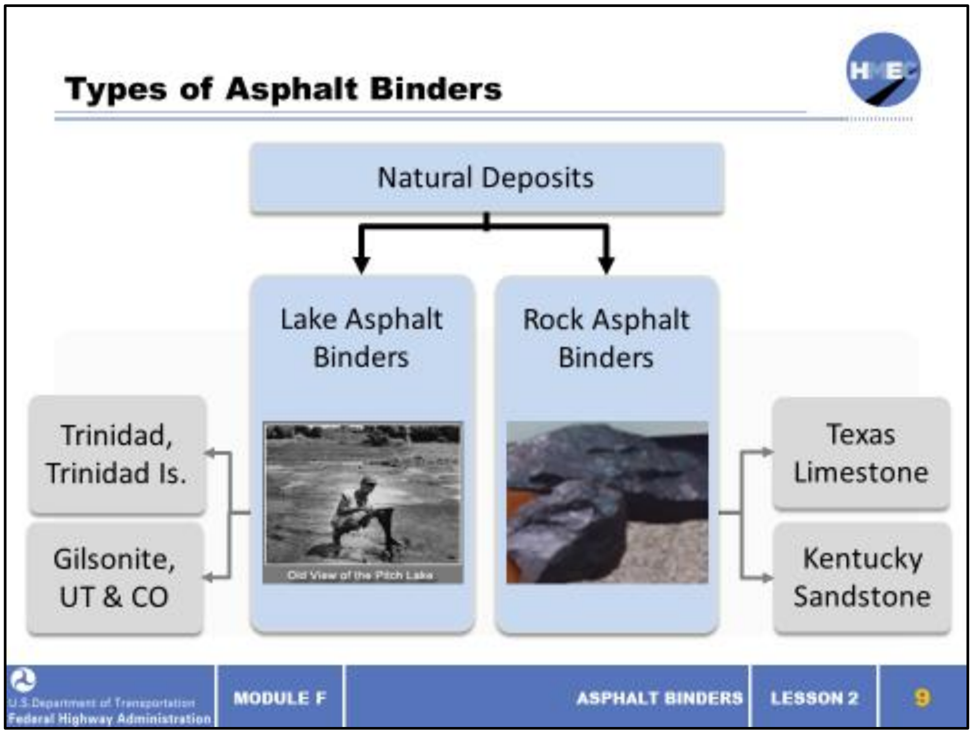

A number of naturally occurring deposits of bituminous material can be found in various parts of the world. Natural asphalts can exist either in a relatively pure form (lake) or in impregnated rock deposits. The two major sources of naturally occurring rock asphalt binders in the U.S. are in the form of rock asphalt binders: one in Kentucky and one in Texas. The more common sources of lake asphalts are termed Trinidad and Gilsonite. Gilsonite is obtained from the mines in Utah.

• Rock asphalt is a naturally occurring rock formation (usually limestone or sandstone), impregnated throughout its mass with a minor amount of asphalt binder. Rock asphalt deposits exist throughout the world and were used for road building in the 1800s. A major commercial source is located in South Texas. This limestone rock asphalt is used for cover stone in chip seals, patching materials, base course and surface courses for roadways, parking lots, and foot paths.

• The Trinidad Lake asphalt is marketed in the United States and in other countries. This material exists in a lake on Trinidad Island in an emulsified form comprised on about 40% bitumen, 30% water, and 30% mineral matter. The material is refined prior to use in road construction. Only a few thousand tons of natural asphalt binders are used per year in the United States.

Properties of several of these lake asphalts are shown below. These materials are often harder than paving grade asphalt binders. Infusible in the table below means does not blend when heated, while R&B means Ring and Ball Temperature.

Name	**Source**	**Softening Point (R&B), °F**	**Penetration at 77 °F**	**Mineral Matter**
Bermudez	Venezuela	145-160	20-30	1-7
Trinidad	Trinidad IS.	205	2-4	39
Gilsonite	Utah, Colorado	250-350	0-3	Trace
Manjak	Numerous	275-400	0	0-30
Glance Pitch Grabamite	W. Virginia, Utah	370-600	0	Trace-50
Elaterite	Europe, Asia	Infusible	0	2-7
Wurtzilite	Utah	Infusible	0	2
Albertite	Numerous	Infusible	0	Trace-15
Impaonite	Numerous	Infusible	0	Trace-15

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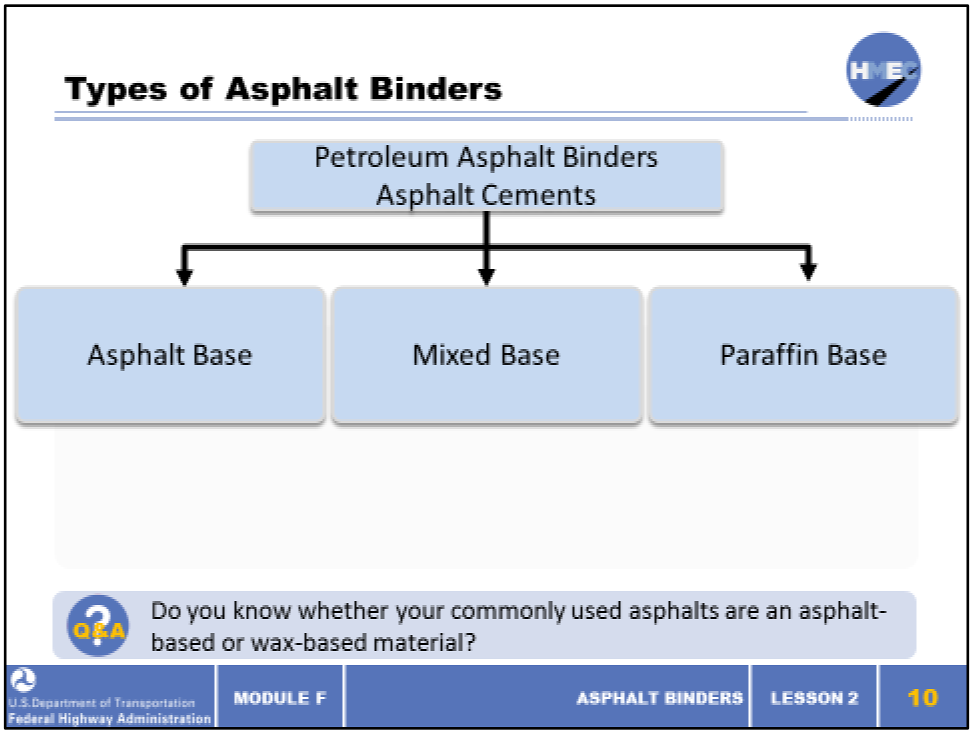

All crude oils do not contain asphalt binder as the heavier portion. In general, crudes can be classified as asphalt base, paraffin or wax base, and mixed base (asphalt and wax).

Note that the refining process for producing asphalt cements will be covered in the next topic of Lesson 2. The performance of the asphalt and the types of distresses are influenced by the crude oil and whether the asphalt cement is a wax-based or asphalt-based material.

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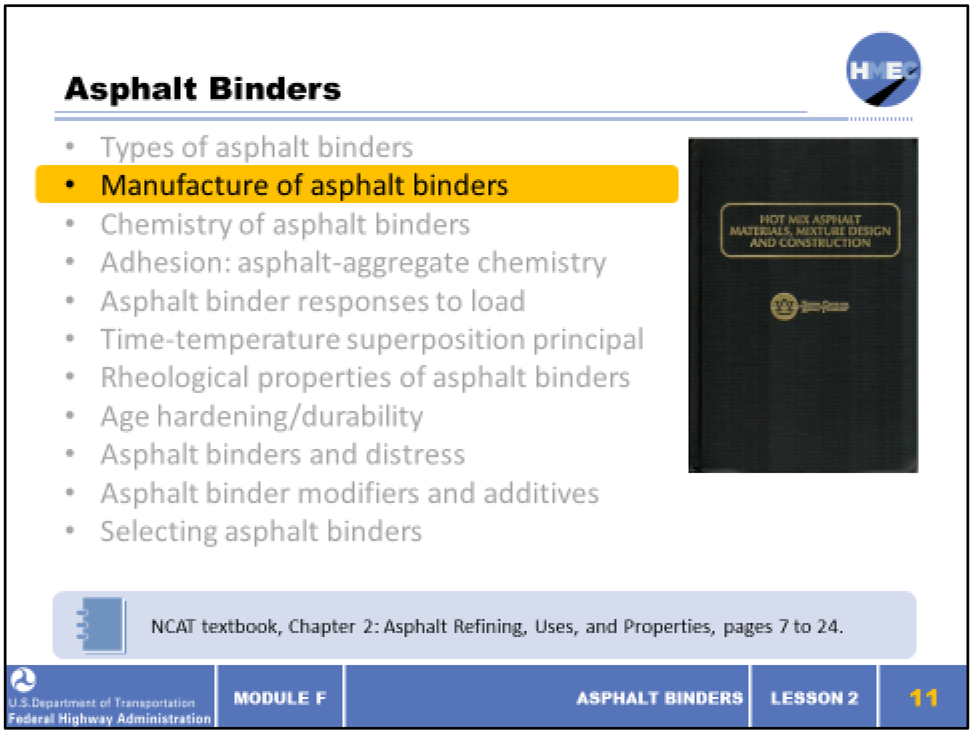
Asphalt Binders
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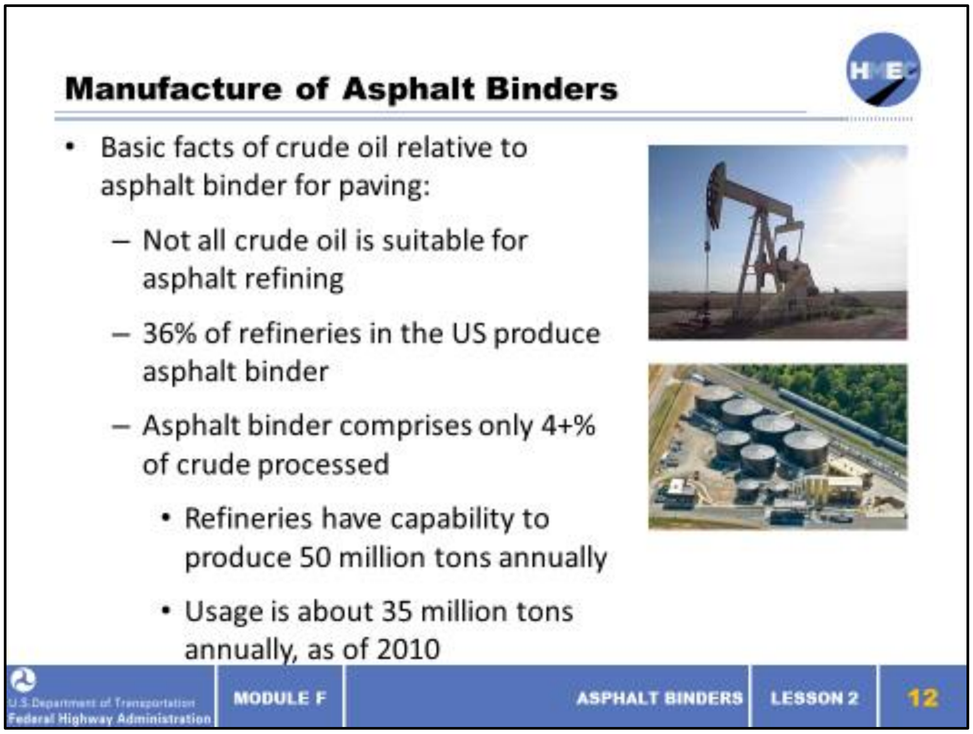

Approximately 1,450 crude oil streams exist in the world. About 975 of these crude streams are presently being used in the United States. Approximately 190 of the 975 crude streams found in the US are suitable for manufacturing paving asphalt binders. For a given region of the US (i.e., East Coast, West Coast, Gulf Coast, Mid Continent, and Rocky Mountain areas), a maximum of only 40 crude streams are economically available.

The number of crudes utilized today in US refineries is only slightly higher than the number used as many as 25 years ago. The capacity to produce asphalt binder in the US is greater than the demand.

Asphalt binder revenue must recover at least the cost of the crude. However, asphalt binder is no longer a waste product and refinery economics are controlled by the selling price of all products produced in the refinery processes.

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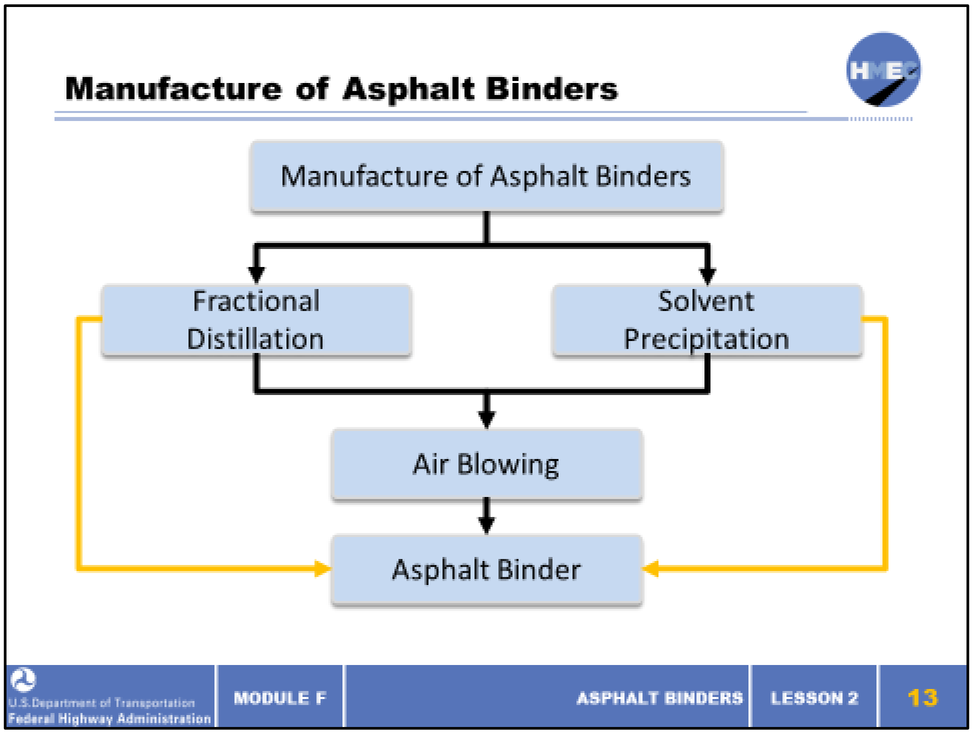

In a general sense, asphalt binders (and other products) are produced through fractional distillation of crude oil, solvent distillation of crude oil, or a combination of both processes. Residuum from these processes can be air blown to produce different grades (consistencies) of asphalt binders.

The majority of asphalt binder used for road construction is obtained by refining crude oil. As discussed earlier, asphalt properties are dependent on the crude oil source and the refining process.

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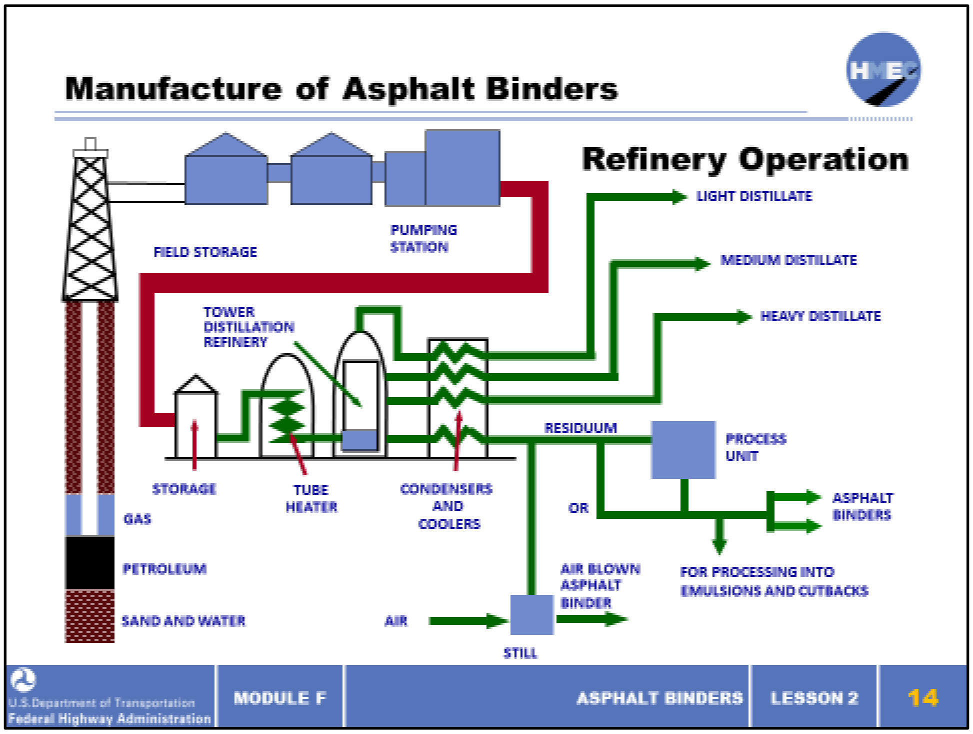

One might say that the main purpose for distilling crude oil is mainly for fuel oil (money maker), and that is considered the light distillate. The "bottom of the barrel" also has a use, and that use is producing asphalt binder (bitumen).

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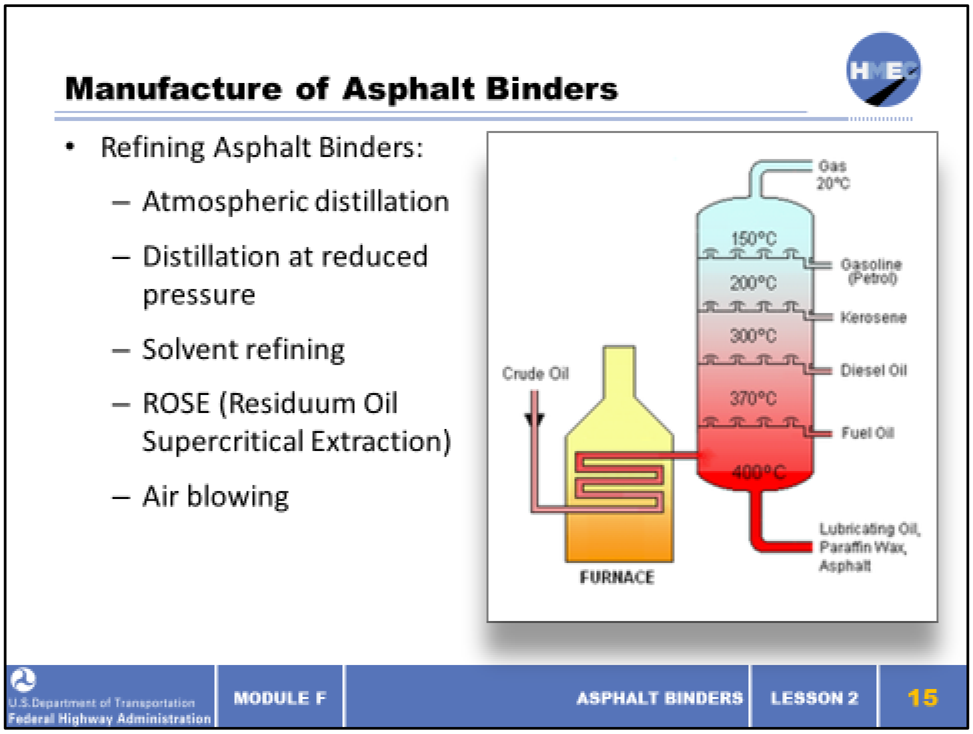
Slide 15

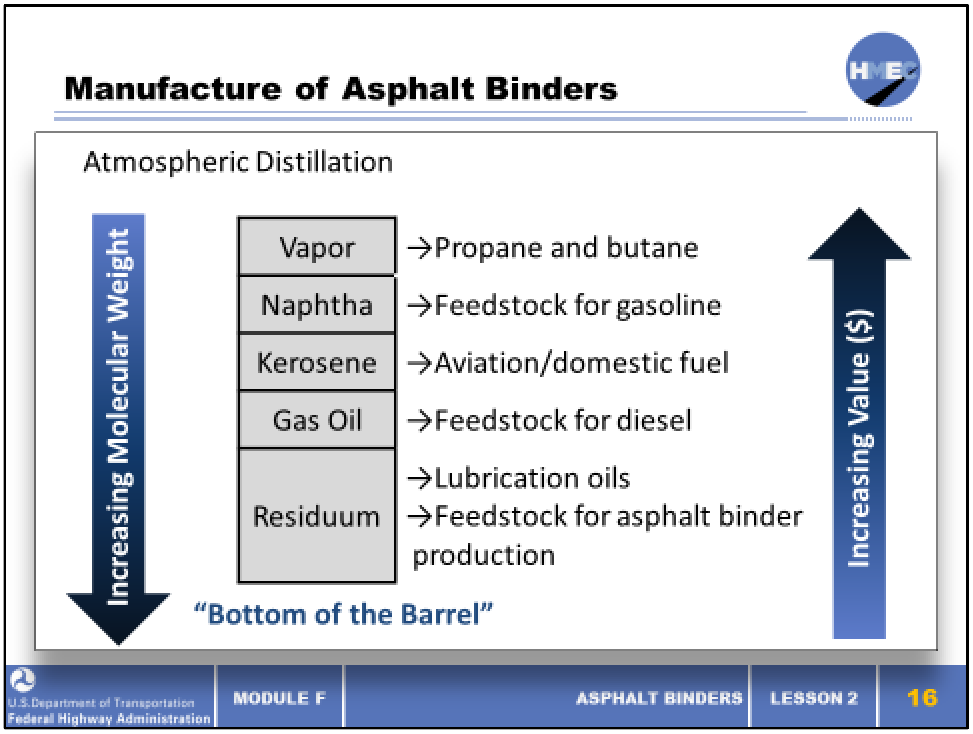


The schematic included in this slide shows a typical range of distillation temperatures to obtain different overhead fractions. It also shows a cut point 427 to 565 °C (800 to 1,050 °F) or the atmospheric equivalent vapor temperature to effect this fractionation at which the asphalt residuum is fractionated from the overhead fractions above it in the vacuum tower.

The first step in the processing of all crude petroleum is straight reduction by distillation. The distillation principle is used to separate various crude fractions which have different boiling ranges. Asphalt cement is made up of the highest boiling fractions so it becomes the residuum from the vacuum tower. The crude oil is heated in a large furnace to about 650 °C (343 °F) and partially vaporized. It is then introduced into a distillation tower where the lightest components vaporize, rise to the top, cool, condense, and are drawn off for further processing. At various heights, different fractions reach their boiling point and then condense on trays inside the tower as the temperature is reduced. The lower temperature or upper-tower components result in gasoline, while the mid-tower components are drawn off at those levels and treated to make more expensive fuels (jet fuel, kerosene, and diesel). The bottom fraction from the distillation tower is the material used to produce lubricating oil to asphalt binder. The grade of the asphalt binder is controlled by the amount of heavy gas oil removed.

The bottom fraction from the distillation tower is subjected to solvent refining or solvent deasphalting (SDA) to extract additional amount of high boiling fractions for other applications, like lube oil. The residuum oil supercritical extraction (ROSE) process provides a greater flexibility in resid fraction characteristics to separate asphaltene concentrate. The asphaltene or resin fraction from the ROSE process is used as a blending component for asphalt binders to meet selected specification requirements. Air blowing is used when the viscosity of vacuum resids must be increased; such as for roofing asphalts. Air blowing involving the pumping of the vacuum resid through a heater and air is injected at the bottom of the resid and flows upward through the resid.

Slide 16



These products typically include propane and butane (from the vapor at the top of the distillation tower), naphtha that is used to produce gasoline, kerosene, gas oil that is used to produce diesel, and the residuum (the "bottom of the barrel") that is used to produce lubricating oils and asphalt binder.

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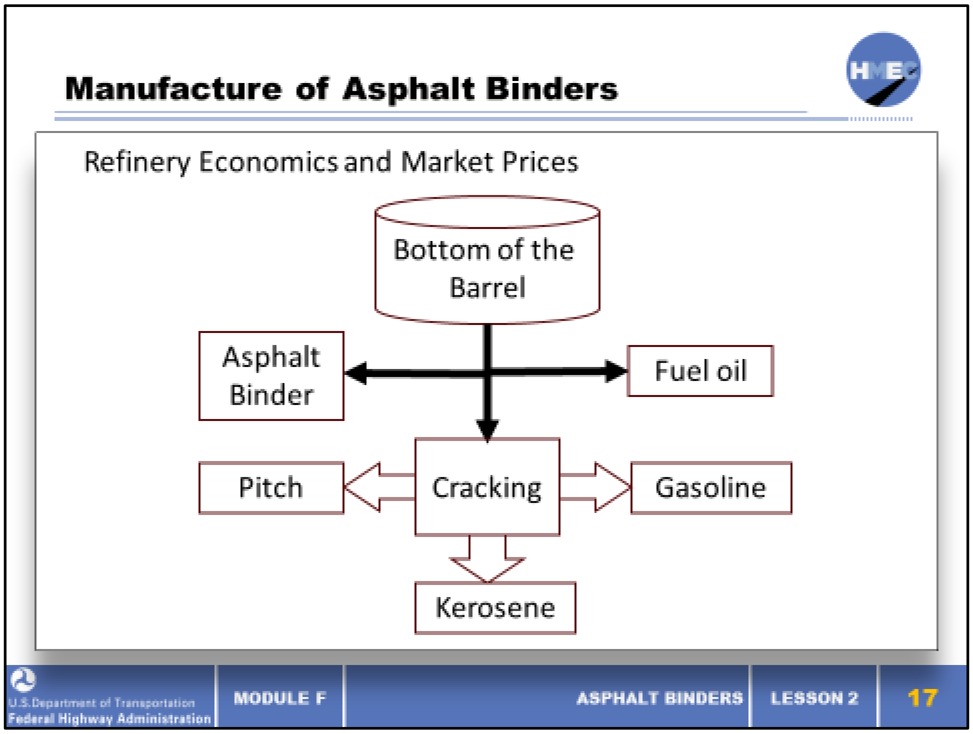

Historically, profits within oil companies have been from the sales of the lighter fuels (gasoline, kerosene, and diesel) and petrochemical feedstocks. In early years, products from the bottom of the barrel, such as asphalt binder, were often difficult to dispose of at a profit. As a consequence, refineries developed processes to alter the higher molecular weight materials for use as fuels or feedstocks. Furthermore, the addition of cokers to many refineries further limits asphalt production. Coking is a refinery progress that produces 19% of the finished petroleum product exports. More specifically, coking is a refinery unit operation that upgrades material called bottoms from the atmospheric or vacuum distillation column into higher-value products and, as the name implies, produces petroleum coke, which is a coal-like material. A coker is processing device that converts the residuum from the refining process into low molecular weight products, such as naptha and gas oils. The byproduct of this process is petroleum coke, and no asphalt can be produced. As the costs and demand for high-end petroleum products continues to rise, more cokers are installed and less asphalt is produced. Thus, asphalt is no longer a waste product, and is now considered a specialty petroleum product.

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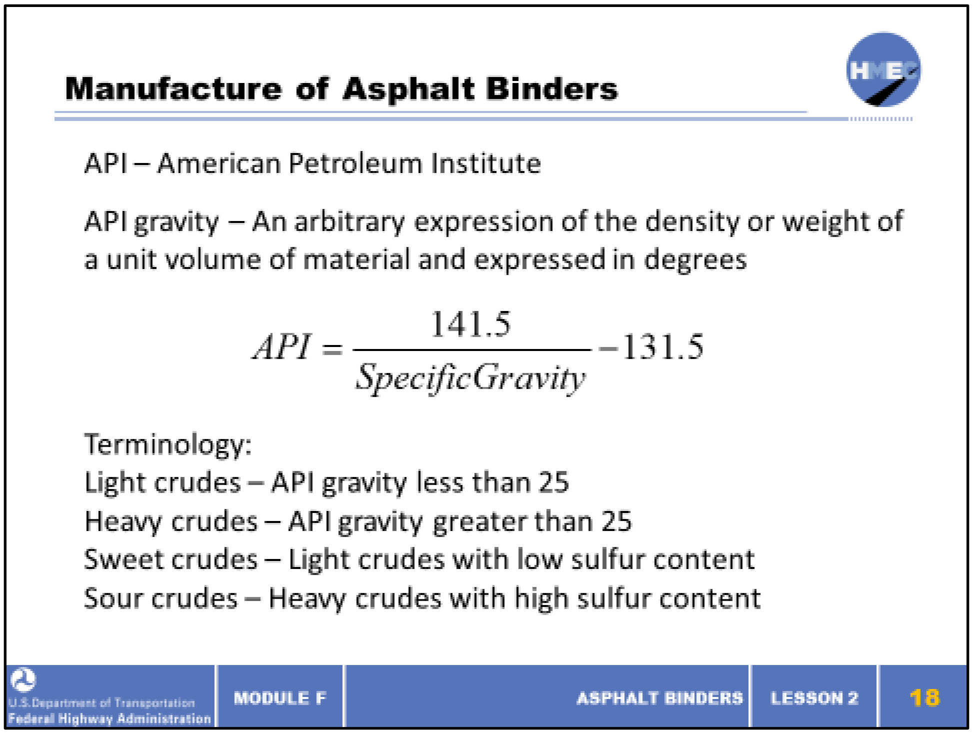
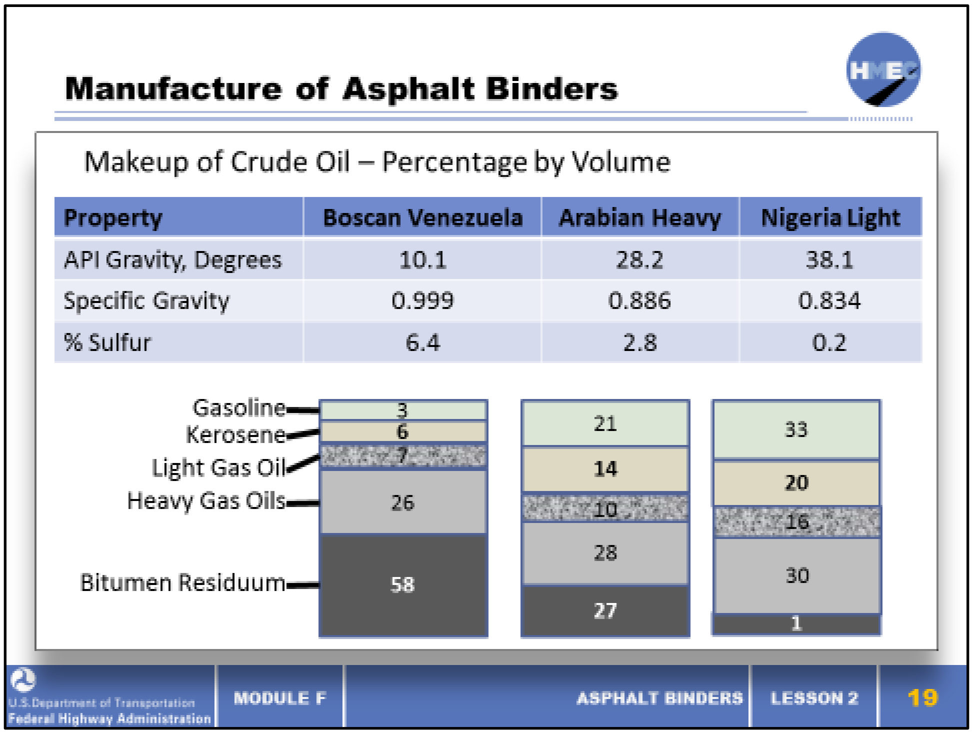
Slide 18



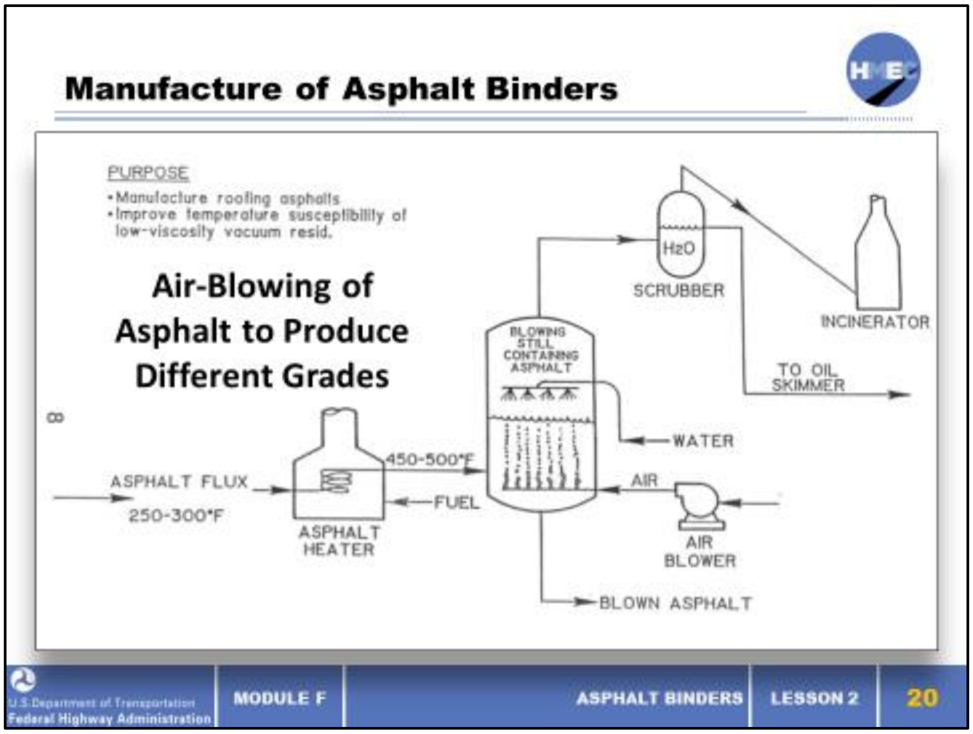
The API gravity is an arbitrary expression of the density or weight of a unit volume of material expressed at 16 °C (60 °F) and is obtained by the formula included in the slide. Low API gravity crudes (API less than about 25) yield relatively low percentages of distillable overhead fractions and high percentages of asphalt binder, while high API gravity crudes (API greater than about 25) yield relatively high percentages of overhead fractions and low percentages of asphalt binder.

Slide 19



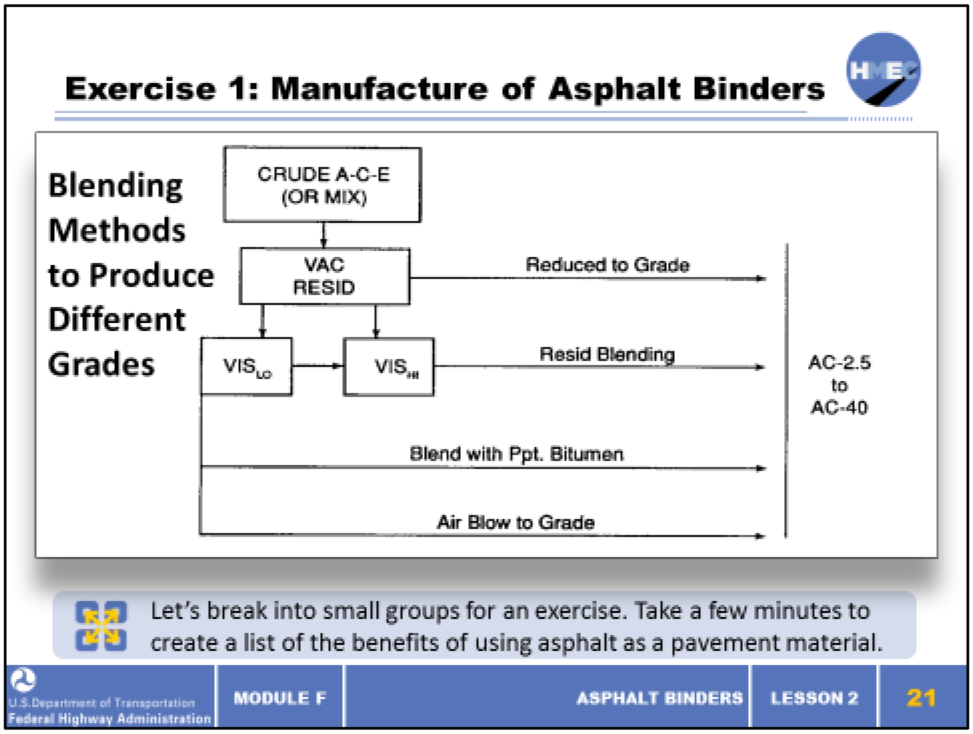
Low API gravity crudes (API gravity in degrees less than about 25) are generally referred to as heavy crudes or as sour crudes if their sulfur contents are high. High API gravity crudes (API gravity in degrees greater than about 25) are known as light crudes or sweet crudes if their sulfur contents are low. Thus, a refiner must select the crude depending on the types and amounts of end products that are to be produced. The types and amounts of end products are typically driven by market demand.

Slide 20



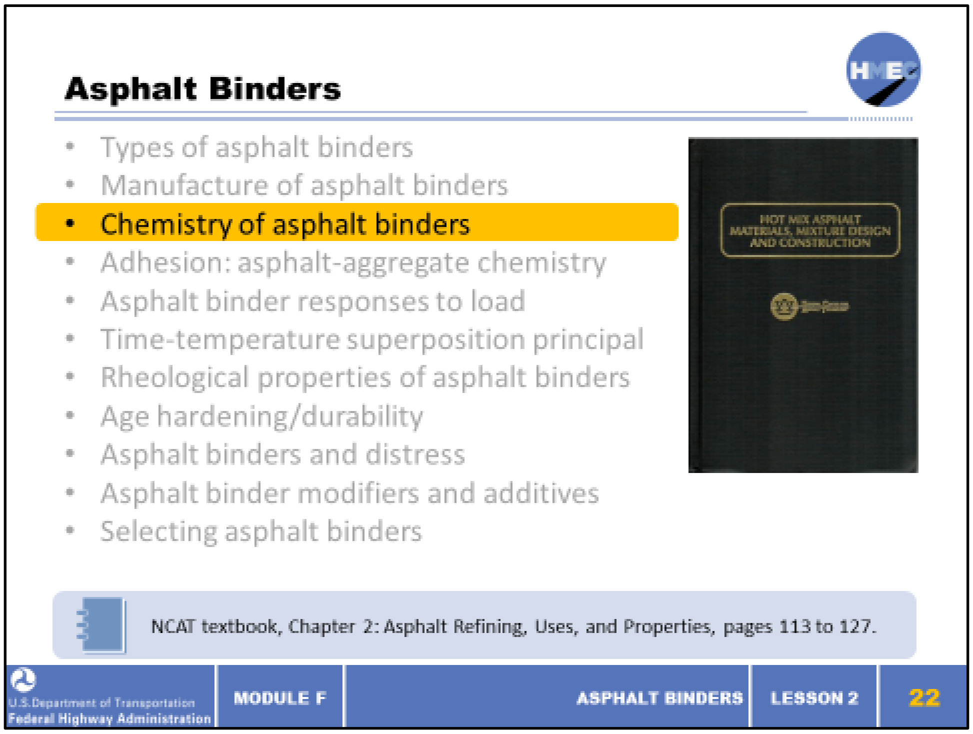
Asphalt binders produced by the vacuum and steam process or the solvent extraction method may be further processed by blowing air through the asphalt binder in large stills at elevated temperatures. This process is employed to make specialty binders for roofing, pipe coatings, waterproofing linings for hydraulic structures, etc. Materials prepared by air blowing, solvent de-asphalting, and vacuum and steam refining can be combined to produce satisfactory asphalt binders.

Slide 21



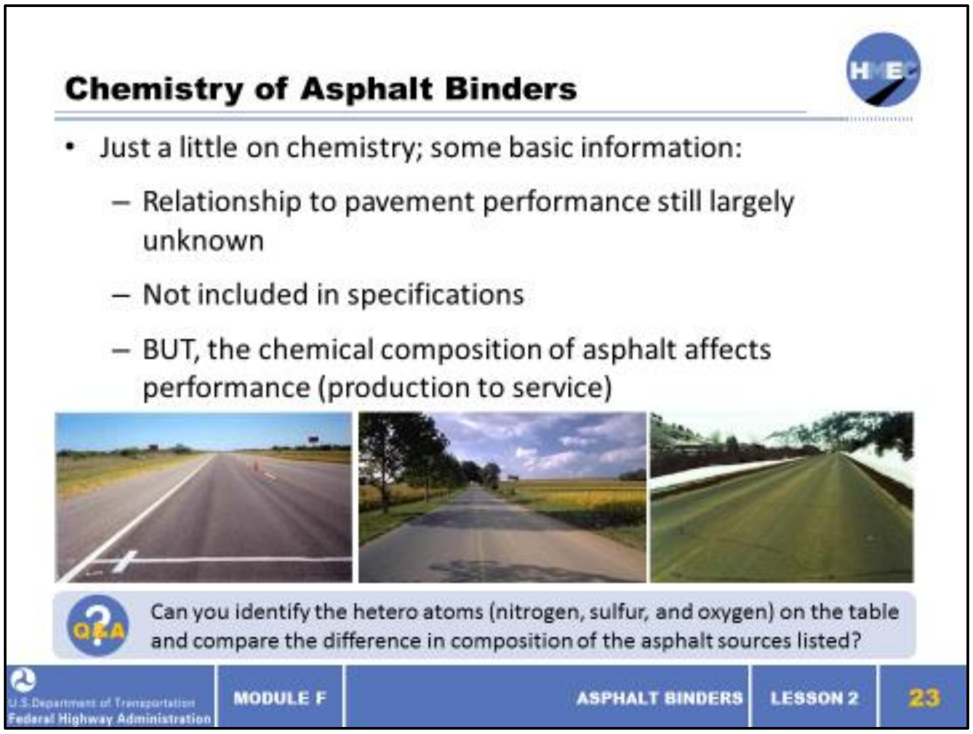
Relatively high flash distillates, such as gas oils have been used as blending materials with hard asphalt binders. Low viscosity (soft) asphalt binders can be blended with the harder solvent extracted asphalt binders or air blown to grade.

Slide 22



The next set of slides discuss the chemistry and aging of asphalt binders.

Slide 23



Review table 2-4 on page 114 in the NCAT textbook to overview the different elements that typically make up the asphalt binder. Four different crude sources of asphalts used in the US are shown on this table with the percentage of the major elements of each crude source. Carbon is dominating element, followed by hydrogen and sulfur. Asphalt binders have extremely diverse molecular structures depending on the crude source. Nitrogen, sulfur, and oxygen are considered “hetero atoms” and often impart polarity onto the molecules that they are associated with. For this reason, while these three molecules represent a relatively small proportion of asphalt binder’s elemental composition, they tend to dominate the material behavior. In part, even small differences in the presence of sulfur, nitrogen, and oxygen causes natural variation in the material properties.

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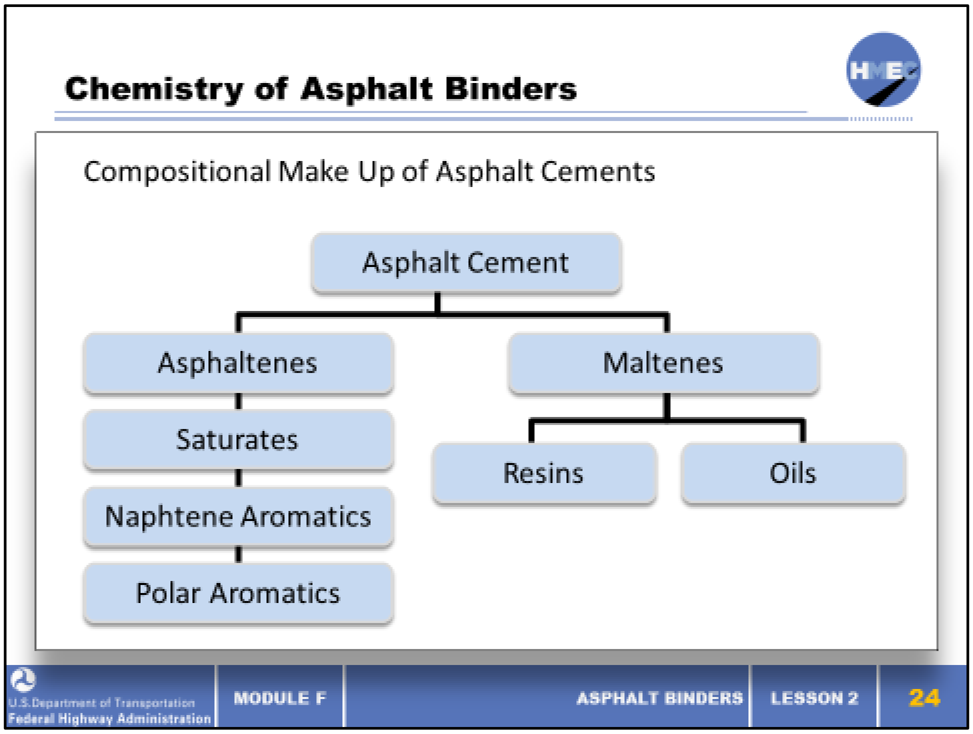
Slide 24



Asphaltenes are insoluble when the asphalt cement is dissolved in a nonpolar solvent such as pentane, hexane, or heptane. Asphaltenes are dark brown and friable solids. The type of nonpolar solvent used to precipitate the asphaltenes affects the determination of its total amount in the asphalt content. Asphaltenes are the complex components with the highest polarity. They play a major role as the viscosity building component of asphalt cements. Low asphaltene content or less than about 10% or weakly associating asphaltenes have been linked with tenderness in HMA mixtures.

Saturates are liquid at ambient temperatures and hardly change with time. They have a negative contribution to temperature susceptibility of asphalt or how properties of the asphalt change with temperature changes.

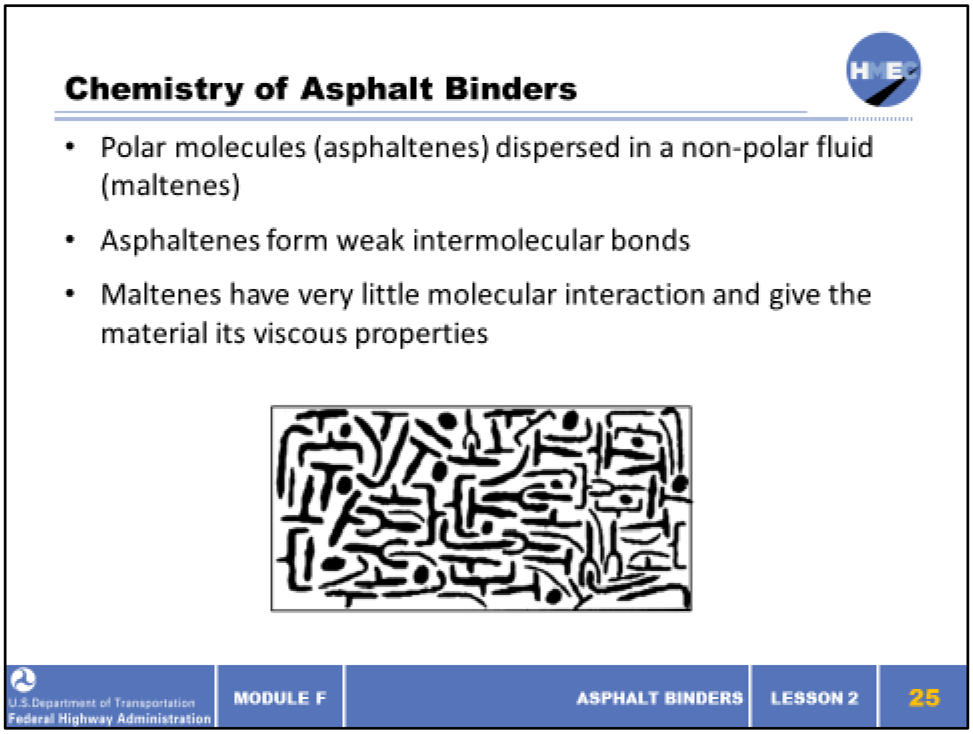
Naphtene aromatics are liquid at ambient temperature and are considered to be the softening component in asphalt. They are also the part that determines the aging fraction of how the properties of the asphalt change with time under service conditions.
Polar aromatics are solid or semi-solid at ambient temperatures and are related to the ductility (how the asphalt can stretch prior to fracture) of the asphalt. They are also one of the components that define the aging fractions.

Maltenes are soluble and are composed of resins and oils.

Resins are dark and semi-solid in character. They are fluid when heated and become brittle when cold. The work as agents that disperse the asphaltenes throughout the oils to provide a homogeneous liquid.

Oils are usually colorless or white liquids. They are soluble in most solvents. They have paraffinic and naphthenic structures with no oxygen and nitrogen present.

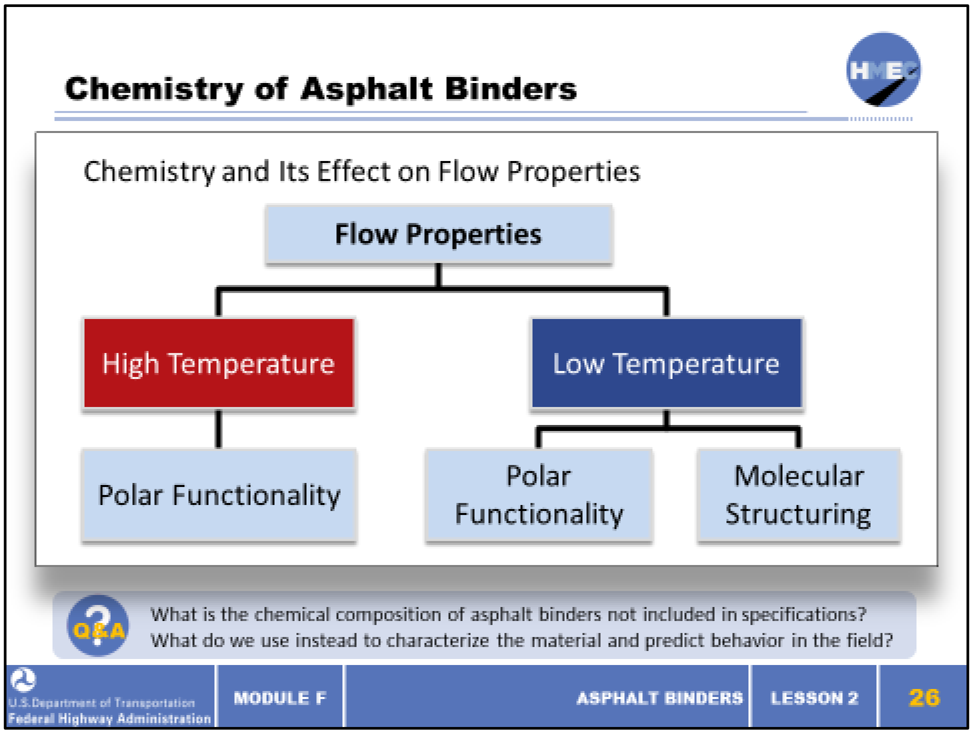
Slide 25



The theory behind the model is that polar molecules (asphaltenes) are dispersed in a non-polar fluid (maltenes). The asphaltenes form weak molecular bonds, while the maltenes have very little molecular interaction. The physical properties of asphalt are the result of intermolecular bonds of the asphaltenes forming, breaking, and reforming.

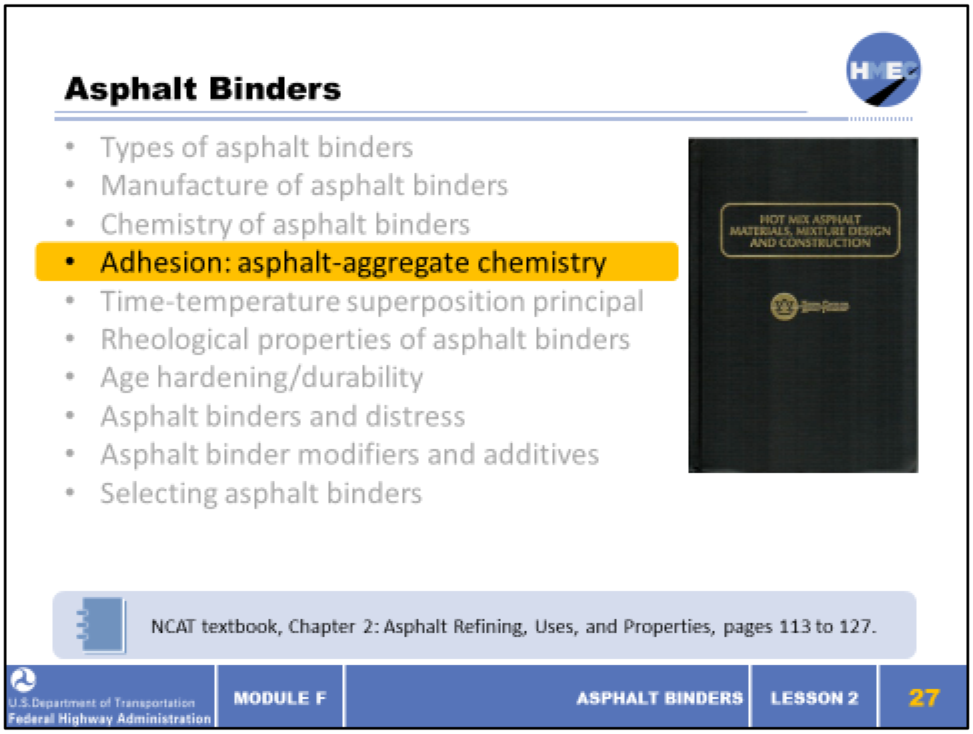
Picture a bowl full of spaghetti. The asphaltenes are like the noodles, and the tomato sauce are like the maltenes.

Slide 26



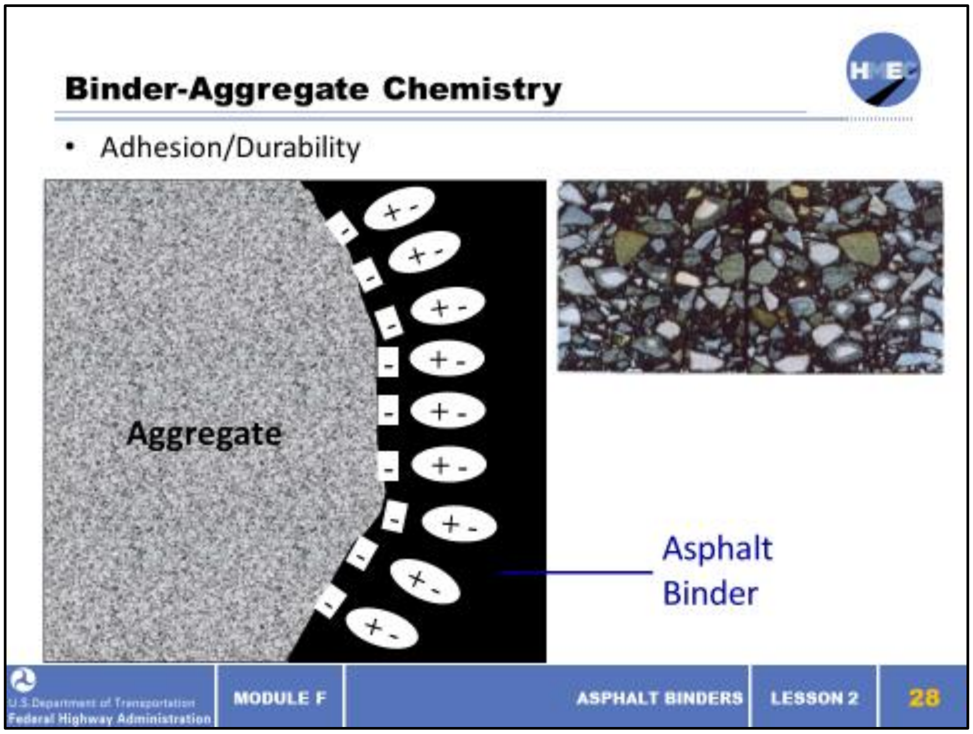
According to much of the literature, and specifically Petersen, the polar interactions between molecules dominate in influencing the flow behavior at high temperatures. At low temperatures, the asphalt molecules tend to associate or agglomerate into immobilized entities with a more or less ordered or structural arrangement. This ordered arrangement is influenced by polar functionality and by the geometry of the molecules.

Slide 27



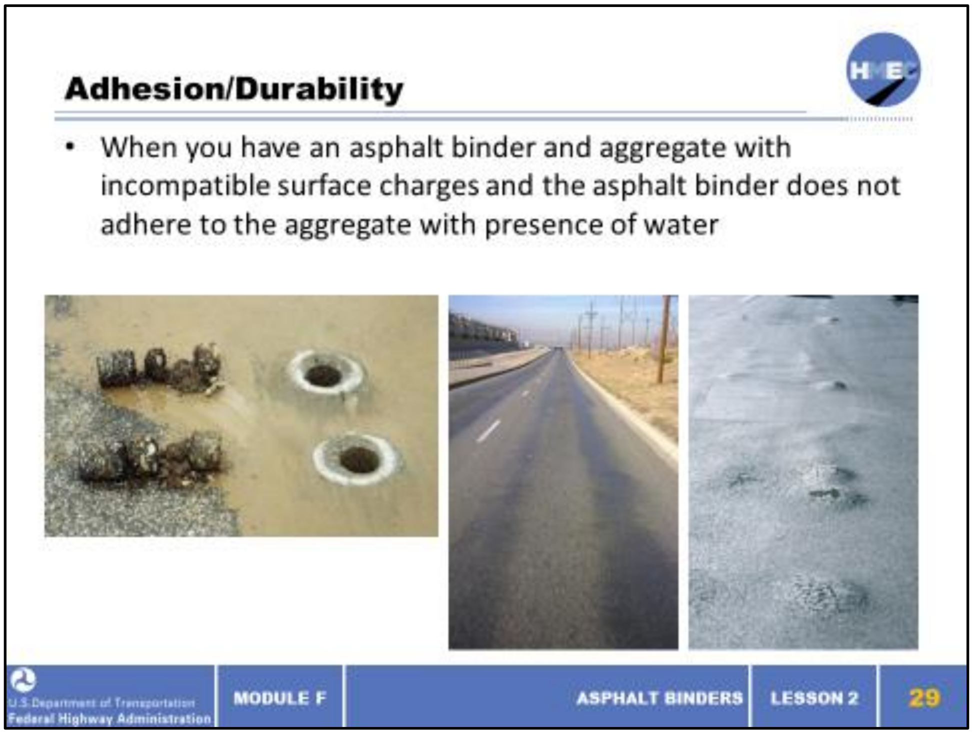
The asphalt binder-aggregate chemistry, as well as other properties play a significant role in the adhesion between the asphalt binder and aggregate. Adequate adhesion is also determined by having a sufficient amount of binder in the mixture to ensure complete coating which also depends on both asphalt and aggregate properties, as well as the amount of asphalt in the mixture and the absorption of the aggregate particles. Thus, it will be discussed in greater detail in a latter lesson on mixture design.

Slide 28



Physico-chemical aspects of the binder and aggregate may affect the long-term adhesion of an asphalt binder to an aggregate. Certain molecular weight components of the asphalt binder tend to be polar in nature. The surface of the aggregate is charged; hence the polar molecules of the binder will tend to orient themselves with respect to the charged aggregate surface and promote adhesion. This orientation may take place quickly and/or gradually develop with time. It is also heavily affected by how the asphalt properties change over time after production and compaction.

Slide 29



The photo in the center shows HMA bleeding and flushing because stripping is occurring in the wearing surface. The photo on the right shows stripping that is occurring in a layer beneath the surface, so shoving is exhibited at the surface. The surface is cracking in localized areas where shoving or lateral displacement of the mix in the lower layer has occurred. The layer with the stripping has little to no shear strength. The photo on the left shows cores recovered from an HMA base layer that was susceptible to moisture damage. The lower part of the cores disintegrated from the wet coring process. This pavement exhibited cracking and rutting. As such, the adhesion between the aggregate and asphalt is very important in determining performance over time.

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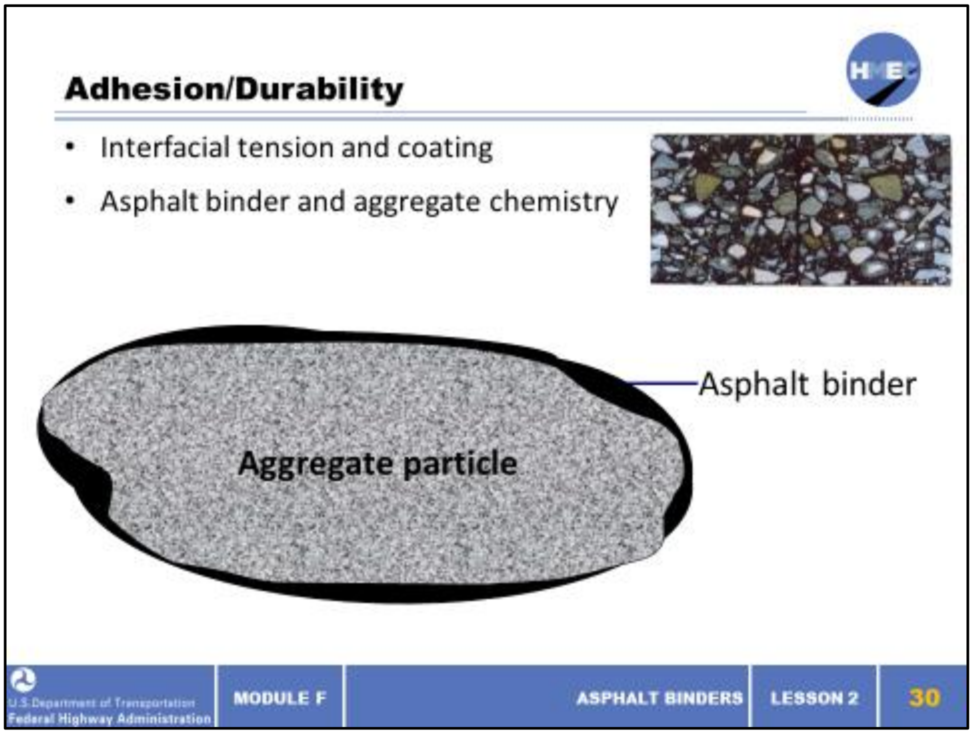
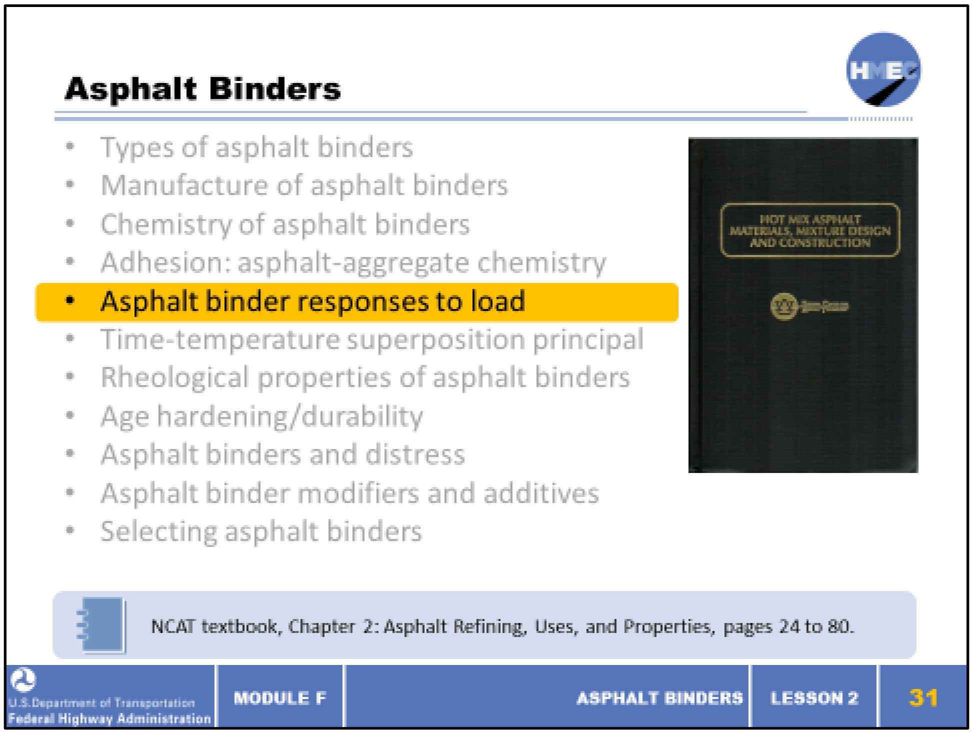
Slide 30



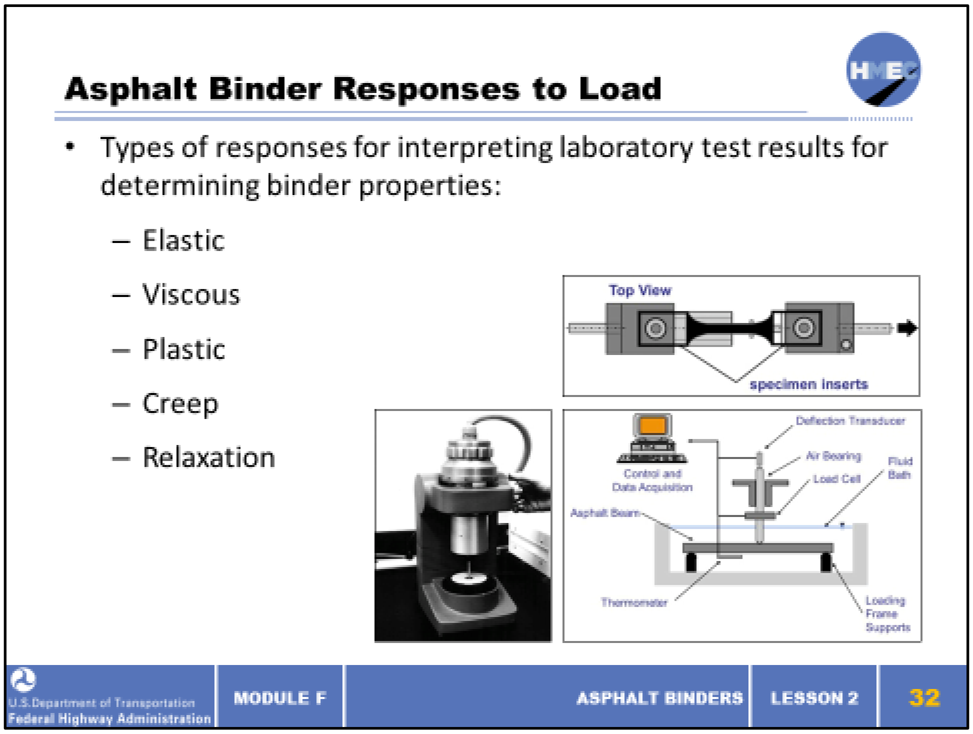
The aggregate normally plays the dominant role when considering the adhesion of an asphalt binder to an aggregate in the presence of water. However, certain physical and chemical properties of asphalt binders are of importance. Asphalt viscosity being an important property because the effects of viscosity of an asphalt binder may tend to offset interfacial tension considerations.

Slide 31



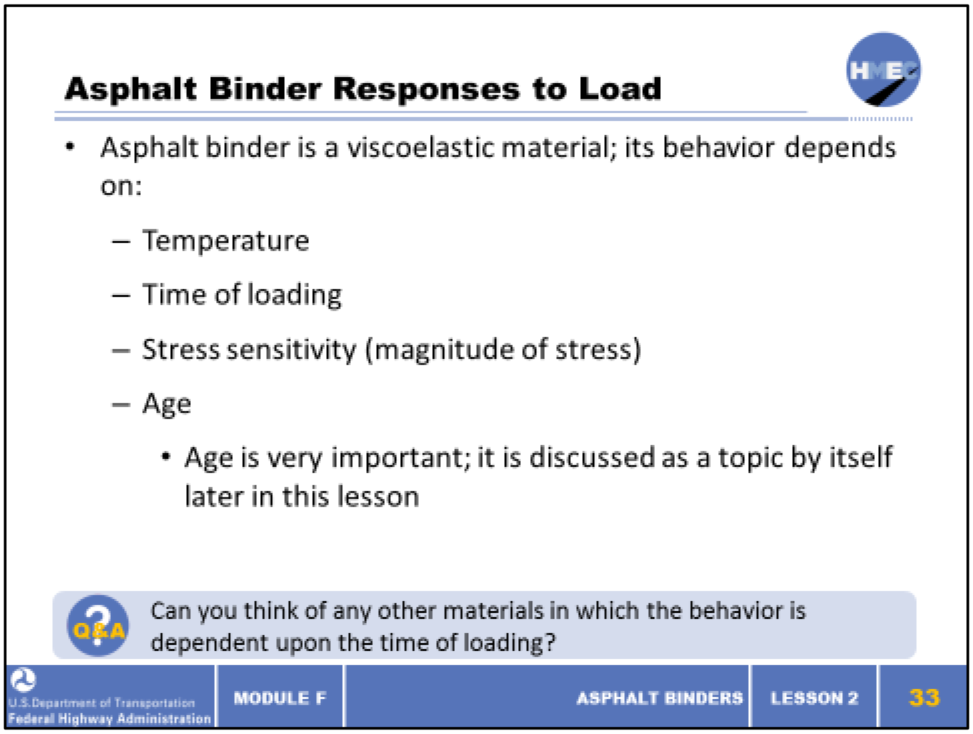
To understand the new asphalt binder grading system, one must understand how the asphalt binder responds to wheel and climate loadings. This section of Lesson #2 provides a basic understanding on the behavior of the asphalt binder under different types of loading conditions.

Slide 32



These responses are used to adequately interpret laboratory test results for estimating the asphalt binder properties. Each will be explained and demonstrated within this topic and used throughout Module F, so they are important.

Slide 33



When these materials are loaded very rapidly, they exhibit elastic behavior and when they are loaded very slowly, they exhibit viscous behavior. For an intermediate range of load rates they exhibit a combination of elastic and viscous behavior. Such materials are referred to as viscoelastic materials. Asphalt binders are viscoelastic materials and, as a consequence, so are HMA mixtures.

Slide 34


Asphalt Binder Responses to Load
• Demonstration: Silly Putty
Here Is Putty with a Bounce
Research in silicone rubber yields a strange by-product that may have its own uses.
U.S. Department of Transportation
Federal Highway Administration
MODULE F
ASPHALT BINDERS
LESSON 2
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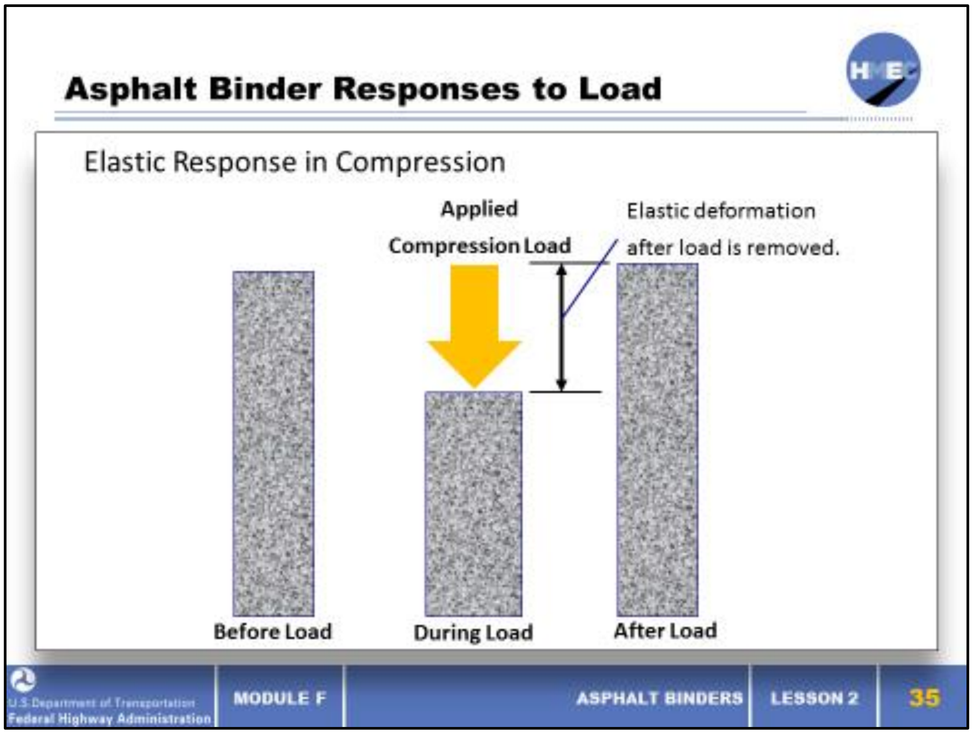
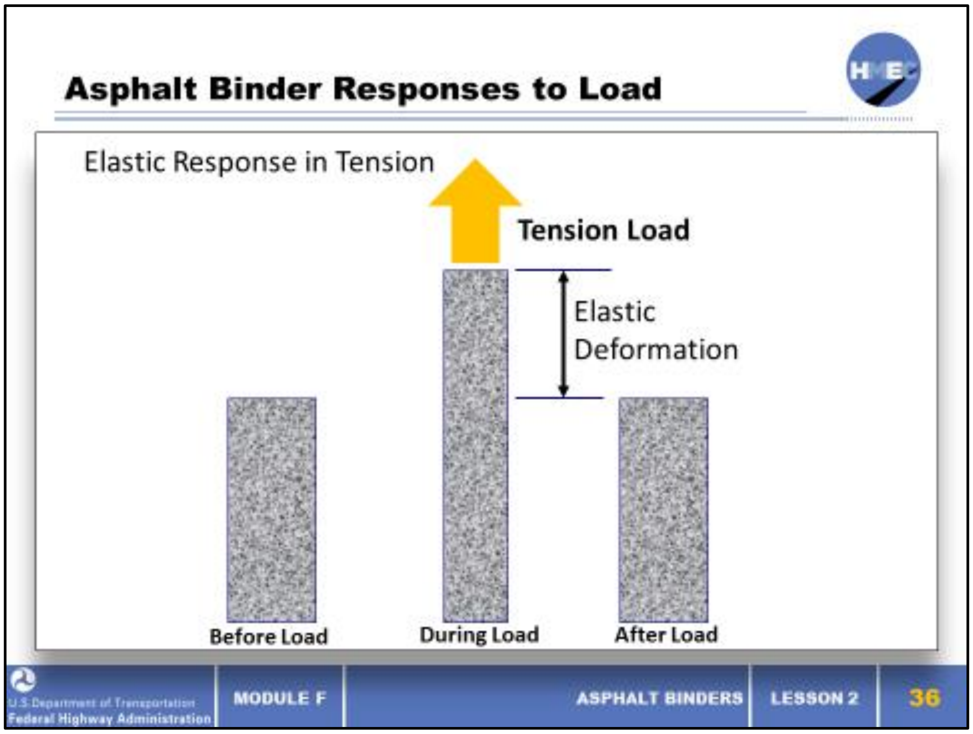
Slide 35



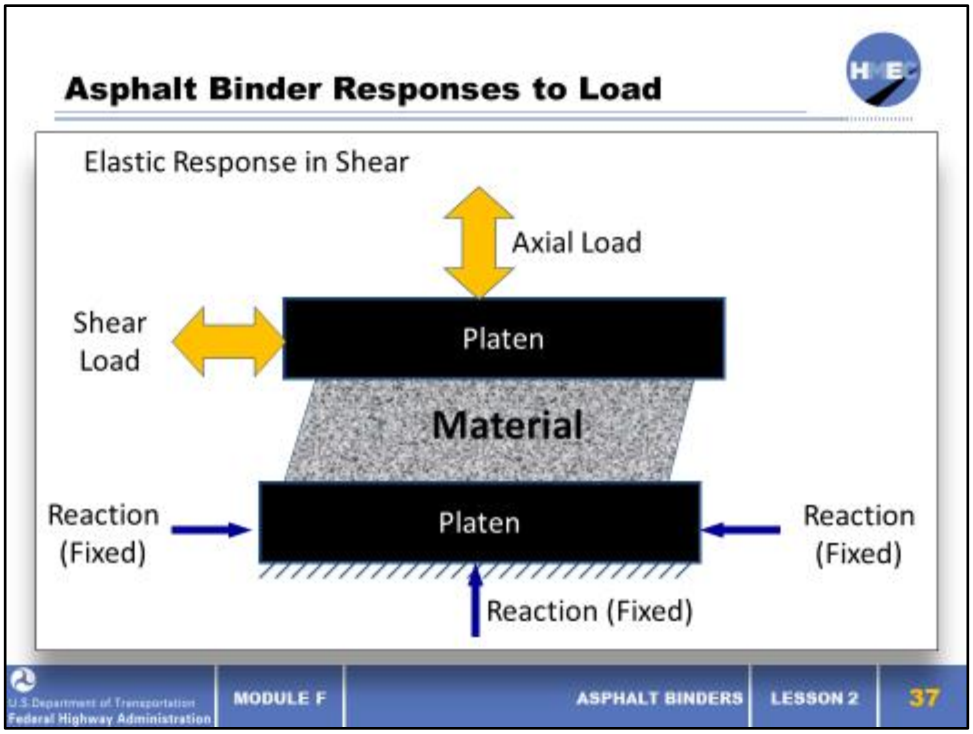
This slide shows the original shape of an elastic material, the shape of the material and resulting deformation when a compressive load is applied, and the shape of the material after the load is removed. Truly elastic materials have the exact same shape after loading as before loading.

Slide 36



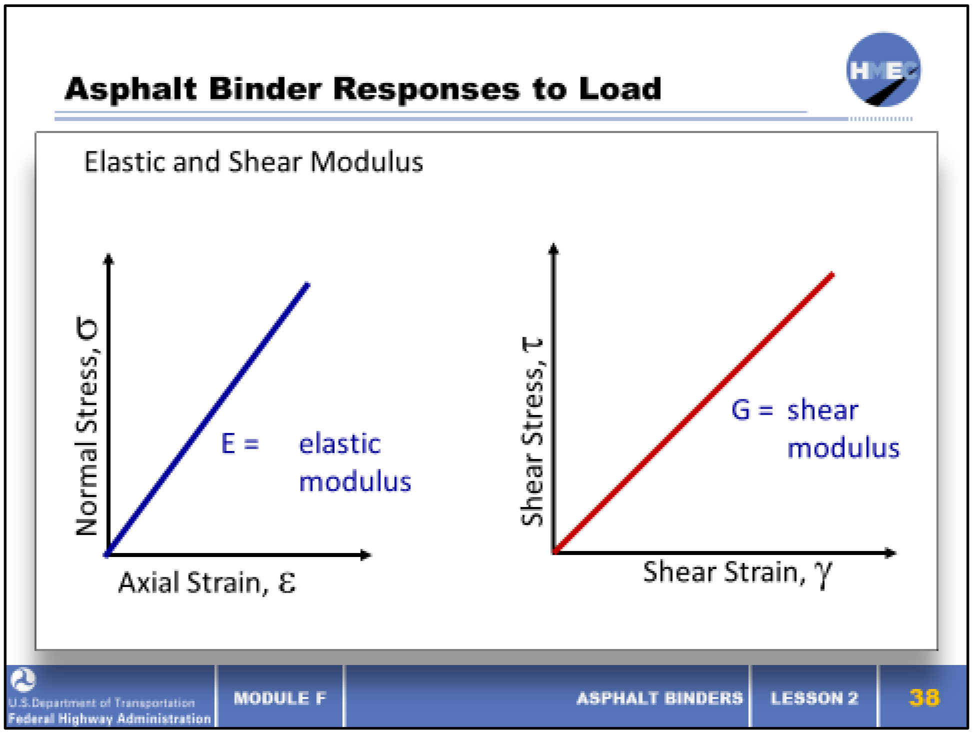
Certain engineering materials exhibit proportionality between applied stress and deformation. Moreover, when a given stress level is reached and maintained, the deformation remains constant. When the stress is removed, the material completely recovers from the deformation that had resulted. Such materials are called elastic materials and Hooke's law governs their behavior.

Slide 37



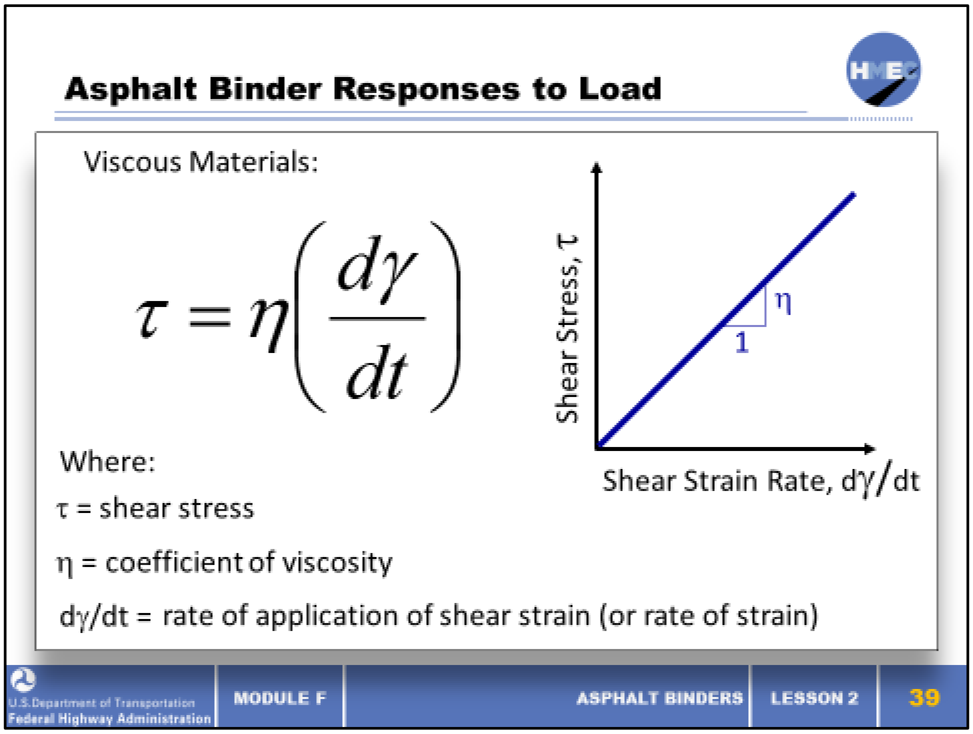
Independent of the form of the load—whether it is compressive, tensile, or a shear load, as shown here—elastic materials will recover their shape when the load is removed.

Slide 38



When elastic materials are subjected to tensile or compressive (normal) loads, the proportionality constant between the applied normal stress and resultant axial strain is the elastic modulus, E. If they are loaded in shear, the proportionality constant is the shear modulus, G.

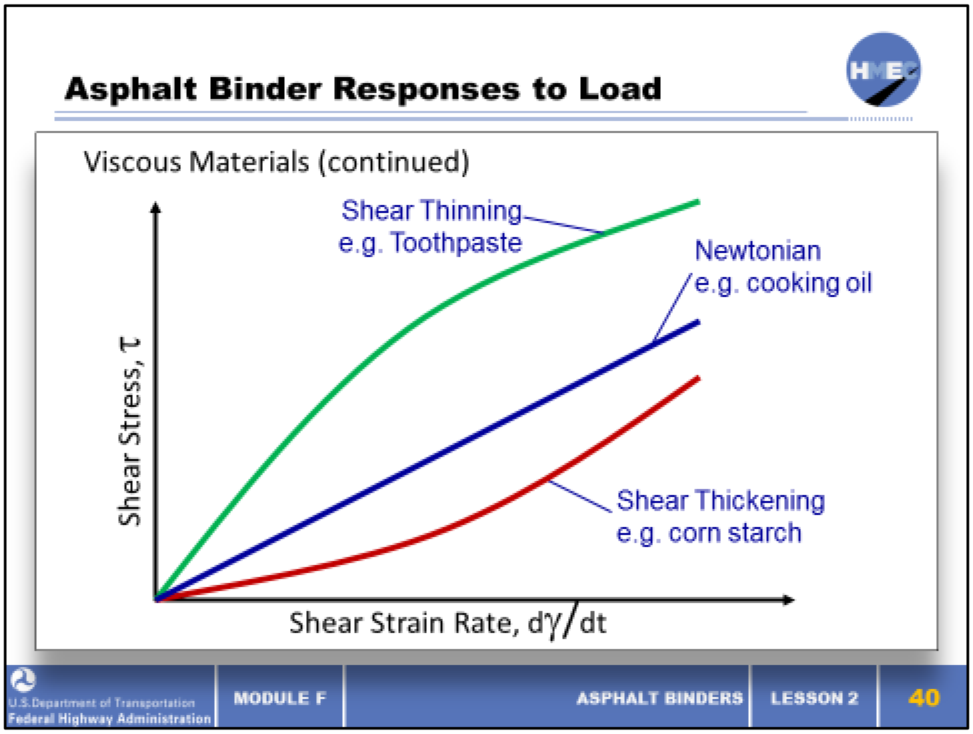
Slide 39



The relationship between shear stress and shear strain rate is shown. If shear stress is measured in dynes/cm^2 (1 dyne = 1/980 grams) and the rate of shear strain in sec^{-1}, then the coefficient of viscosity is given in poises (1 poise = 1 dyne-sec/cm^2).

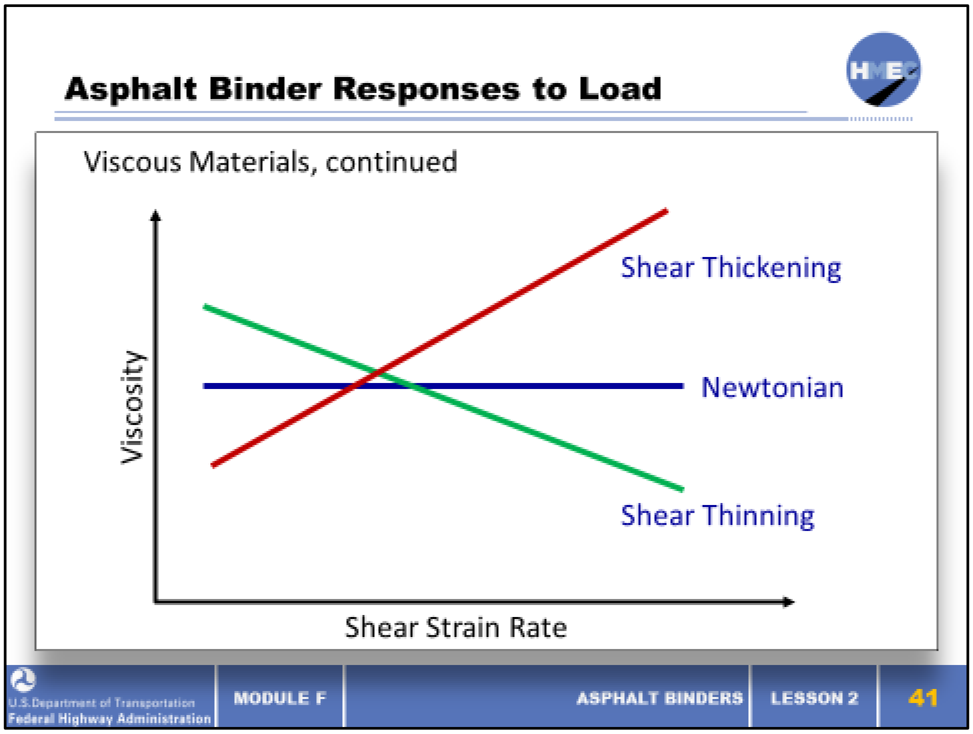
Asphalt binder heated at 163 °C is an example of a Newtonian material.

Slide 40



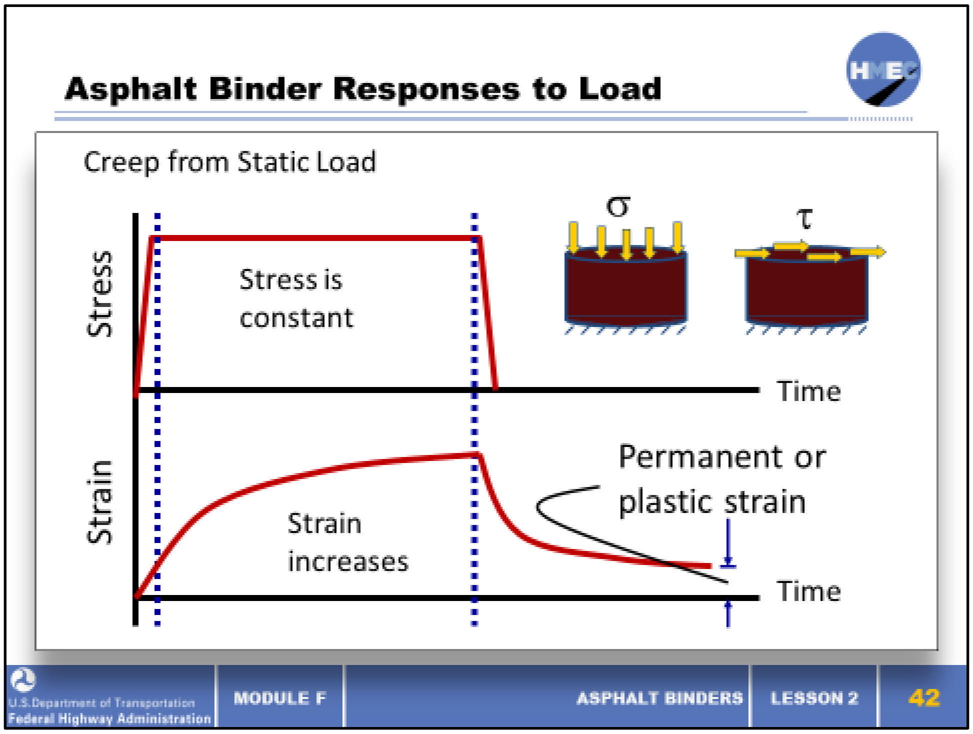
For example, some materials exhibit a curvilinear relationship between shear stress and shear strain rate. Such materials are referred to as non-Newtonian or shear thinning, such as toothpaste, or shear thickening, such as corn starch behavior. The coefficient of viscosity for these materials is the instantaneous slope of the line at a particular shear strain rate. A Newtonian example would be cooking oil.

Slide 41



This plot of the relationship between the coefficient of viscosity (as referred to as simply viscosity) and shear strain rate better identifies changes in viscosity with shear strain rate. Many modified asphalt binders are non-Newtonian in behavior.

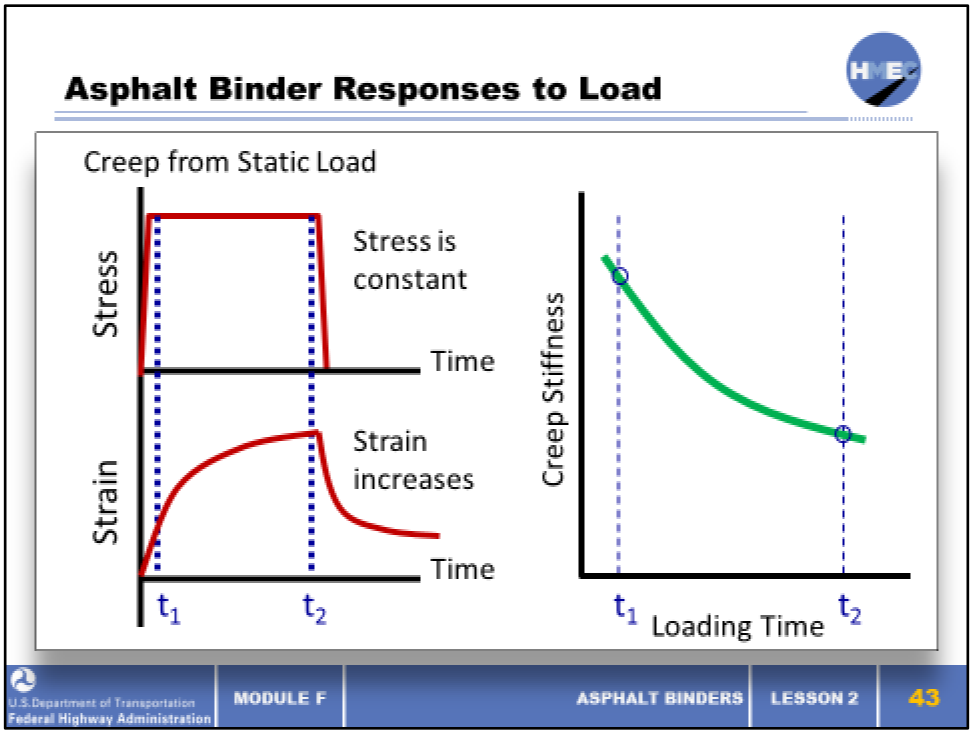
Slide 42



The rate of strain developed under a constant load is a function of binder viscosity and temperature. When the load is removed, the binder will rapidly recover some of the deformation due to the elastic properties of the binder, but at a progressively slower rate due to the viscous properties of the binder.

At intermediate to elevated temperatures not all of the deformation is recovered and that which is not recovered is termed permanent deformation or inelastic strain. Asphalt binders can also be tested in bending at low temperatures and will exhibit similar behavior.

Slide 43



In a creep test at a constant temperature, the rate of strain developed in asphalt binder decreases with time and the total strain developed accumulates at a progressively slower rate. Hence, the stiffness (stress divided by total strain at any given time) of the binder decreases with time.

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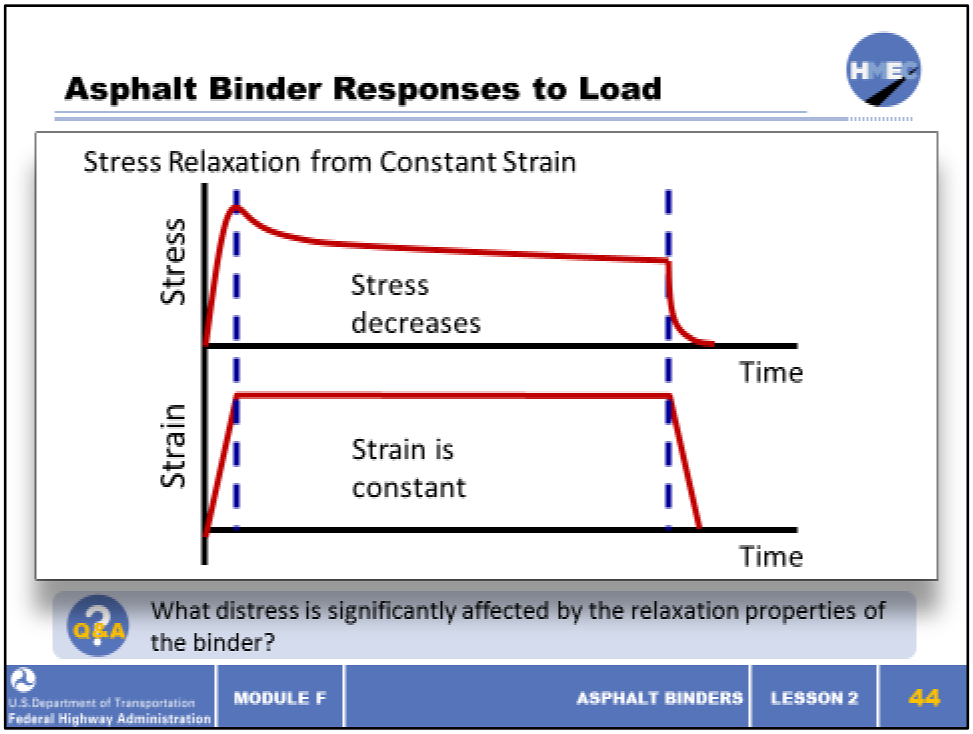
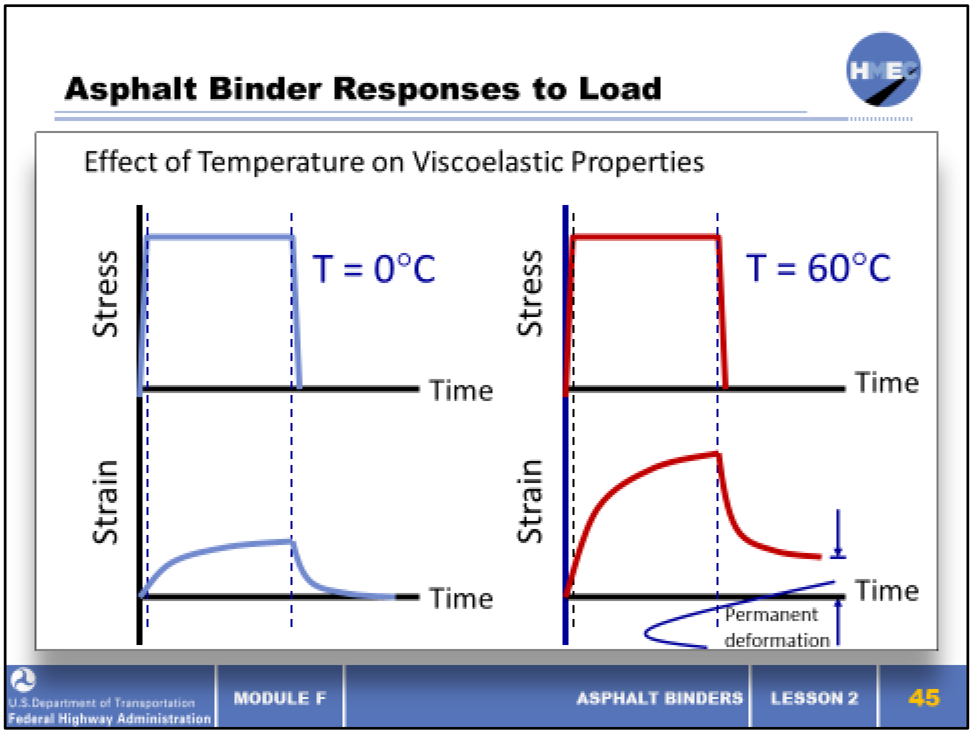
Slide 44



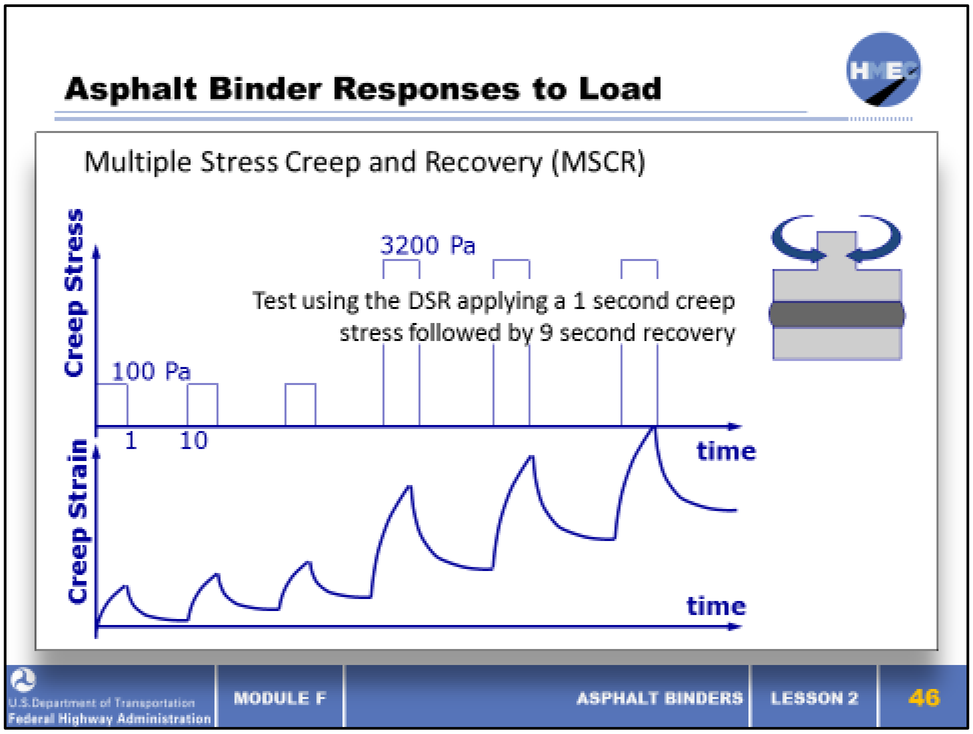
Asphalt binders subjected to a constant rate of strain will exhibit stress relaxation wherein the stress will reach a peak soon after the strain rate is applied, then it declines (becomes less) over time. When the strain is removed, the stress in the binder will rapidly dissipate initially due to the elastic properties of the binder, then at a slower rate due to the viscous properties of the binder.

Slide 45



Asphalt binders tested in a creep test (constant stress) at low temperatures will develop much less deformation (and a lower strain rate) than those tested at high temperatures as they behave in a more elastic manner at low temperatures. In addition, the deformation may be completely recovered when the load is removed at low temperatures, but not at high temperatures.

Slide 46


Asphalt Binder Responses to Load
HMEC
Multiple Stress Creep and Recovery (MSCR)
Creep Stress
3200 Pa
Test using the DSR applying a 1 second creep stress followed by 9 second recovery
100 Pa
1
10
time
Creep Strain
time
U.S. Department of Transportation Federal Highway Administration
MODULE F
ASPHALT BINDERS
LESSON 2
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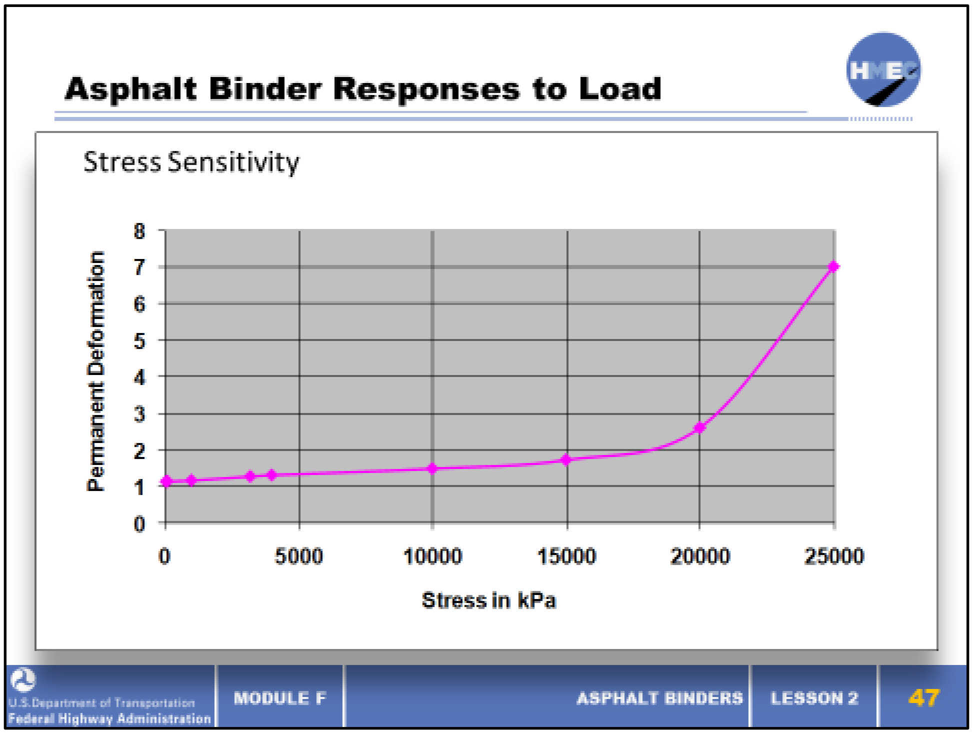
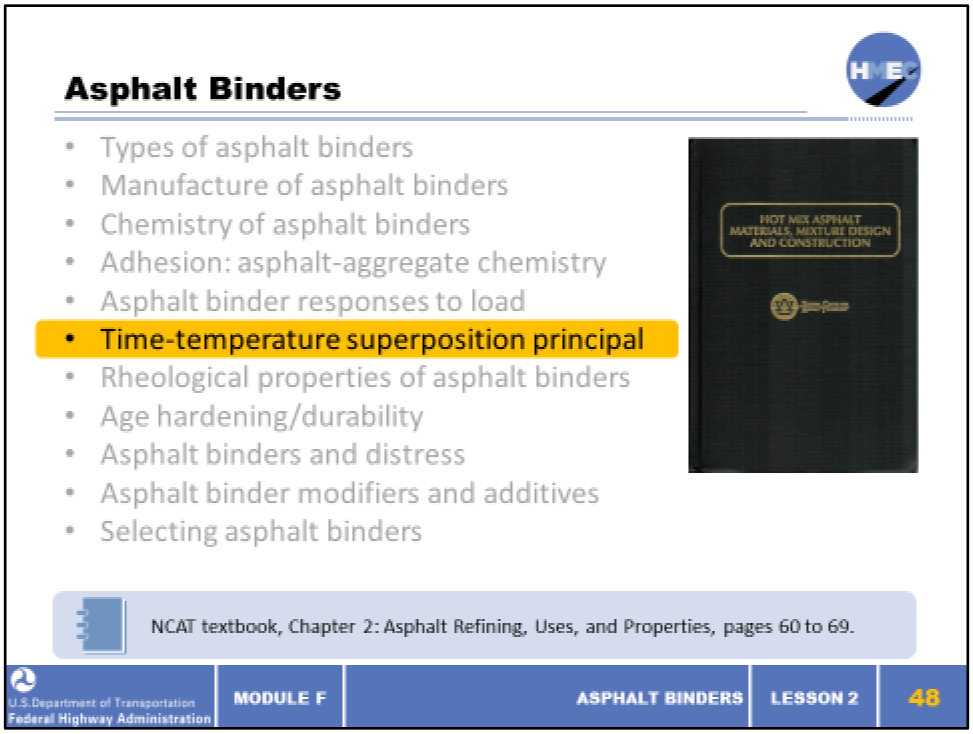
Slide 47



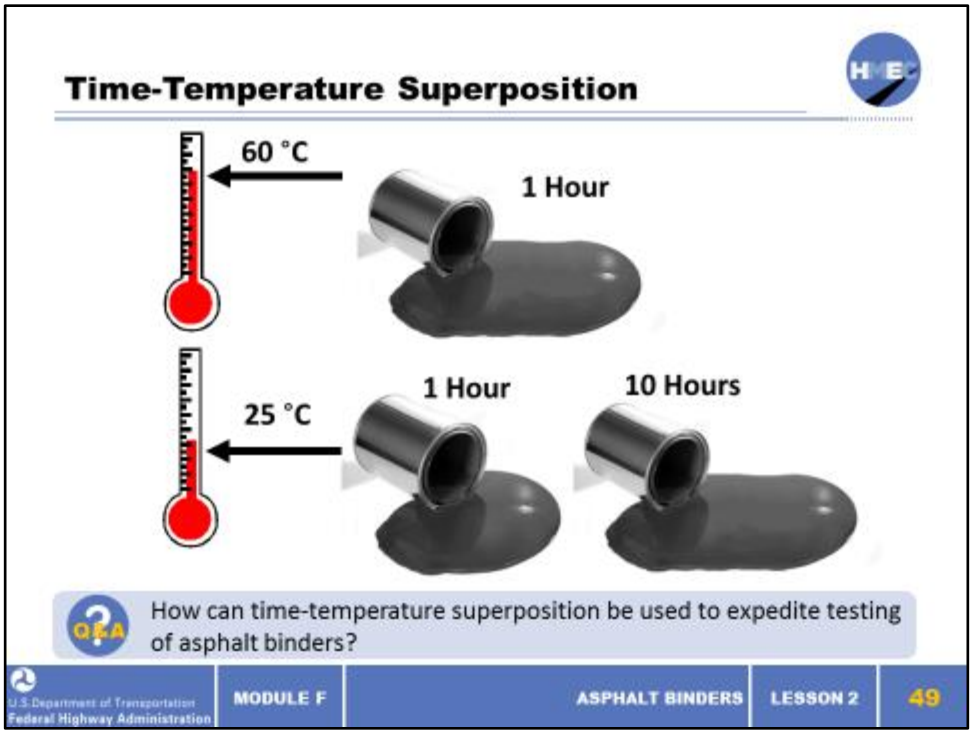
This slide shows the stress sensitivity of asphalt binders at a specific temperature. At high stress levels or axle loads, the amount of permanent or plastic deformation starts to increase at an accelerating rate. The stress applied begins to exceed the strength of the binder, so large plastic deformations start to occur.

Slide 48



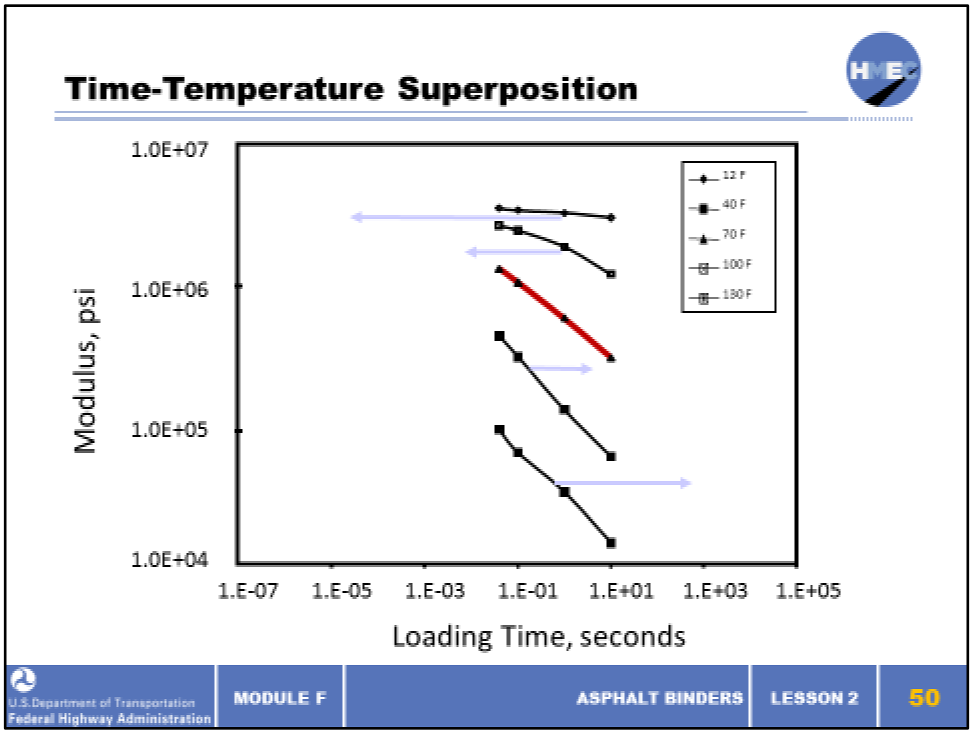
This topic explains and demonstrates this principle and how it simplifies the determination of stiffness for a wide range of conditions.

Slide 49



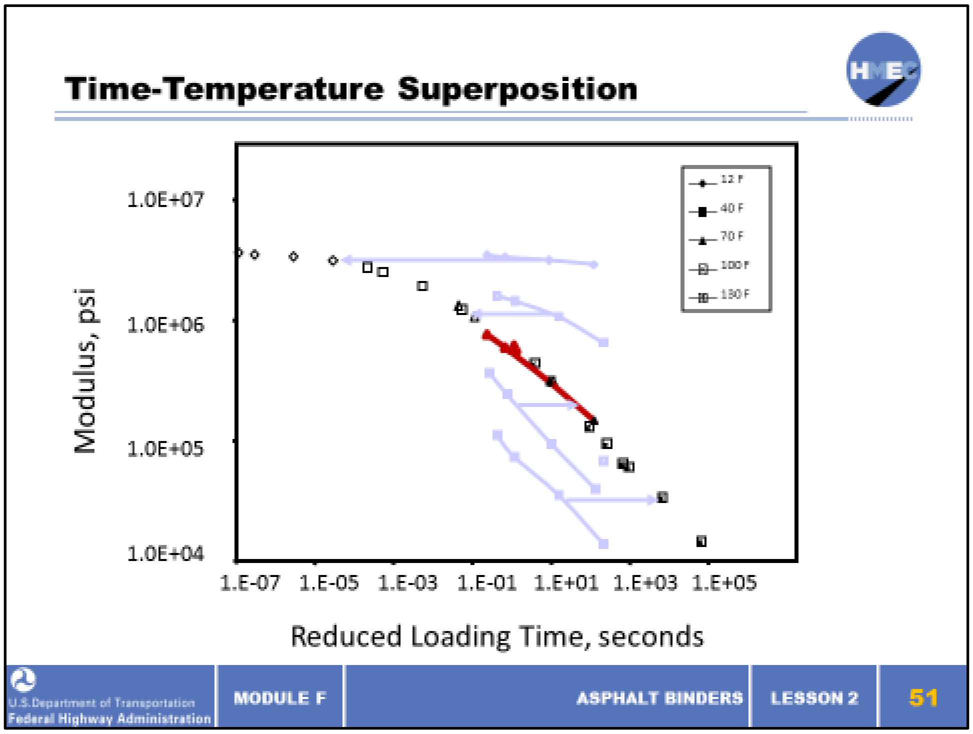
This slide illustrates conceptually the influence of temperature on the behavior of an asphalt binder under constant load (by virtue of constant mass and gravity). It indicates that the amount of binder that flowed from the canister at 60 °C (140 °F) in 1 hour is about the same as that which flowed from the canister at 25 °C (77 °F) in 10 hours.

Slide 50



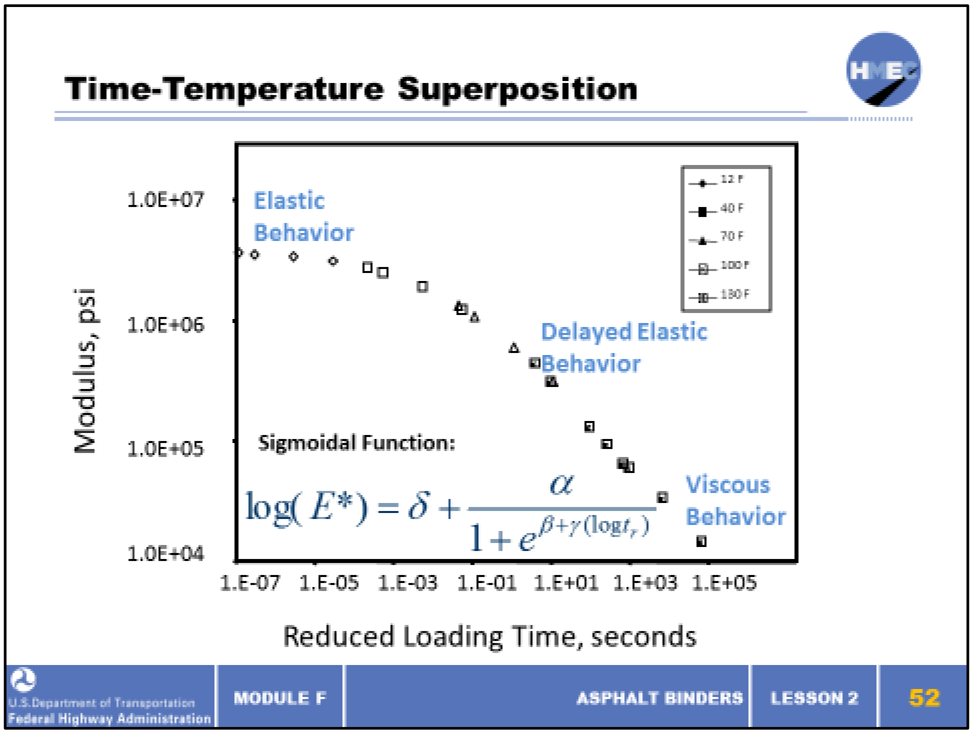
The preceding slide indicated that binder stiffness decreases both with increased temperature and increased time of loading (at a constant temperature). At low temperatures and short loading times, the stiffness or modulus is essentially elastic (loading time and temperature independent). However, at high temperatures and long loading times, the modulus decreases at a more uniform rate and the behavior can be considered viscous. With this being the case, it is possible to superimpose the stiffness-versus-time of loading relationship onto the stiffness-versus-temperature relationship (or vice versa) through Boltzmann's superposition principle. In other words, with asphalt binders (viscoelastic materials), the behavior (stiffness, viscosity) at various loading rates (at a given temperature) can be correlated to the behavior at various temperatures.

Slide 51



The relationship for the shift factors are provided in the next slide.

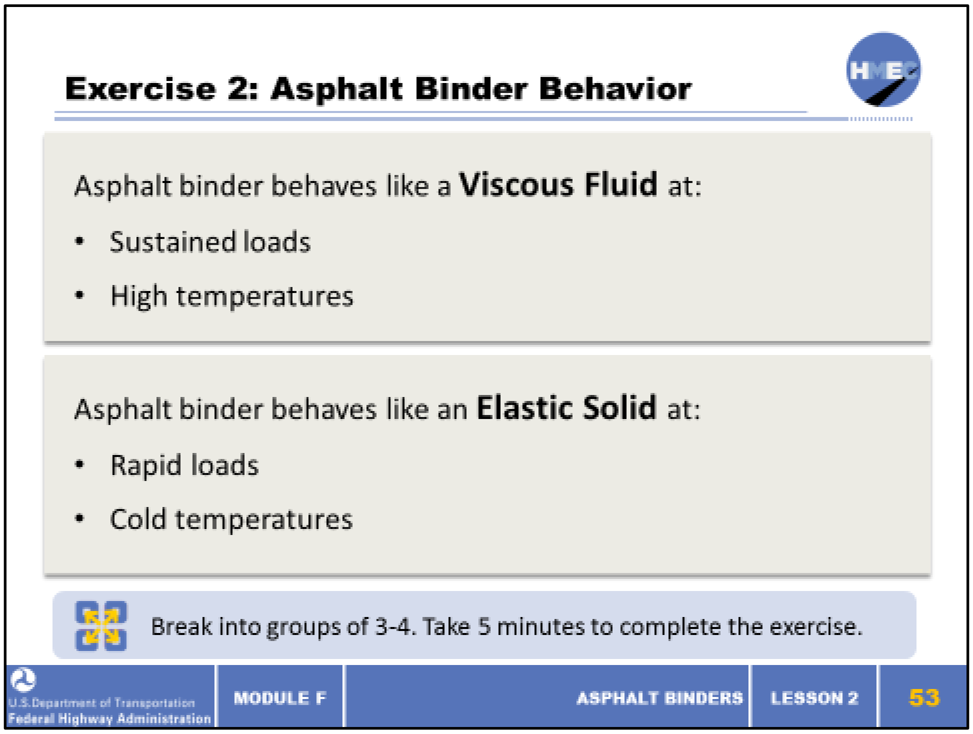
Slide 52



As shown in the slide, at very short loading times the behavior is almost elastic (loading time and temperature independent); as the loading time increases and temperature increases the modulus is simply a relation between the applied stress and resulting strain for which the strain is dependent on the loading time and temperature; and at very long loading times and high temperatures the modulus will decrease with an increase in loading time and temperature but at a uniform rate so the behavior is considered viscous. The sigmoidal function included in the slide is the mathematical relationship for shifting the loading time so that the reduced loading time and temperature result in the same dynamic modulus, E*, through the master curve for the specific asphalt binder.

• E^* is the dynamic modulus measured in the laboratory.
• t_r is the reduced time.
• δ, α, β, and γ are regression constants determined from fitting the data and creating the master curve.

Slide 53


Exercise 2: Asphalt Binder Behavior
HMEC
Asphalt binder behaves like a Viscous Fluid at:
Sustained loads
High temperatures
Asphalt binder behaves like an Elastic Solid at:
Rapid loads
Cold temperatures
Break into groups of 3-4. Take 5 minutes to complete the exercise.
U.S. Department of Transportation
Federal Highway Administration
MODULE F
ASPHALT BINDERS
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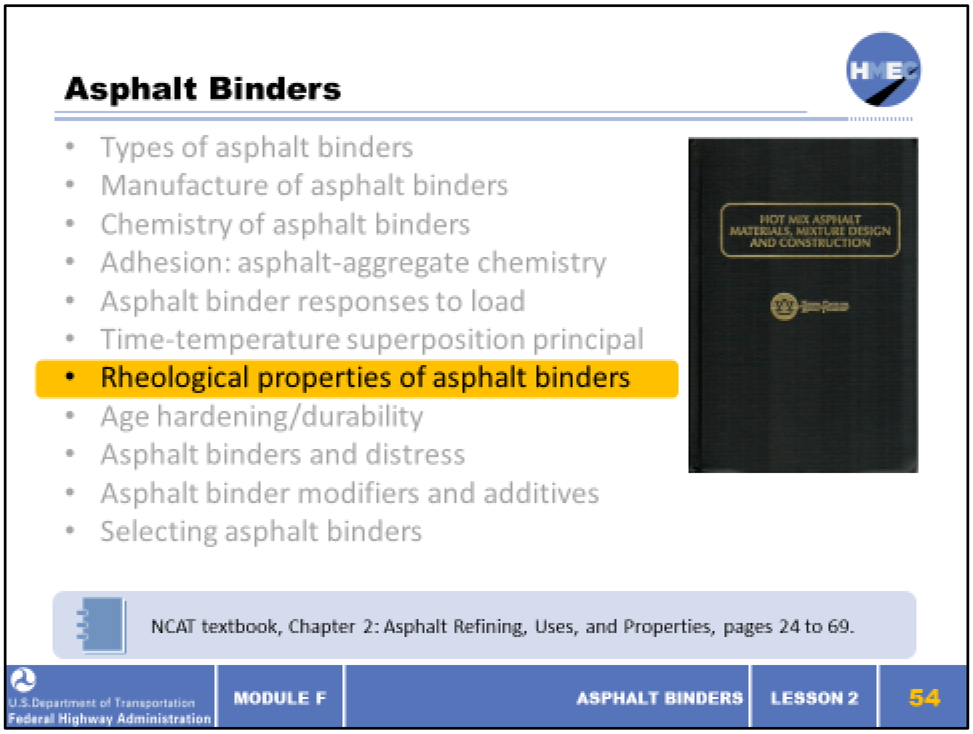
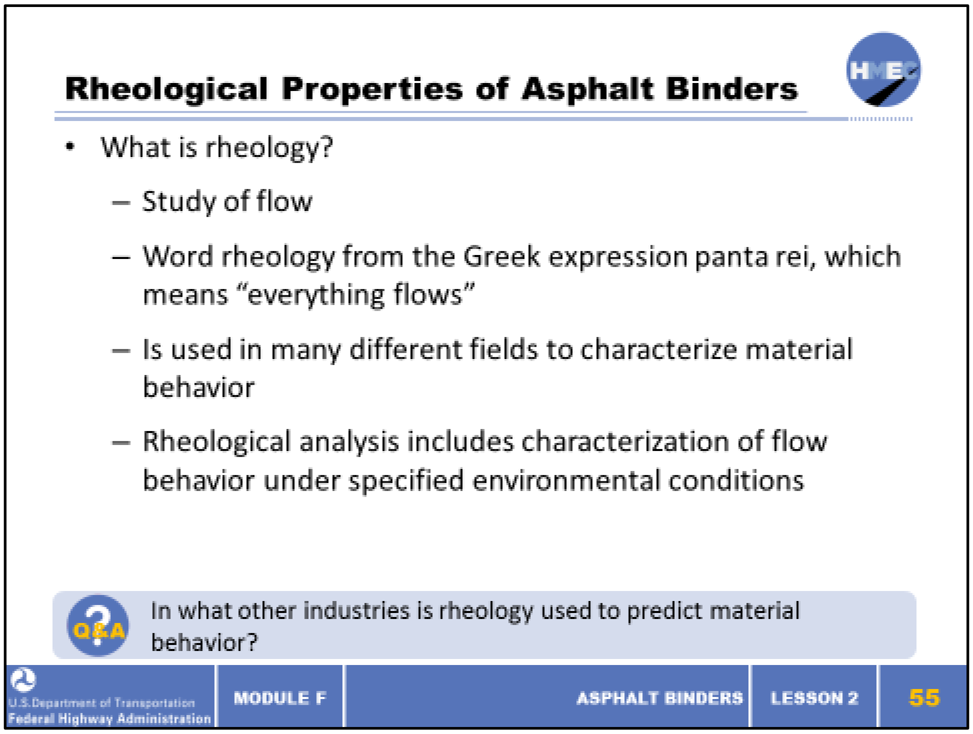
Slide 54



The rheological properties are also used in the binder specifications and binder selection, so they are extremely important.

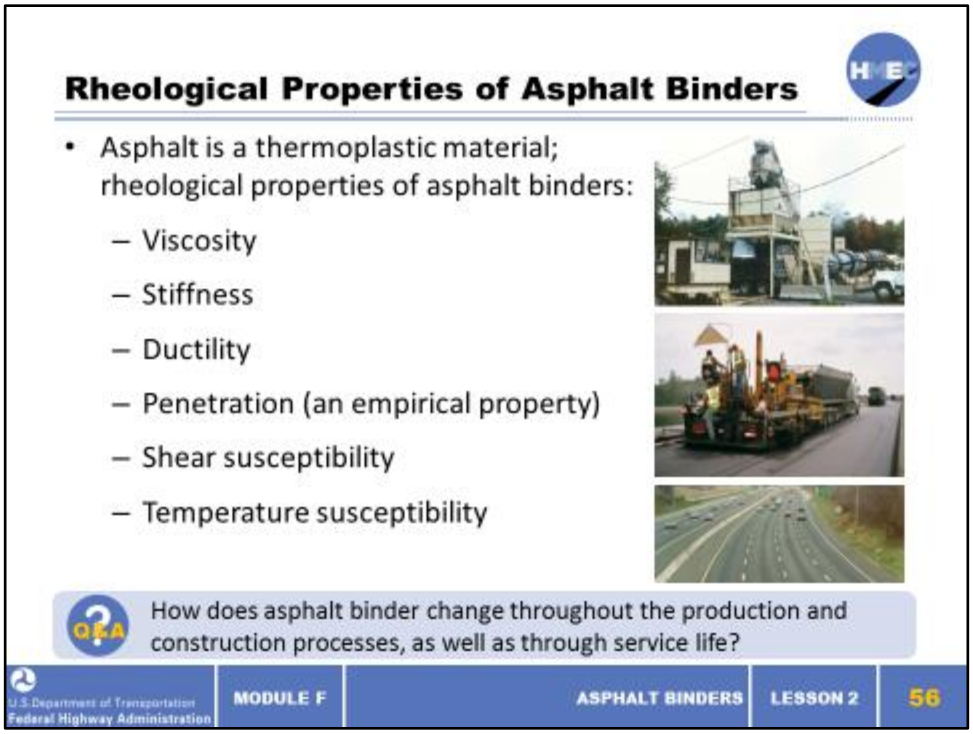
Rheology is defined as the science of flow and deformation of matter and describes the interrelation between force, deformation and time.

Slide 55



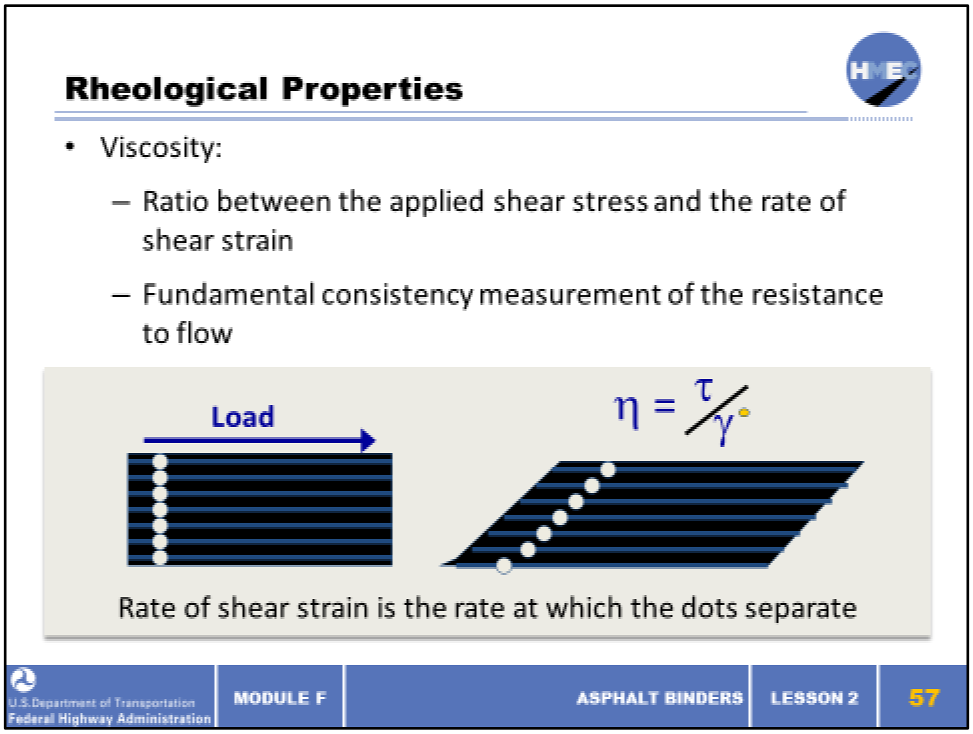
Rheology is the study of flow. The word *rheology* itself has Greek origins, coming from the phrase *panta rei,* which means "everything flows." We use rheology to characterize the flow behavior of asphalt binder under specified environmental conditions.

Slide 56



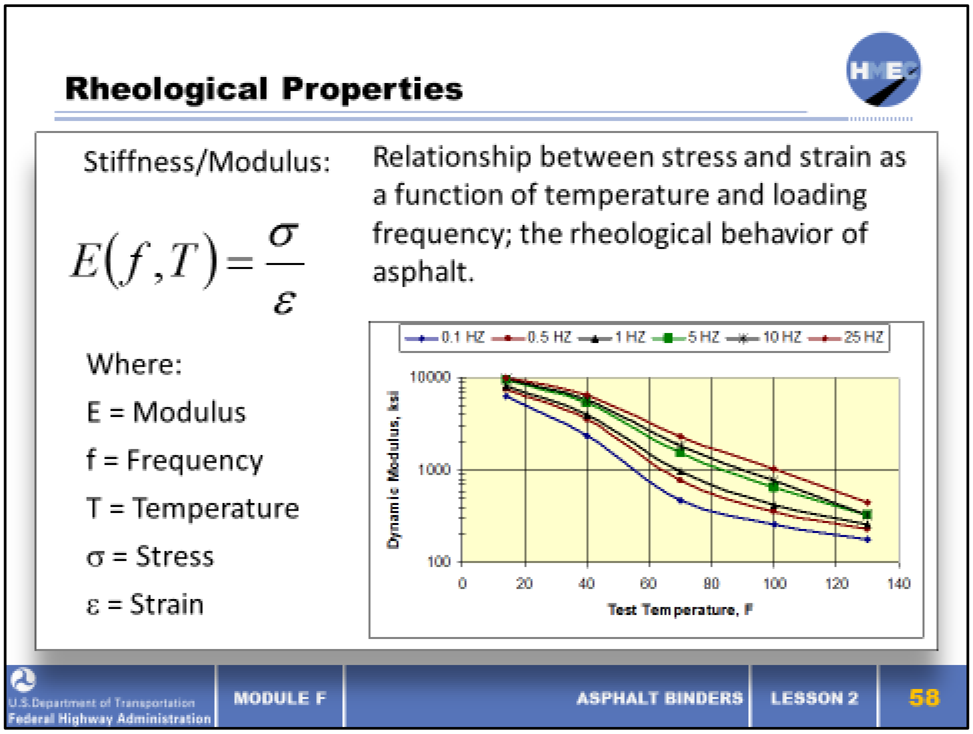
A thermoplastic material is defined as a material that softens when heated and hardens again when cooled. The rheological properties change significantly during the production process and over time. As such, age hardening is considered to be one of the most important factors, but will be discussed in a latter topic under Lesson 2.

Slide 57



Some materials, when subjected to stress, will flow and continue to do so with the stress at a constant level. Moreover, when the stress is removed, the deformation will not be recoverable. In addition, when such materials are loaded at different rates, they will develop different levels of stress, depending on the rate of load application. Water is an example of such a material. These materials are referred to as viscous materials. Shear stress is related to shear strain rate by the coefficient of viscosity (η) in these materials.

Slide 58



It is shown that indeed due to its viscoelastic nature, the stiffness of an asphalt binder is a function of loading time and temperature. The chart illustrates the stiffness of asphalt binders as a function of temperature (i.e., temperature susceptibility) in a very simplified manner.

At low temperatures, asphalt binders are much stiffer than at high temperatures. Binders with steep instantaneous slopes at a given temperature are more temperature susceptible than are binders with gentle instantaneous slopes at the same temperature.

The stiffness of an asphalt binder is a function of both time of loading and temperature. At very short loading times, or at low temperatures, the stiffness of the binder will approach the elastic or shear modulus (i.e., linear relationship between stress and strain).

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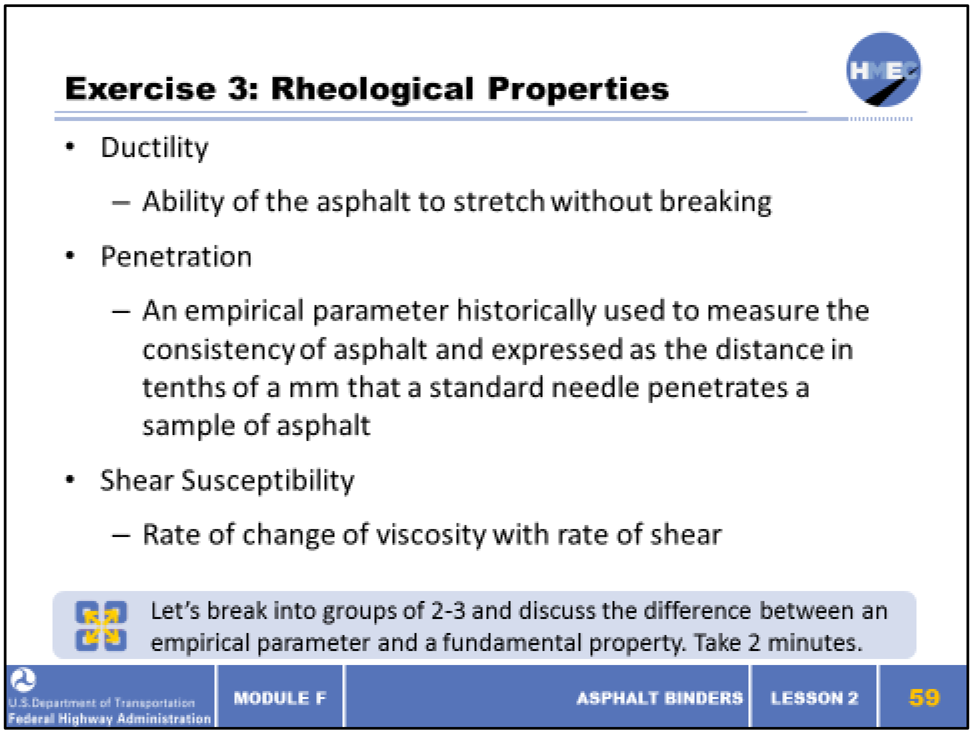
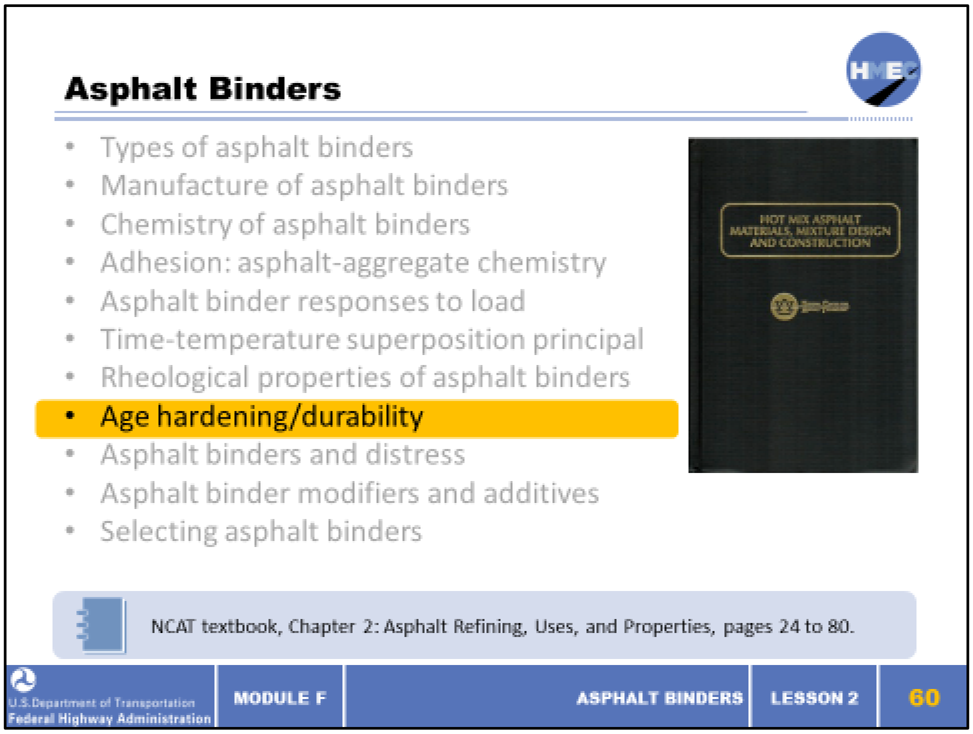
Slide 59



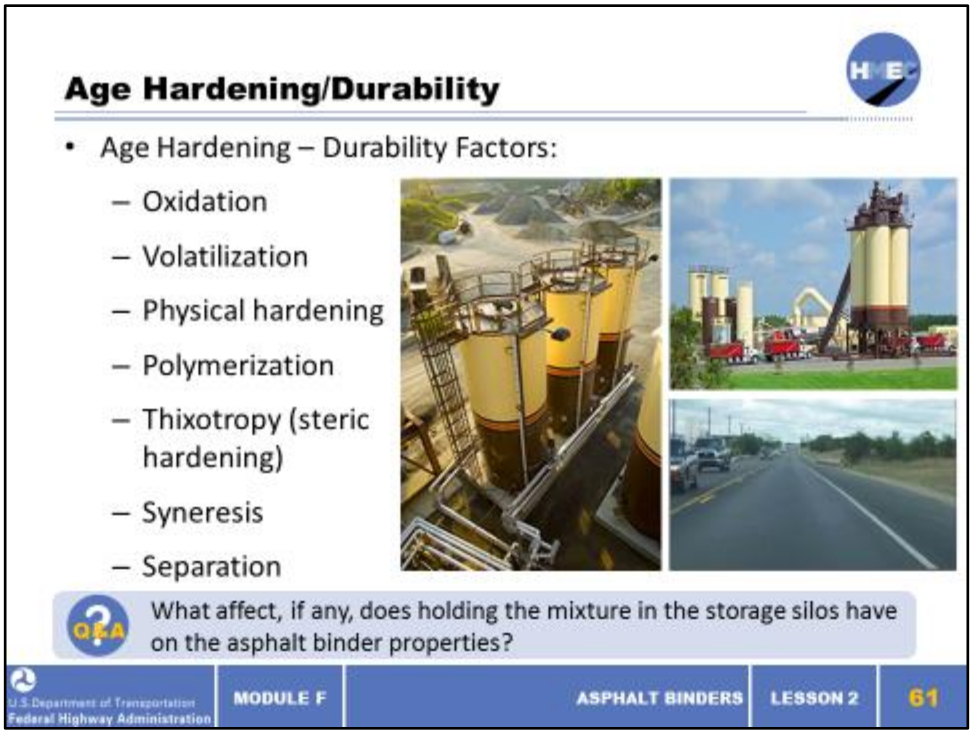
The other item to note and emphasize is that penetration is an empirical parameter and not considered a fundamental property. It is included under this topic because it is listed as a rheological property in the NCAT textbook and has been historically used in specifications and to select binders for specific areas or climates. However, penetration is no longer being used for either purpose by any agency in the US.

Slide 60

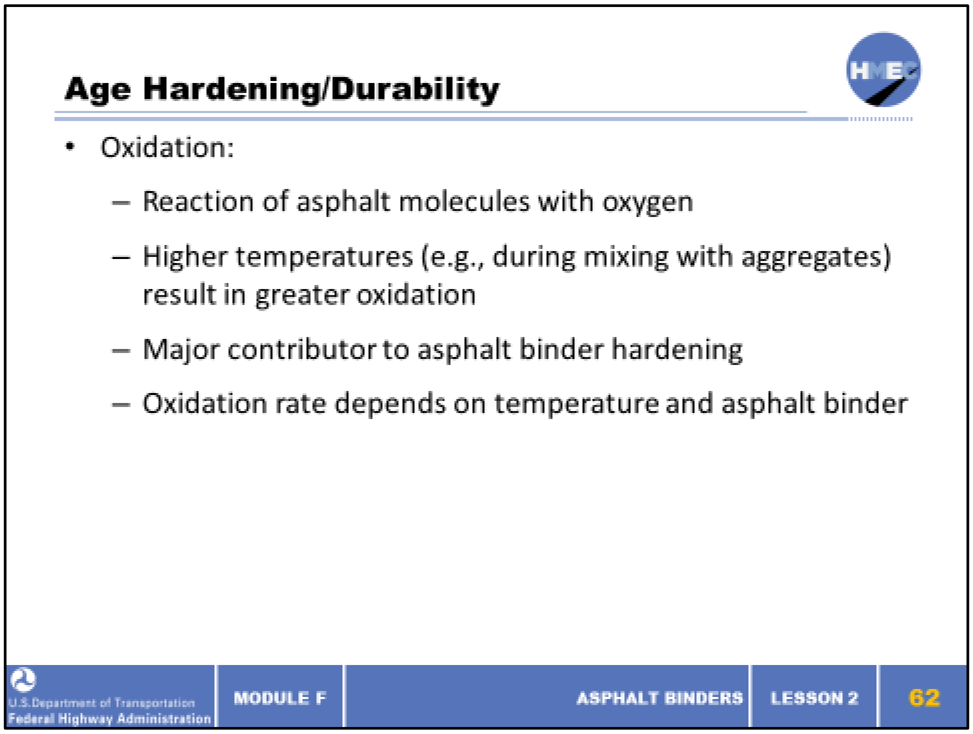


As such, age hardening is considered to be one of the most important factors.

Slide 61


Age Hardening/Durability
HMEC
• Age Hardening – Durability Factors:
– Oxidation
– Volatilization
– Physical hardening
– Polymerization
– Thixotropy (steric hardening)
– Syneresis
– Separation
Q&A
What affect, if any, does holding the mixture in the storage silos have on the asphalt binder properties?
U.S. Department of Transportation Federal Highway Administration
MODULE F
ASPHALT BINDERS
LESSON 2
61

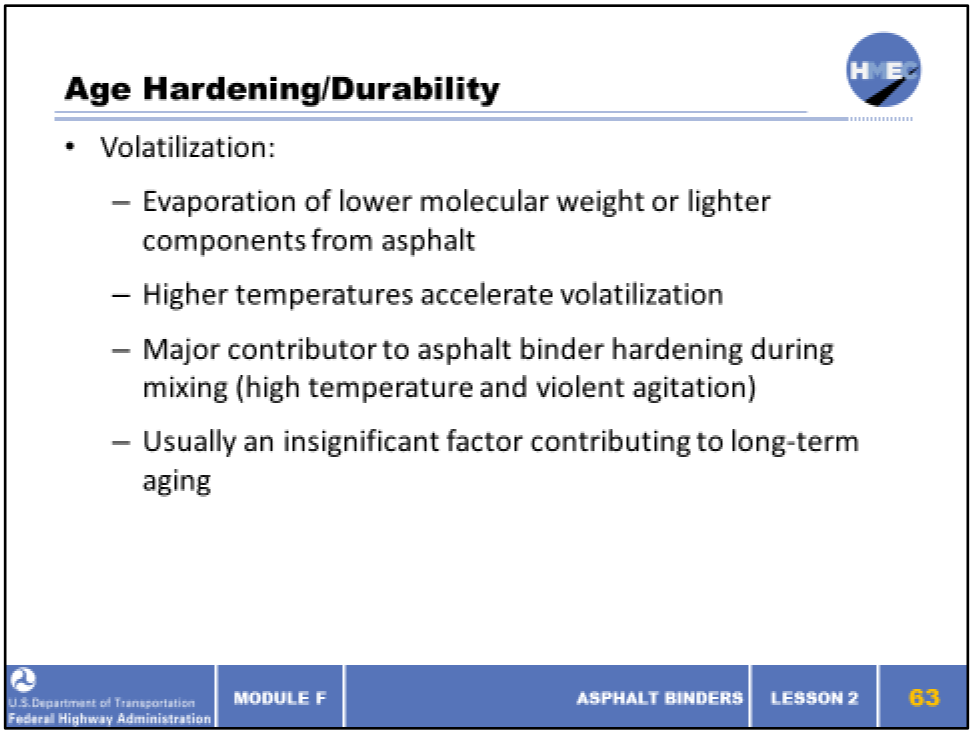
Slide 62



At normal temperatures, the reaction is a slow process and the asphalt binder primarily absorbs the oxygen. In this process, a layer of hard material is formed at the surface of the asphalt binder that, if left undisturbed, will prevent further reaction of oxygen in the material. If the film is cracked, however, new surfaces of asphalt will be exposed which, in turn, will permit additional oxidation to occur.

Oxidation is believed to be one of the primary causes of hardening of asphalt binders in the pavement. Oxidation of asphalt binders at temperatures associated with the mixing of asphalt binders with aggregates is rapid. The amount of hardening that occurs during the process is a function not only of temperature but also the thickness of asphalt binder film, time of exposure, and type of atmosphere (oxygen rich or oxygen depleted).

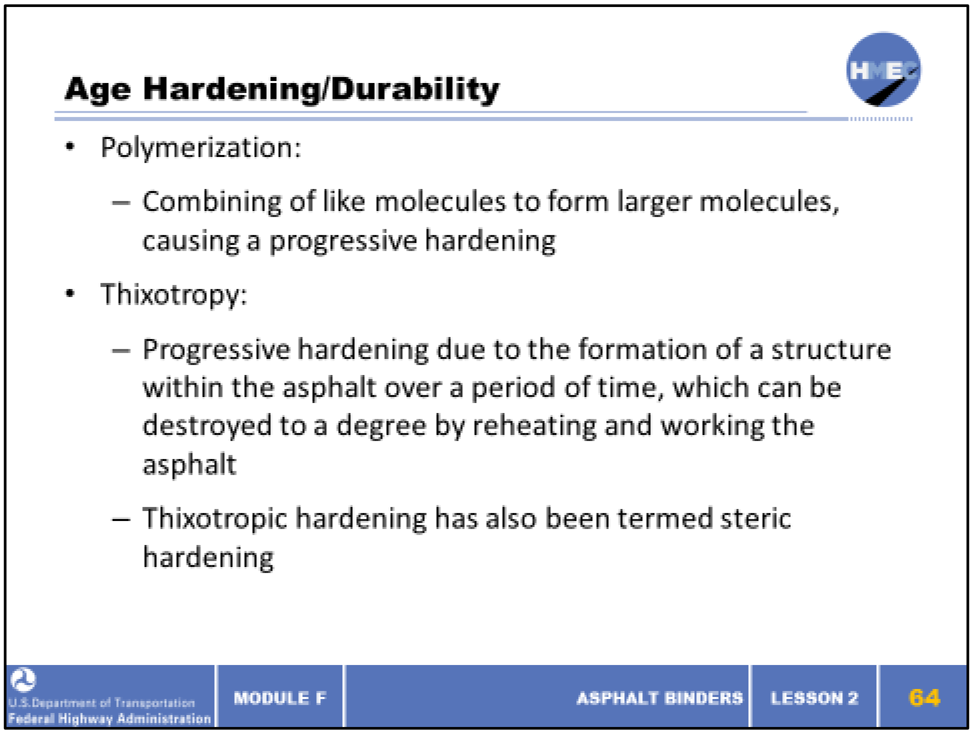
Slide 63



Recall that an asphalt binder is a mixture of hydrocarbons with a wide range in molecular weight. Increased temperature will accelerate volatilization.

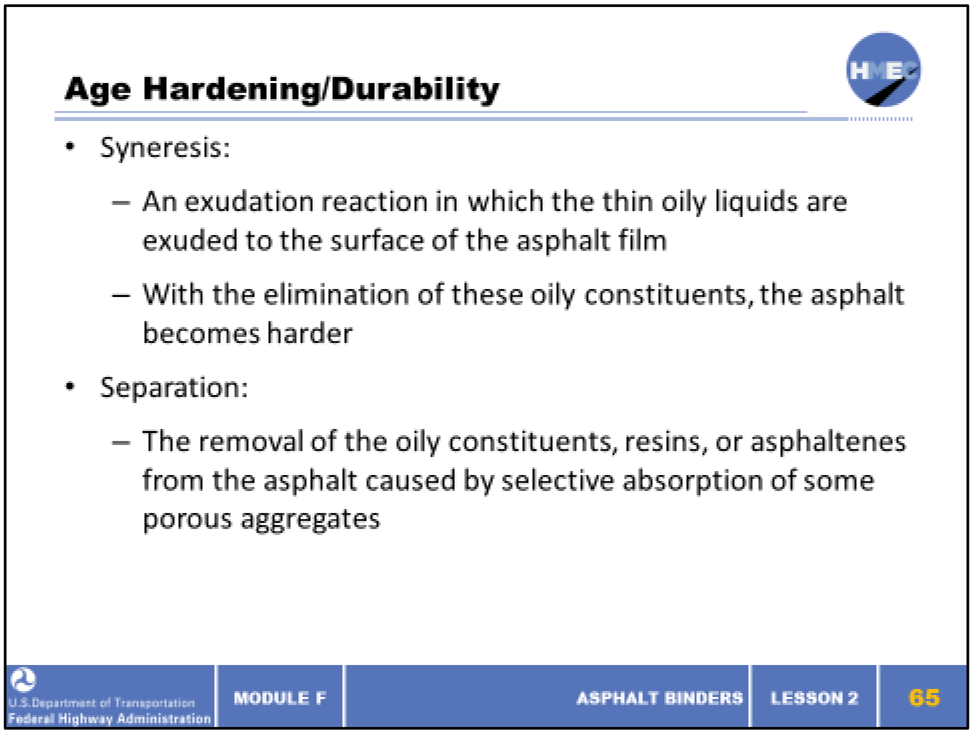
During the mixing process, where high temperature is combined with violent agitation, volatilization is one of the primary reasons for hardening of an asphalt binder. Crude source and refining procedure appear to, in part, affect the degree to which volatilization progresses. Hardening due to volatilization is primarily due to the loss of molecular weight components, below molecular weights of about 400.

Slide 64



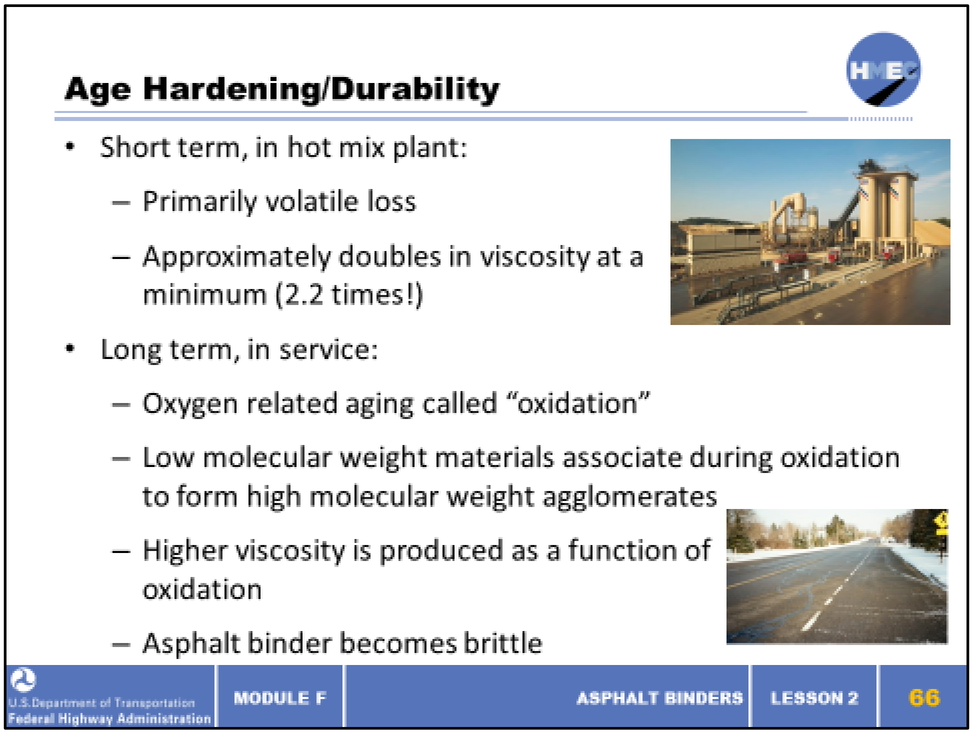
By reheating the asphalt binder, the structure rearranged or changed back to its original condition by reheating. In other words, steric hardening is reversible.

Slide 65



Both of these relate to the removal of the lighter components from the asphalt. The difference is that separation relates to amounts absorbed into voids of porous aggregates, while syneresis are exuded to the surface of the asphalt film, which hardens the asphalt.

Slide 66

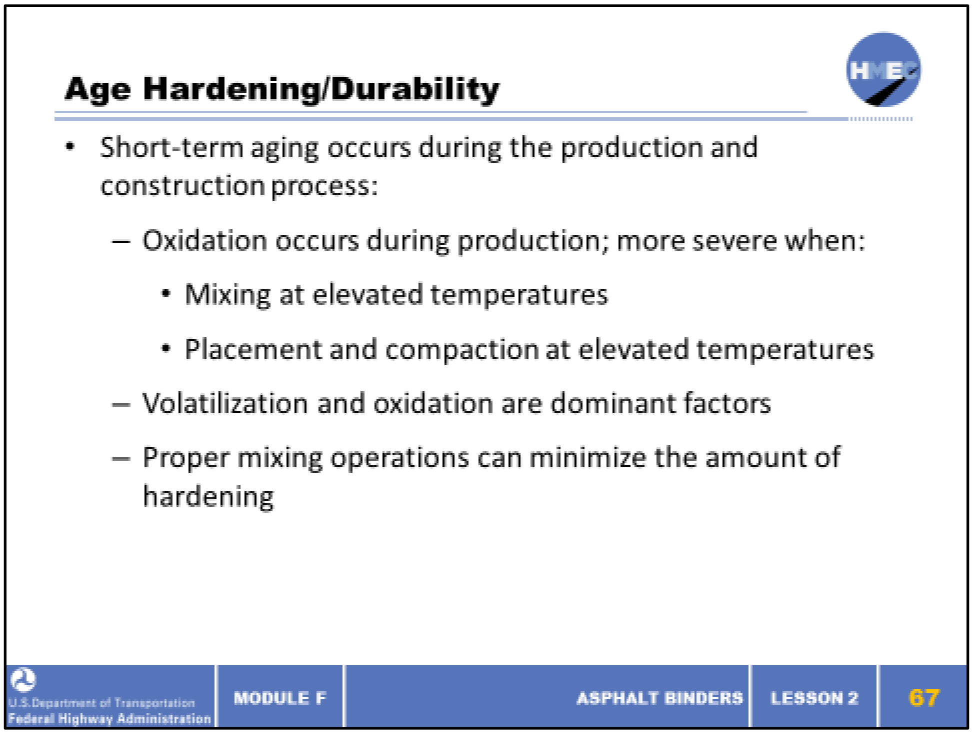


• Asphaltenes are the component of asphalt primarily responsible for the viscosity (consistency) of the asphalt.
• Maltenes are the oily medium that act as dispersants (peptizers) for the asphaltenes.

As asphalt ages, more asphaltenes are formed (large molecular weight agglomerate) due to increasing associations formed during oxidation. Asphaltenes are hard brittle solids when isolated and consist primarily of hydrocarbons.

Maltenes are comprised of oils and resins.

Slide 67

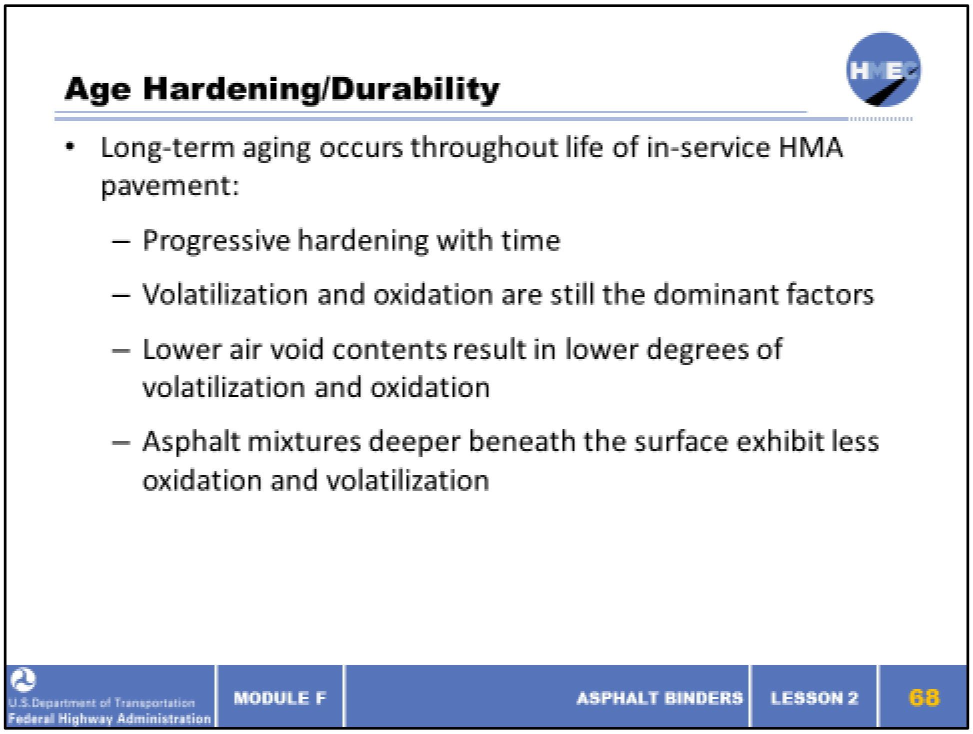


Violent agitation during the mixing process continuously exposes the asphalt binder to oxygen resulting in oxidation and it also accelerates volatilization.

Oxidation and volatilization can also occur during transport of the mixture but is likely to be limited only to the mix exposed to oxygen (i.e., at the surface of the load). Improper control of the mixing operation may result in considerable damage occurring in a short period (say 30 to 35 seconds), which could shorten the pavement life considerably.

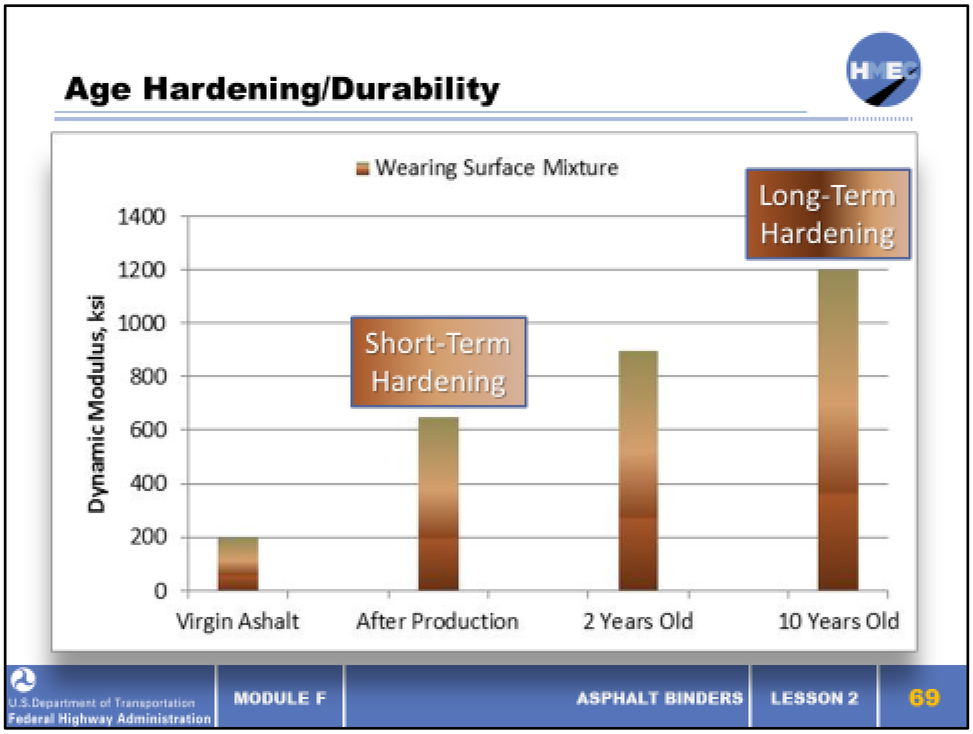
During laydown and compaction, the mixture is again agitated which again exposes the asphalt binder to oxygen resulting in oxidation and because the mixture is at an elevated temperature during compaction, volatilization can occur.

Slide 68



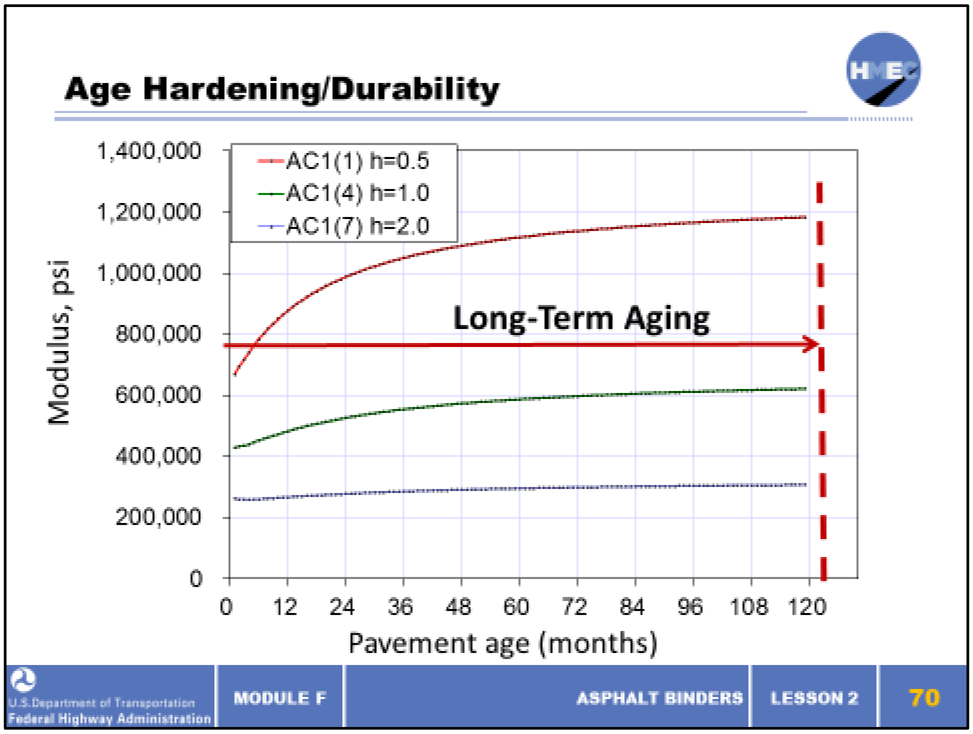
Volatilization and oxidation are the principal mechanisms of long-term aging, but to a lesser extent than that occurring in short-term aging. Mixtures compacted to lower air void contents are more resistant to this form of aging than are mixtures with high air void contents.

Slide 69



The increase in stiffness is often accompanied by brittleness that leads to a gradual deterioration of asphalt pavements.

Slide 70



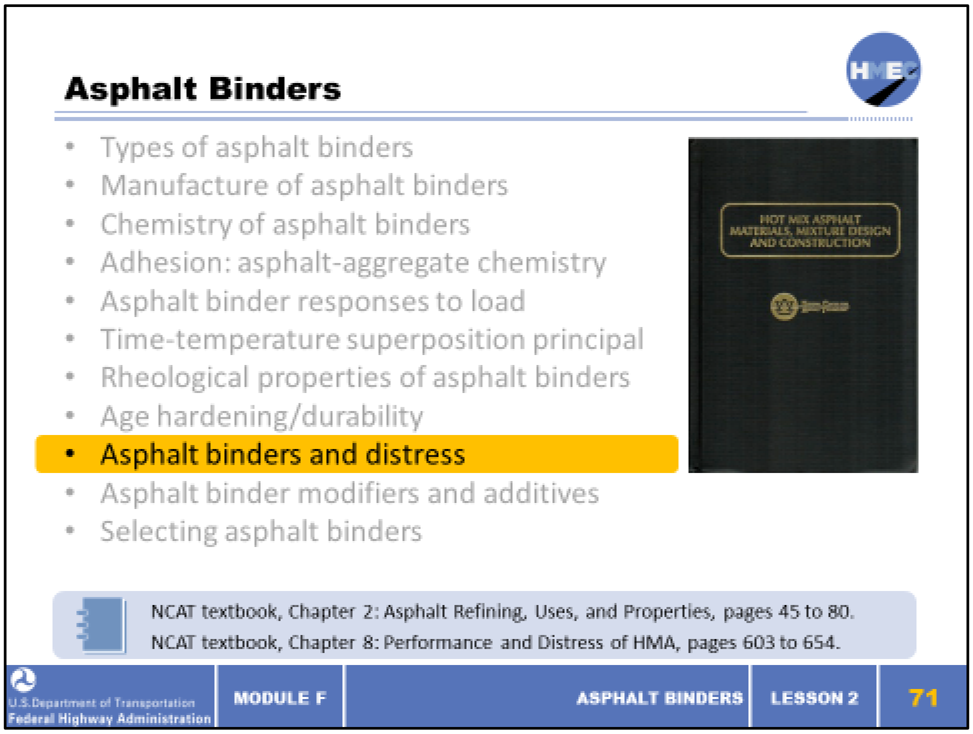
Here you can see the effects of aging. The top layer (1/2”) of the pavement ages much more than the bottom of the pavement structure. Also you can see that the effect of aging is much quicker in the early years of the pavement versus the later years. The aging model used in the MEPDG is called the Witczak–Mirza aging model. It uses binder and volumetric properties of the materials in the aging predictions.

The key point is that different asphalts age differently, and you need to consider differences for your particular application.

The other key point is that there is still debate on what short-term is supposed to simulate: production and construction with or without some service life.

Long-term aging has been defined as 10 years out in the service life, but the long-term aging process used in the laboratory does not distinguish between wearing surfaces and underlying HMA base layers. The long-term aging procedures were developed for the wearing surfaces or layer at or near the surface.

Slide 71


Asphalt Binders
HMEC
Types of asphalt binders
Manufacture of asphalt binders
Chemistry of asphalt binders
Adhesion: asphalt-aggregate chemistry
Asphalt binder responses to load
Time-temperature superposition principal
Rheological properties of asphalt binders
Age hardening/durability
Asphalt binders and distress
Asphalt binder modifiers and additives
Selecting asphalt binders
HOT MIX ASPHALT MATERIALS, MIXTURE DESIGN AND CONSTRUCTION
NCAT textbook, Chapter 2: Asphalt Refining, Uses, and Properties, pages 45 to 80.
NCAT textbook, Chapter 8: Performance and Distress of HMA, pages 603 to 654.
U.S. Department of Transportation
Federal Highway Administration
MODULE F
ASPHALT BINDERS
LESSON 2
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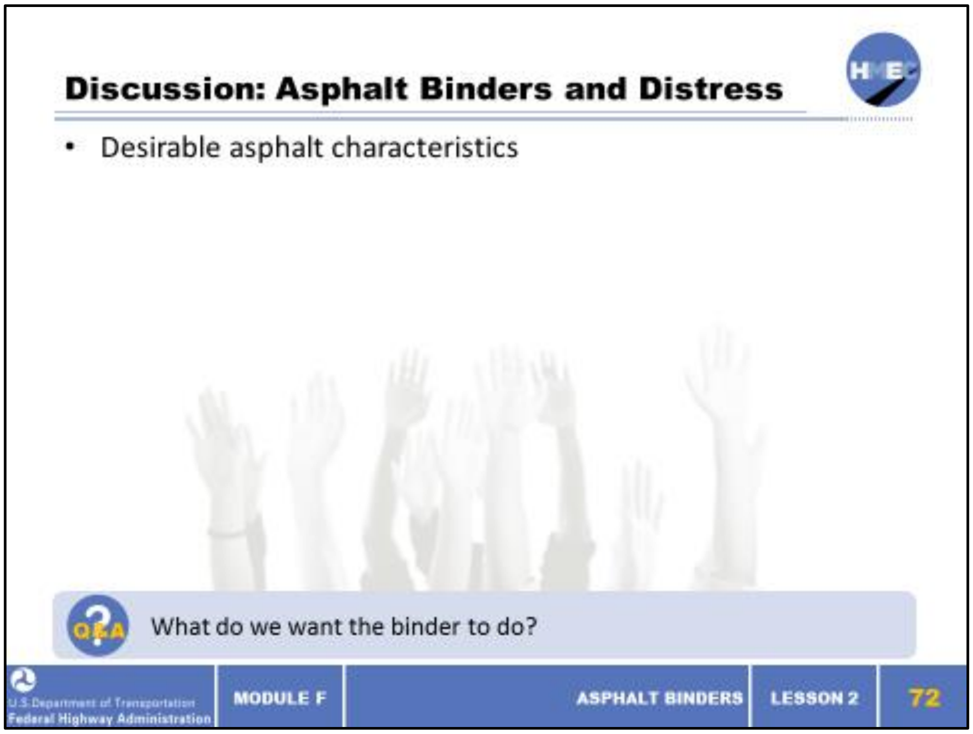
Slide 72



There are some key characteristics that we need asphalt binders to possess.

Slide 73

High stiffness is desirable early in life and right after construction to minimize rutting.

Slide 74

Asphalt Binders and Distress

- Pavement performance and rheological properties; importance of asphalt binder to:
  - Rutting
  - Alligator fatigue cracking
  - Low temperature cracking
  - Raveling
  - Bleeding/flushing

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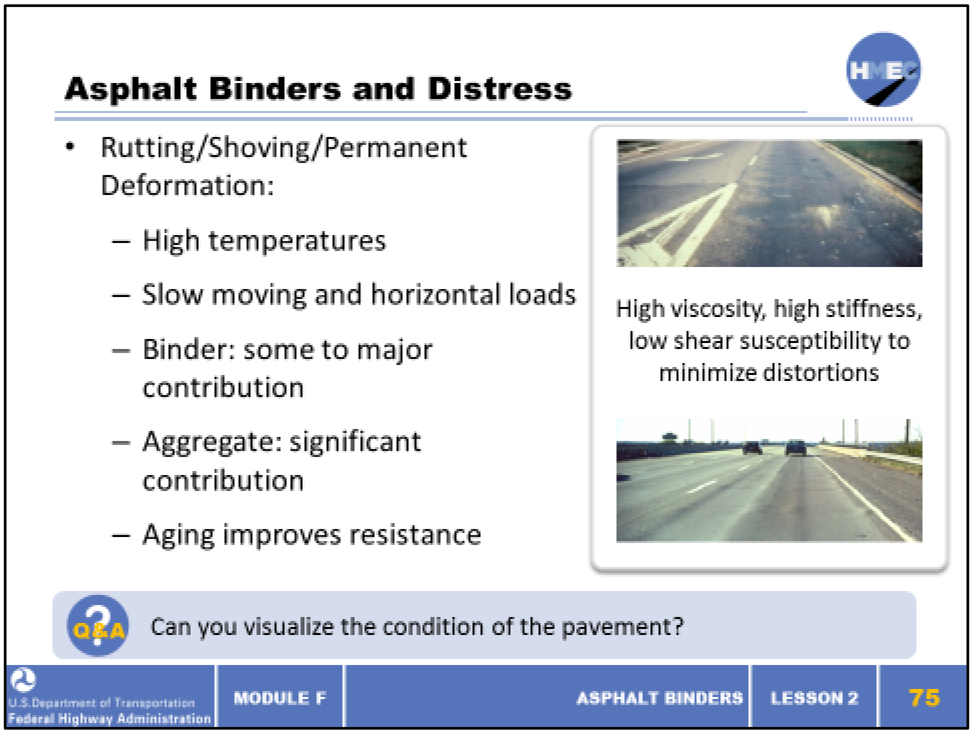
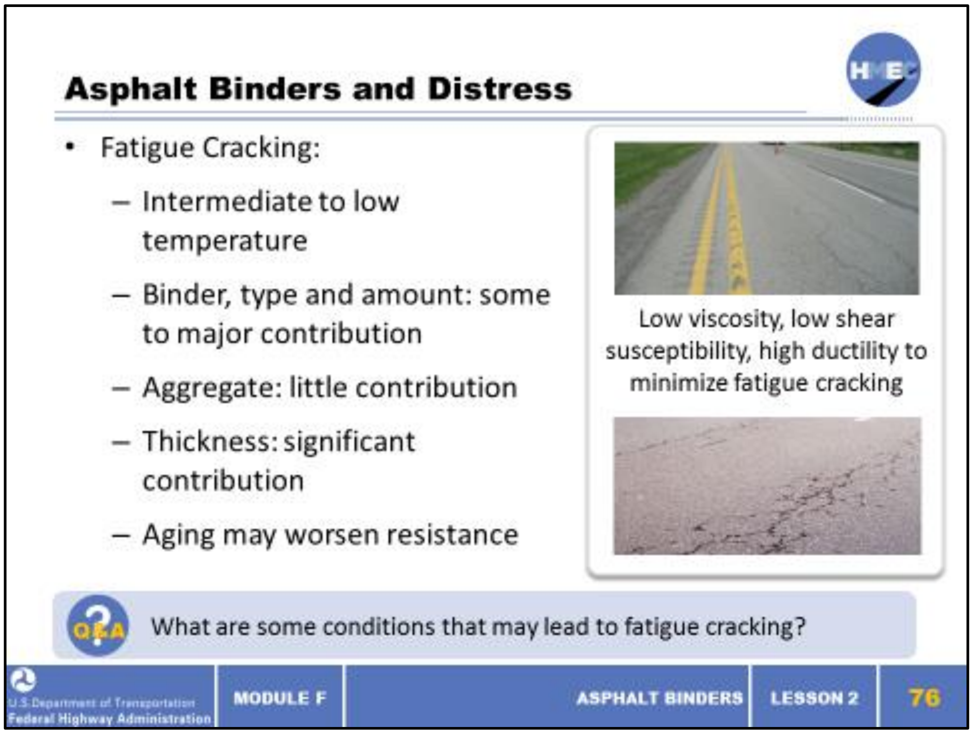
Slide 75



While the asphalt binder contributes to resistance to permanent deformation, aggregate characteristics (i.e., surface texture, angularity, etc.) and aggregate gradation play a more important role. Aging of the asphalt binder improves the resistance of HMA mixtures to permanent deformation.

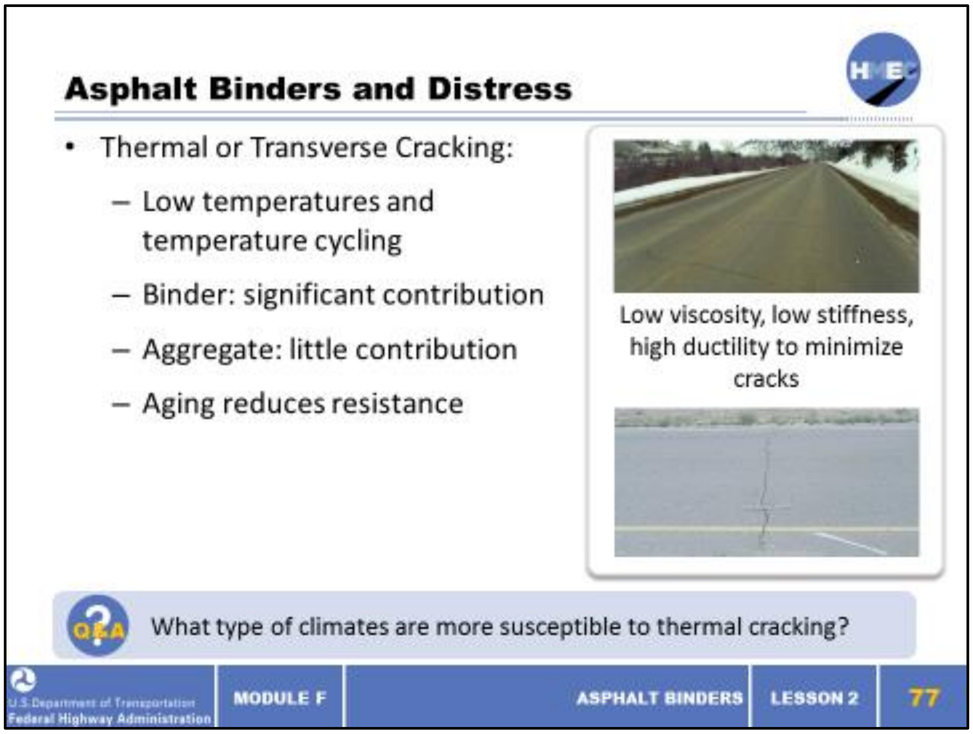
Slide 76



Thus, asphalt binder properties play a more significant role in fatigue cracking resistance than do aggregate characteristics and gradation. However, the HMA pavement layer thickness plays the most significant role because thicker layers provide greater load spreading capability, which, in turn, lowers the tensile strains developed under loading.

Although age hardening results in a stiffer binder and, thus, improved load spreading characteristics of the HMA pavement layer, it may worsen resistance to fatigue cracking by virtue of the binder becoming less viscous.

Slide 77



Thermal cracking may also occur through temperature cycles that approach the limiting temperature but do not reach it causing accumulation of permanent strains that eventually leads to fracture. In both cases the behavior of the binder at low temperatures contributes significantly to thermal cracking resistance whereas aggregate characteristics and gradation have little influence. Aging reduces the resistance of an HMA mixture to thermal cracking by virtue of the binder becoming less viscous.

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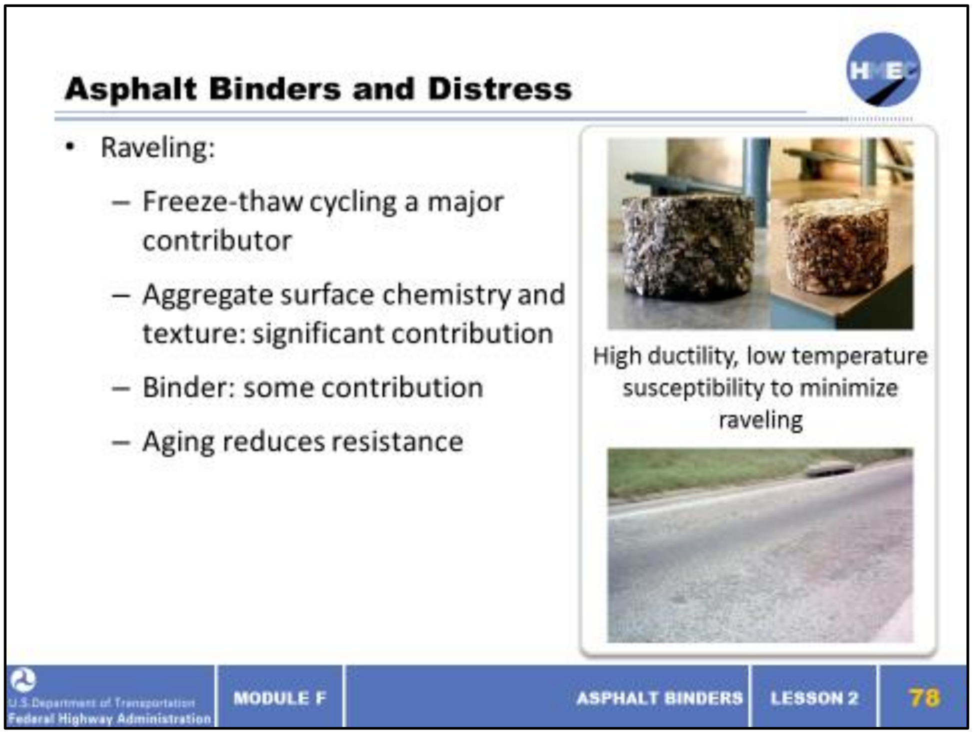
Slide 78



Moisture damage can also be a key factor causing raveling because it results from the failure of the adhesion (or bond) between the asphalt binder and the mineral aggregate (referred to as stripping) or through loss of cohesion within the mixture, both of which are accelerated by freeze-thaw cycling. Moisture damage and stripping, however, are not pavement distresses. The distress that is associated mostly with these mechanisms is raveling. Cracking is another distress that can be accelerated through these mechanisms. As noted above, adhesion is the strength or bond between the asphalt and aggregate and is an important property or characteristic of the mix. Cohesion is the strength of the asphalt binder. The cohesion can be reduced by moisture and/or contamination of the asphalt binder.

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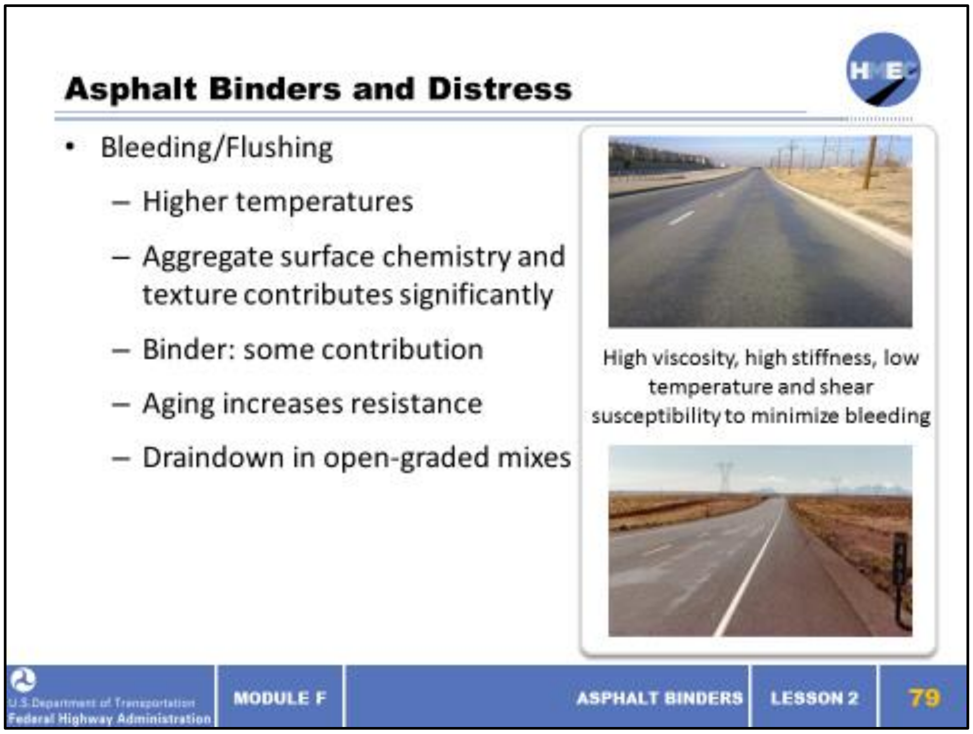
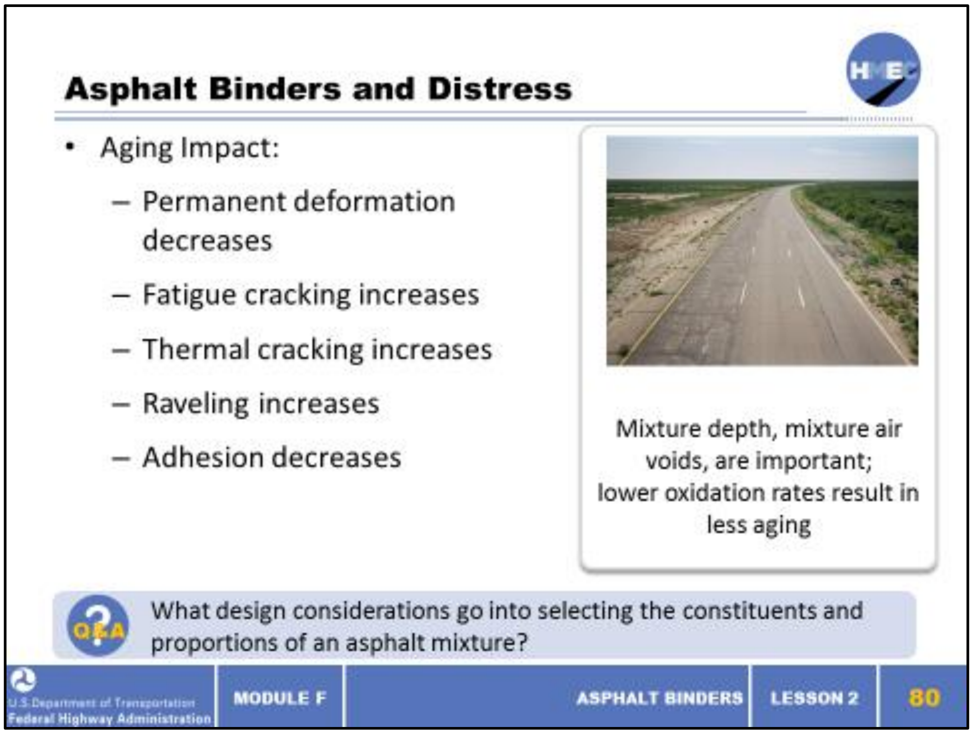
Slide 79



Bleeding is also sometimes caused by poor mix design (too much binder), low air voids, synersis, and draindown in open-graded mixtures.

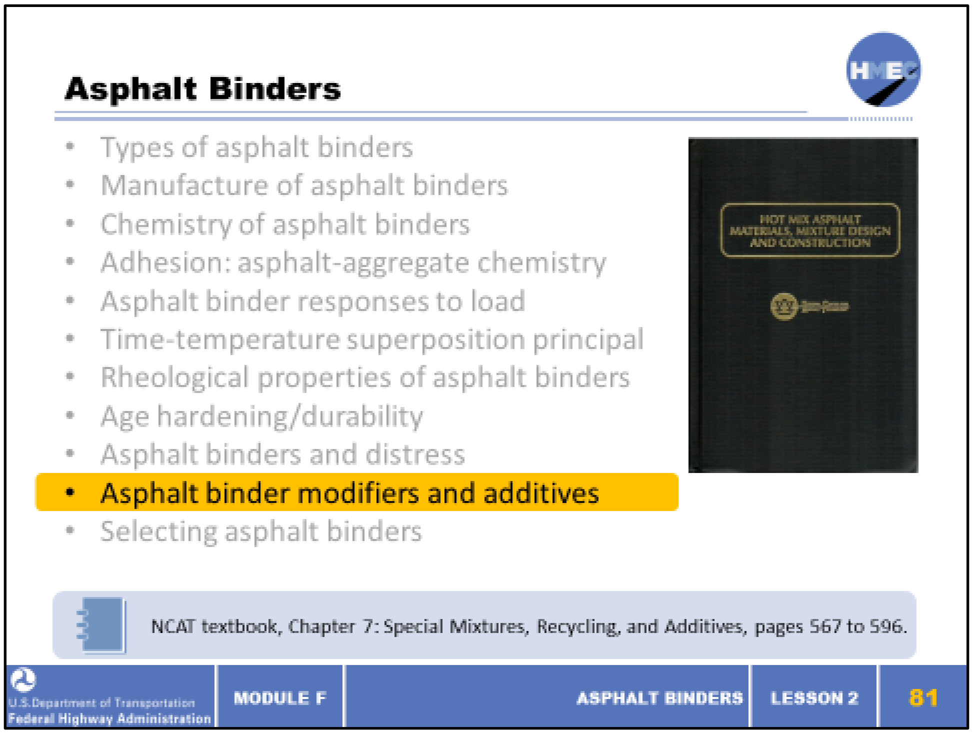
Too-high temperatures can create excessive bleeding, but in combination with a loss of bond between the asphalt and aggregate can accelerate this process of bleeding. In some cases, the presence of moisture in the mixture or moving through the mixture during hot summer days can bring asphalt to the surface and result in bleeding as the water vaporizes and tries to exit the mixture through air void paths to the surface.

Slide 80



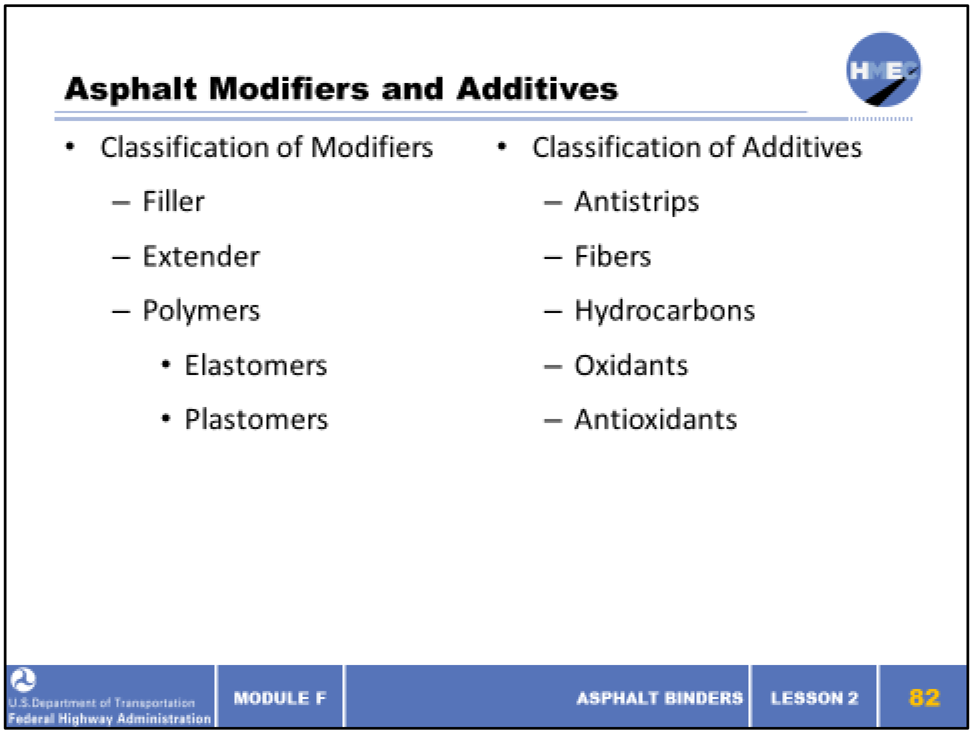
Aging of the asphalt binder impacts the resistance of the HMA mixture to permanent deformation, fatigue cracking, thermal cracking, and moisture sensitivity. Proper prediction and testing for the aging characteristics can prevent these distresses.

Slide 81



The concept of modifying asphalt binders is not new, but it has received renewed interest in the last two decades.

Slide 82



This slide shows how the modifiers and additives are classified based on their previous use and definition in the literature. The majority of the modifiers used in practice are polymers, while the majority of additives used are antistrips and fibers. Fibers are basically used to prevent drain down in special mixtures like stone matrix asphalt (SMA). Both additives and modifiers will also be discussed in the lesson on HMA mixture design.

Slide 83


Discussion: Asphalt Modifiers and Additives
HMEC
Is your agency using modifiers? If so, why and which types are used?
U.S. Department of Transportation
Federal Highway Administration
MODULE F
ASPHALT BINDERS
LESSON 2
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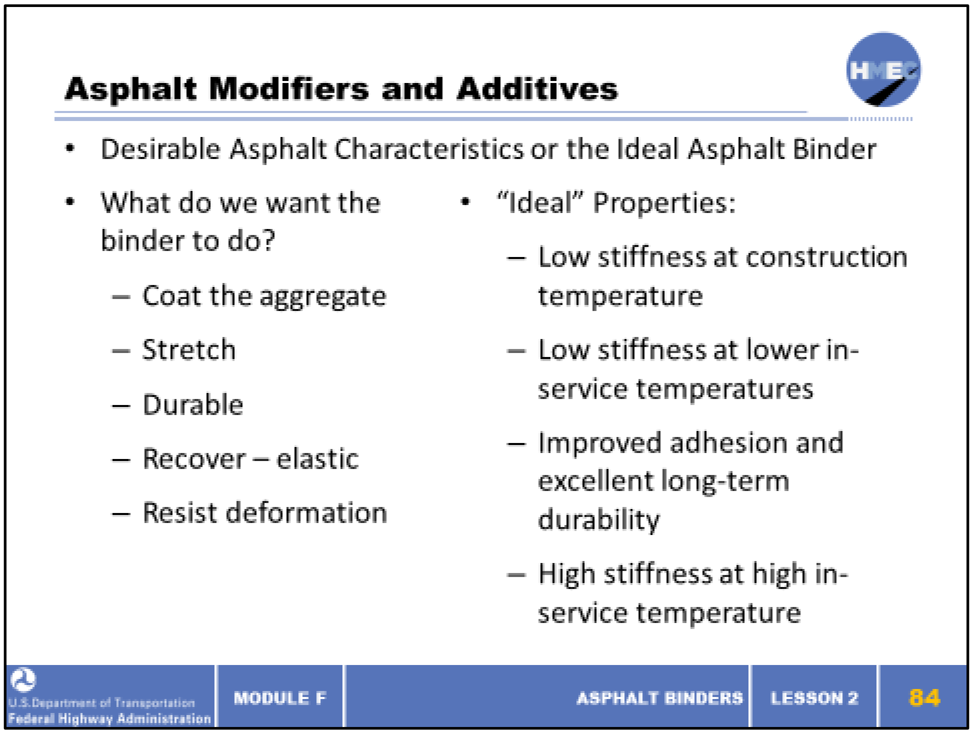
Slide 84

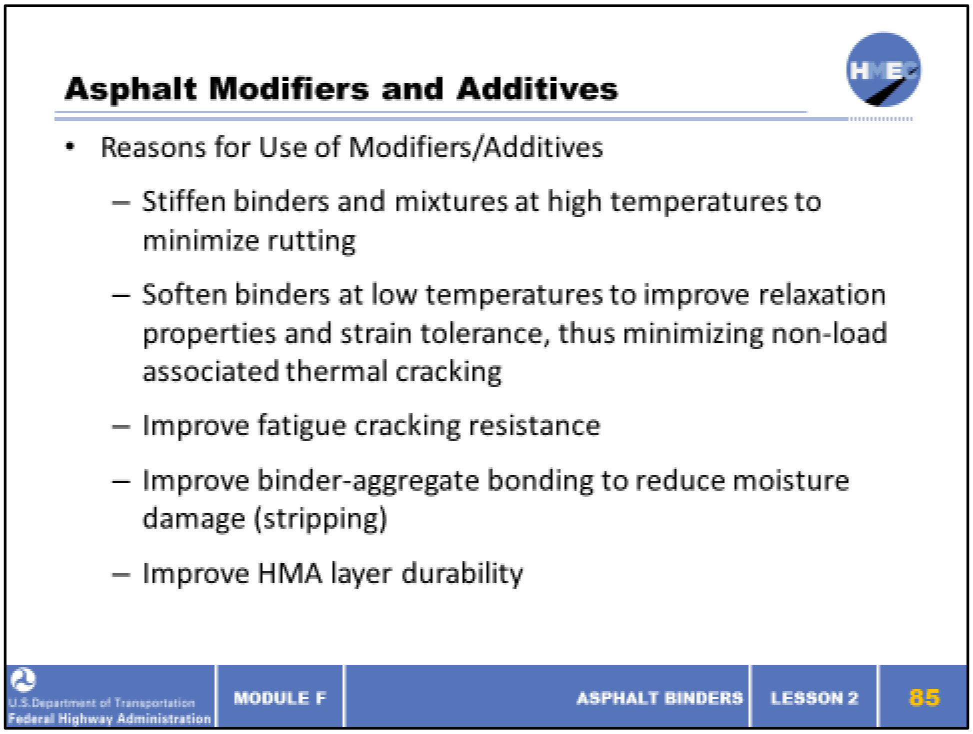


The ideal binder for HMA mixtures should have low stiffness at the time of construction and later in life at low temperatures.

High stiffness is desirable early in life and right after construction to minimize rutting.

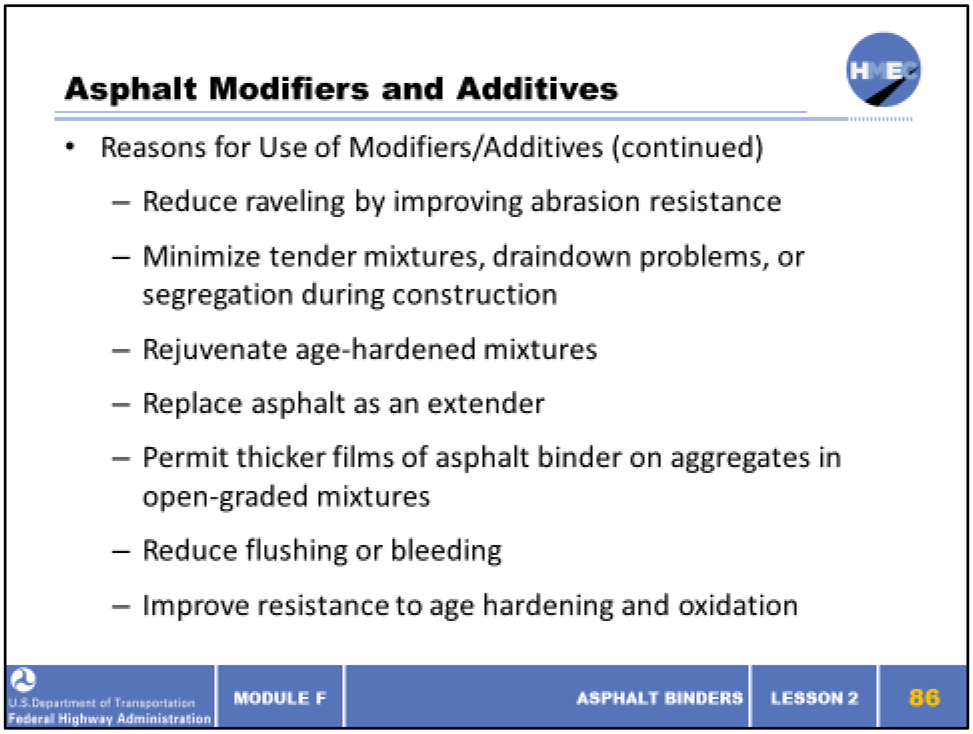
Locally available binders, however, may not provide the expected performance for some severe and site specific conditions.

Slide 85



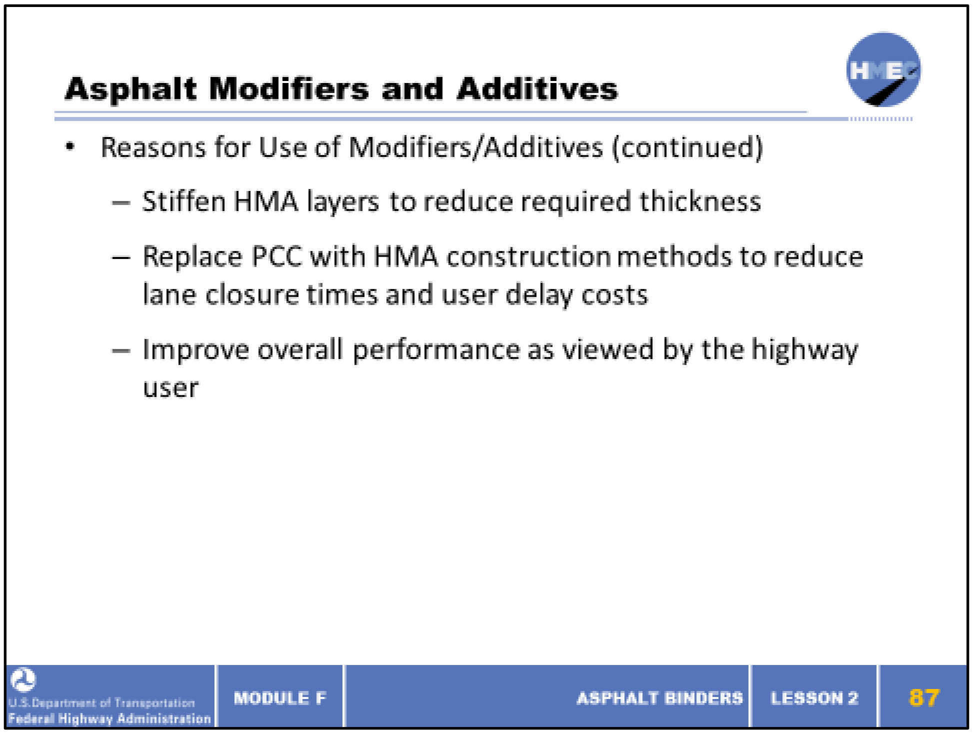
Asphalt binders are modified using polymers for improving resistance to rutting, fatigue cracking, thermal cracking, and moisture damage.

Slide 86



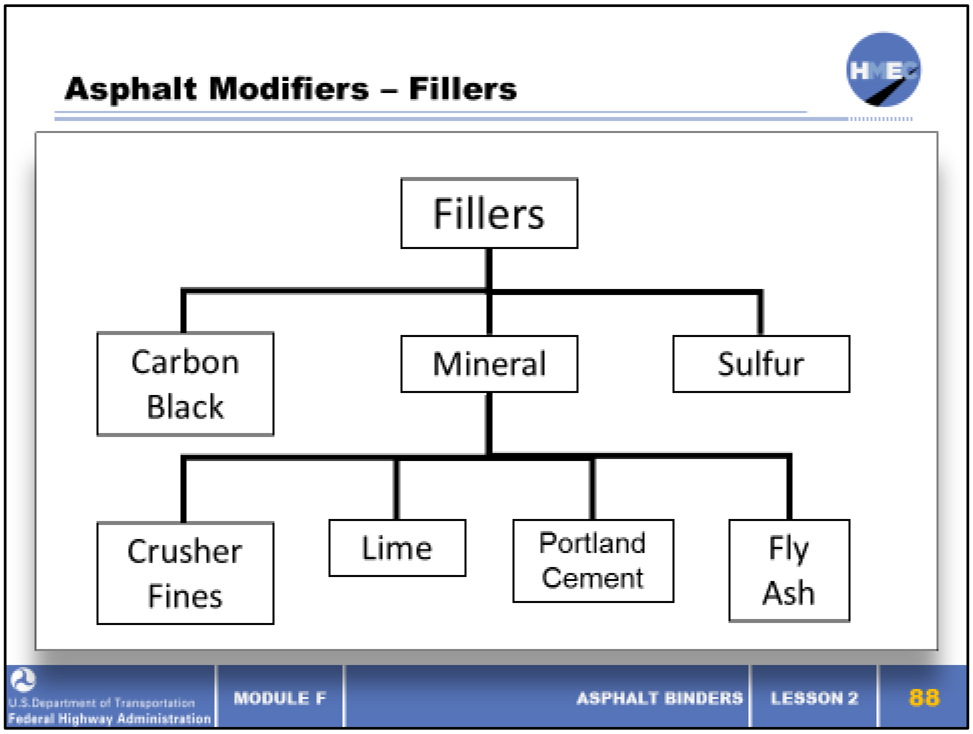
Reasons for polymer modification include improving resistance to raveling, improving coating characteristics during construction, rejuvenating age-hardened pavements, and replacing asphalt as an extender. Other reasons for polymer modification shown in this slide include: improving resistance to flushing, bleeding, and age-hardening, and permitting thicker binder films on aggregates. In some cases, polymer modification does provide a much stiffer mixture. With stiffer mixtures most design procedures will require less thickness, so that HMA layer thickness can be reduced—but, it is mixture dependent.

Slide 87



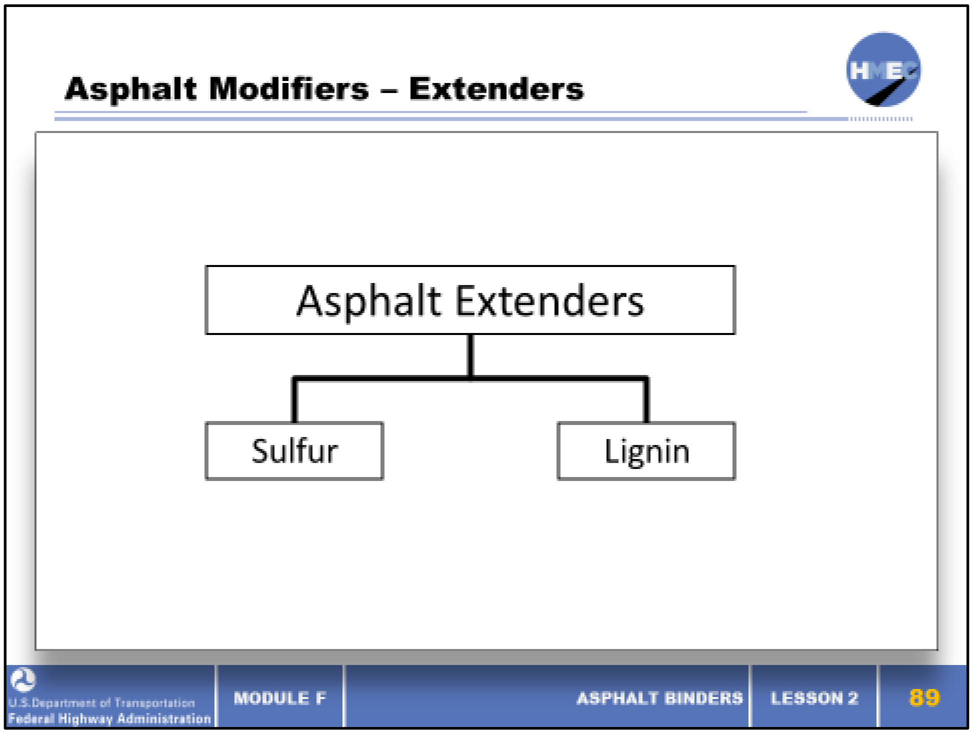
Reasons for polymer modification shown on this slide include improving the durability characteristics of HMA mixtures, using HMA (with a modified binder) in lieu of Portland cement concrete to reduce construction time, and to improve the overall performance as viewed by the road user.

Slide 88



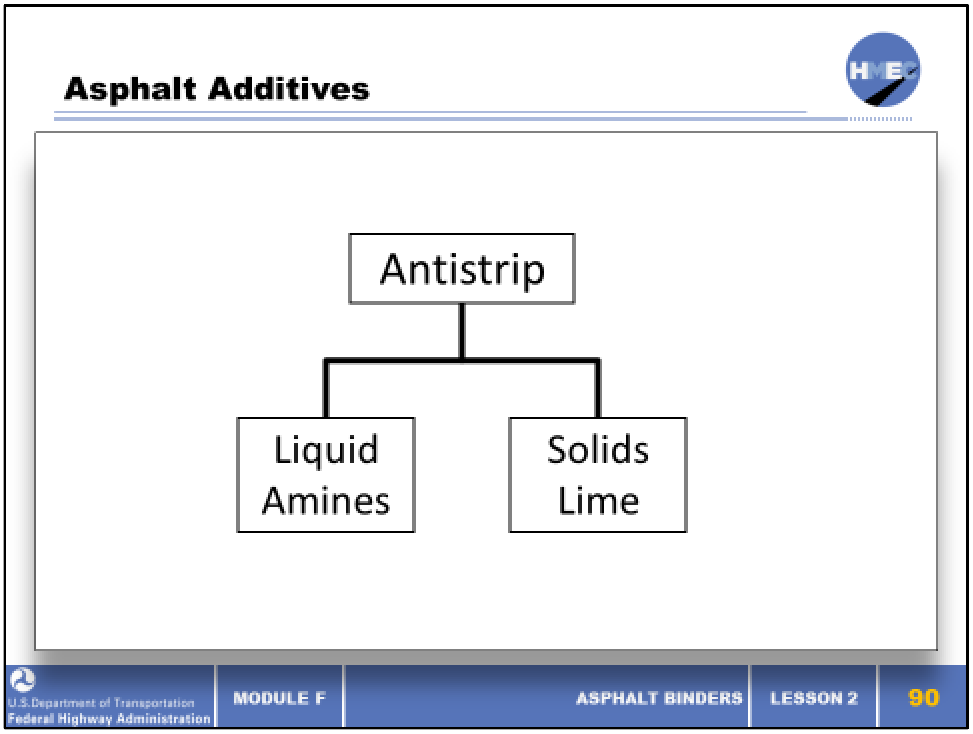
Mineral fillers include mineral dust from crushing and screening of aggregates, fly ash, Portland cement, and lime. Other materials such as sulfur and carbon black have been used more recently to improve mixtures. These materials may not be strictly mineral fillers, as they have special binding qualities in addition to the role as a filler. Fillers have been traditionally used is HMA mixtures for the purpose of providing stiffening or reinforcement to the binder as well as "filling in" the voids in the aggregates.

Slide 89



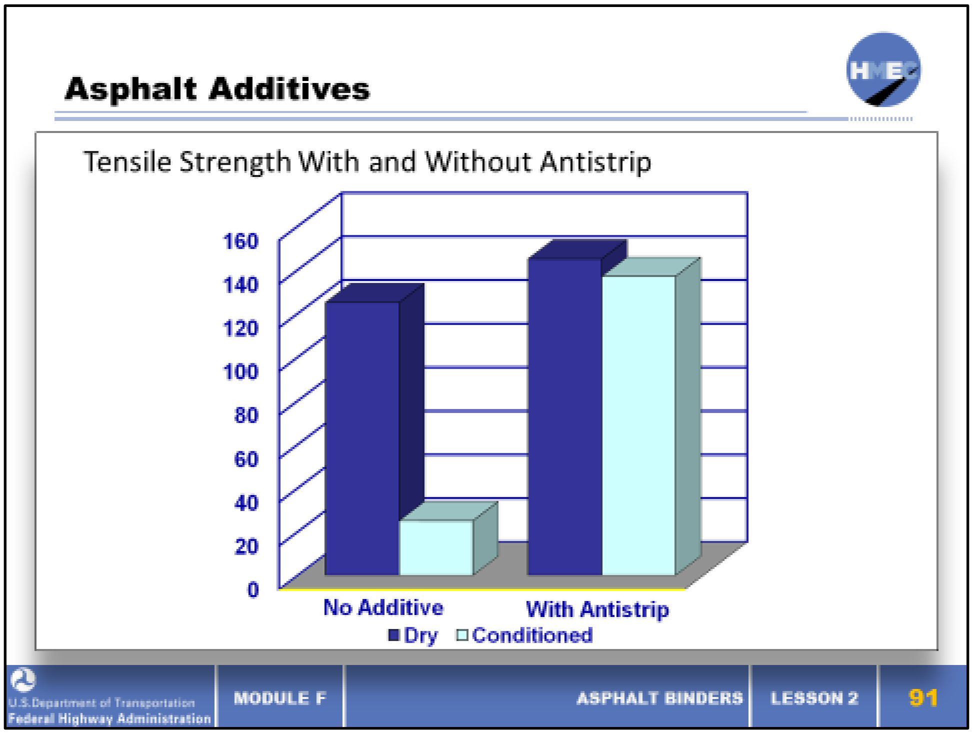
The intent is often to replace some of the asphalt binder with a material that is less costly but also has binding qualities by itself or in combination with an asphalt binder. Further, some extender materials may provide other benefits, such as reduced oxidation rates.

Slide 90



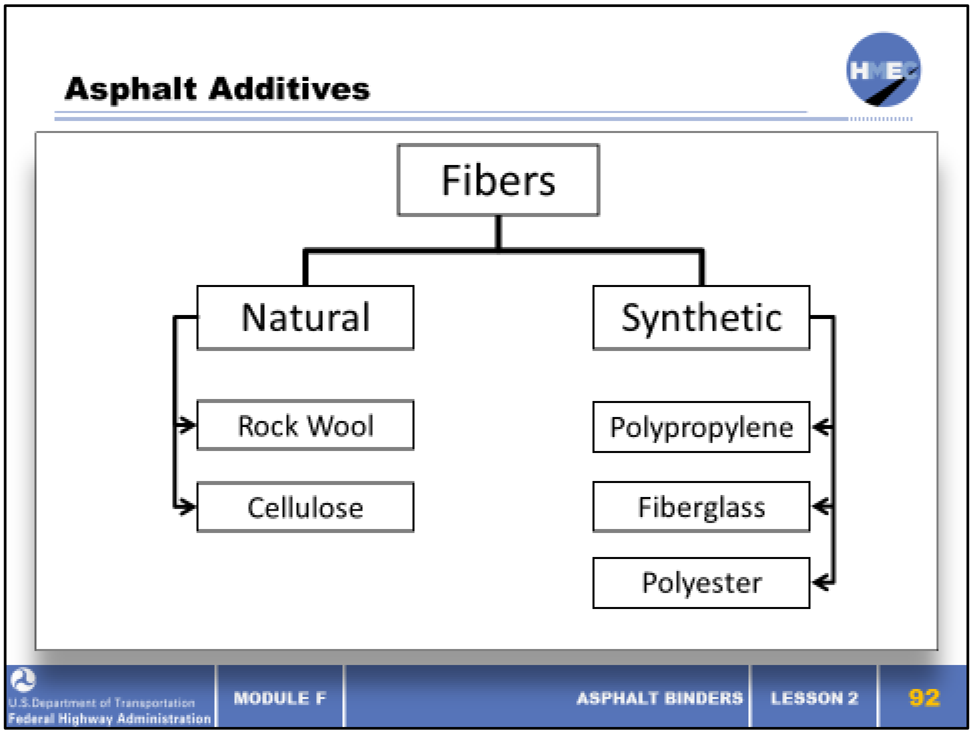
Antistrips are the more common additive used in asphalt mixtures and are used for a specific purpose. Some asphalt-aggregate combinations perform very well while others are notoriously bad strippers. Chemical materials improve the adhesion between asphalt and aggregate in the presence of water. Small amounts of chemicals such as amines or lime, added to either the asphalt binder or aggregate or mixture, often tend to reduce stripping. The use of antistrips is also discussed in greater detail in the lesson on asphalt mixture designs to ensure durable mixtures over time.

Slide 91



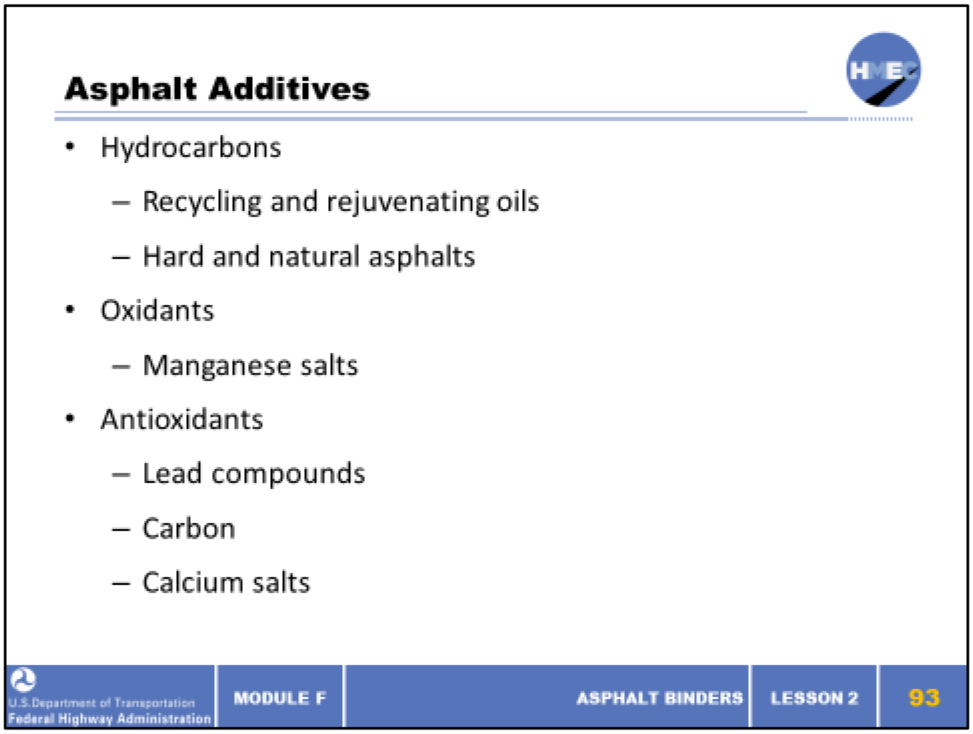
The “dry” test results were from samples that had not been moisture conditioned whereas the “conditioned” results were from samples subjected to moisture conditioning. The retained strength after conditioning of mixture with liquid antistrip chemical was substantially higher than that of the mixture without antistrip chemical.

Slide 92



The usual approach is to incorporate very fine, short fibers into the binder (usually conventional asphalt binders) or mixture, depending upon its form, chemistry, and intended function. The fibers provide some reinforcement, but also provide finely divided material with high surface area that permits the application of thicker-than-normal films of asphalt binder on the aggregate.

Slide 93

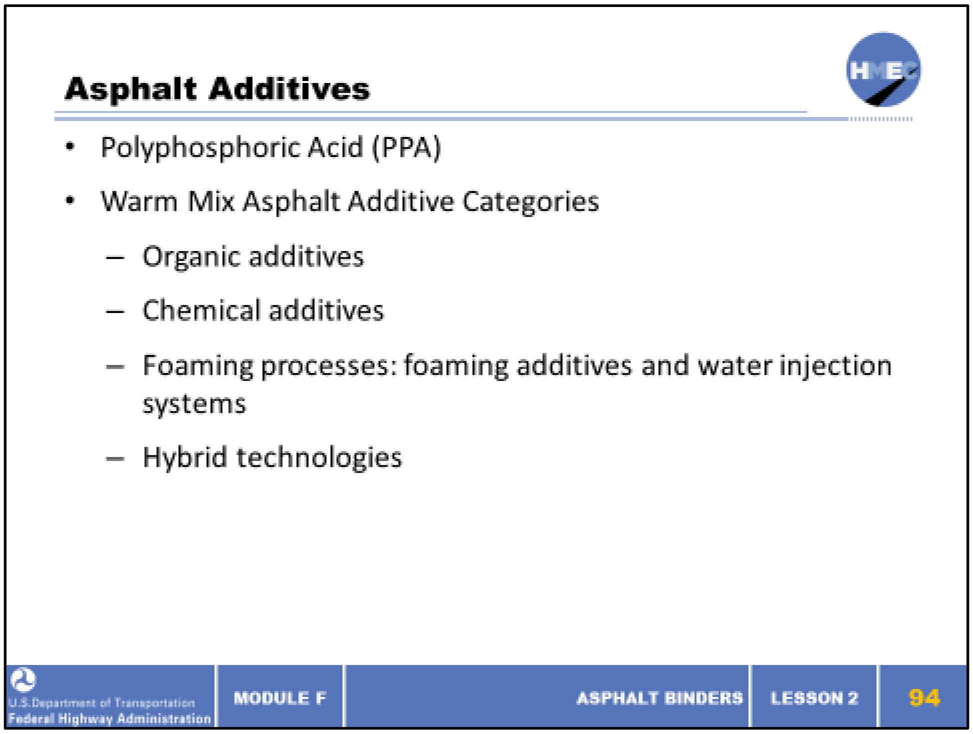


Hydrocarbons additives include recycling and rejuvenating oils and harder or natural asphalt which were discussed in Lesson 1. These additives are more commonly used in recycling aged hardened asphalt mixtures for rehabilitation purposes. The use of hard or natural asphalts are used for new asphalt mixtures to stiffen the asphalt binder under severe loading (climate and traffic) conditions. These severe or high stress areas include, intersections entrances to tollways and weight load stations, or aprons of airfields where heavy static loads with or without horizontal load applications exist.

Oxidants increase the stiffness of the asphalt binder and mixture and also improve on their strengths. Oxidants have been used in such mixes as sand-asphalt mixtures. Manganese salts have been used in some cases, but generally have limited use as an additive. These are oxidation catalysts that stiffen the asphalt and are an oil-like liquid containing a manganese compound. Initially the oxidant lowers the viscosity, but then accelerates the oxidation of the asphalt resulting in higher stiffness over a shorter period of time.

Antioxidants are different from oxidants in that the stiffening of the asphalt binder occurs during production, rather than later. Antioxidants can be lead compounds, carbon, and calcium salts. Antioxidants are generally not used much in the US. Some of the common locations where antioxidants have been used is in high stress areas in hot climates where a stiff asphalt is needed to resist truck load immediately after construction.

Slide 94



PPA is an oxidation catalysts that stiffens the asphalt, while the WMA additive allow the asphalt mixture to be placed at lower temperatures.

PPA is used in the air-blowing oxidation process as an additive in reactive polymer applications. Its use in HMA increases the high temperature Performance Grade (PG) rating of the asphalt binder without affecting the low temperature properties. PPA-modification began to be questioned, as early as 2002. Some agencies suggested the use of PPA would result in accelerated distress. For example, it has been hypothesized that pavement performance could be significantly compromised by the use of PPA in combination with an anti-stripping additive. Whether PPA is a detriment or benefit to pavement performance has yet to be answered.

The WMA additives also can act more like a compaction aid to facilitate getting density. WMA in terms of mixture design and its effect on mixture properties are discussed in more detail in Lesson 5.

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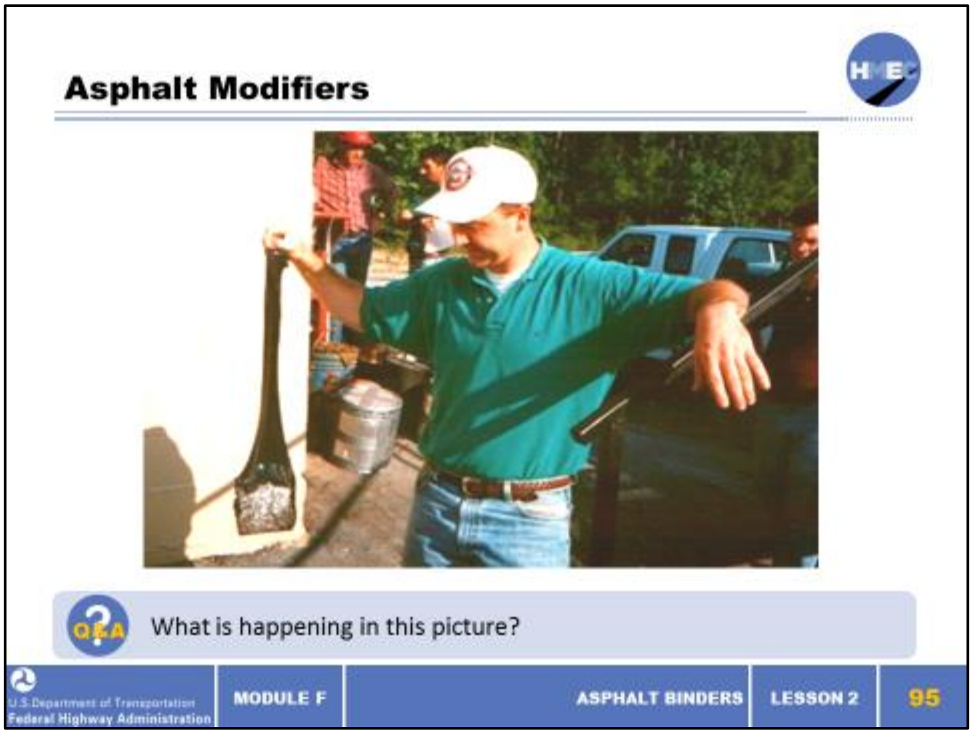
Slide 95


Asphalt Modifiers
What is happening in this picture?
U.S. Department of Transportation
Federal Highway Administration
MODULE F
ASPHALT BINDERS
LESSON 2
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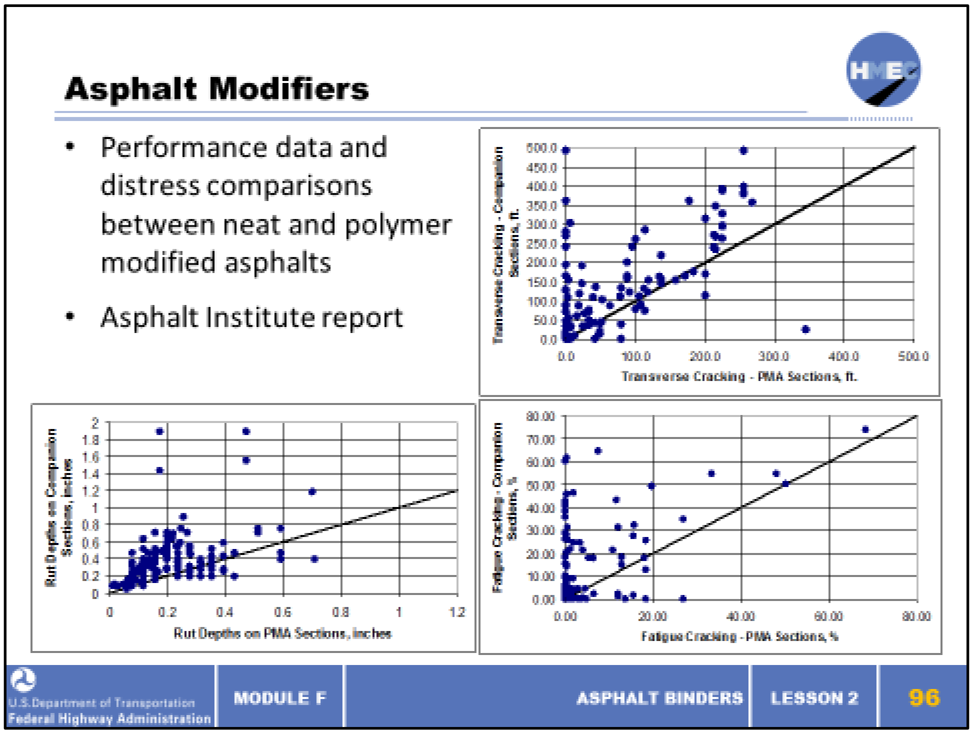
Slide 96



One graph show the comparison of rut depths measured on "companion" test sections. Companion test sections mean two: one containing a neat or unmodified asphalt mix and the other containing a polymer modified asphalt mixture. The other two graphs are for thermal cracking and fatigue cracking. As shown, the companion sections with polymer modified asphalts generally exhibit a lesser amount of distress.

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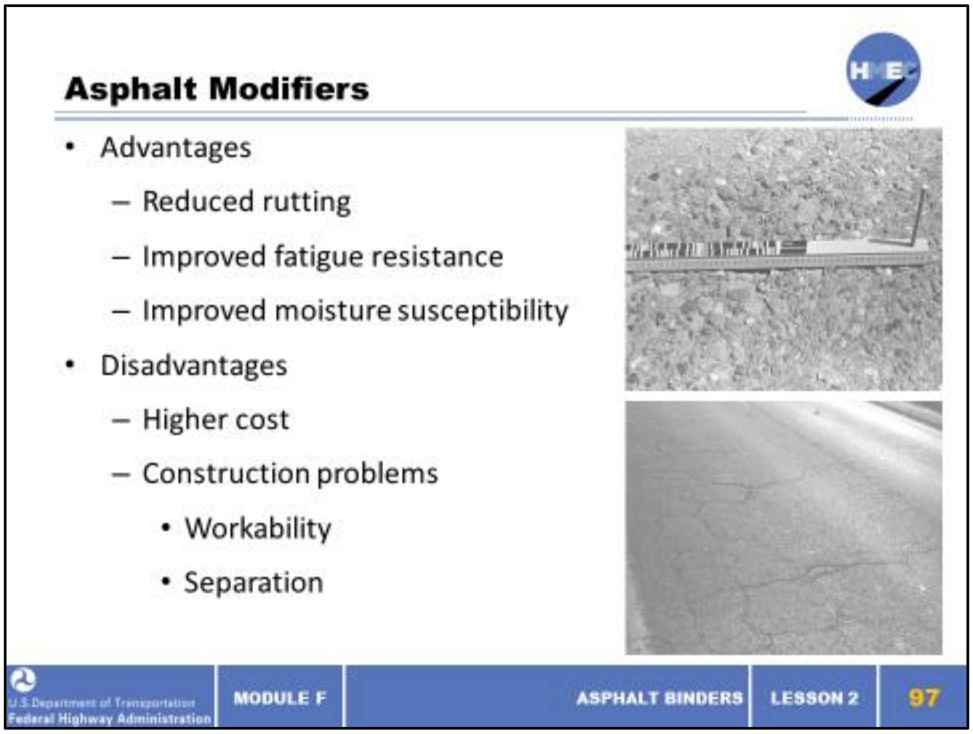
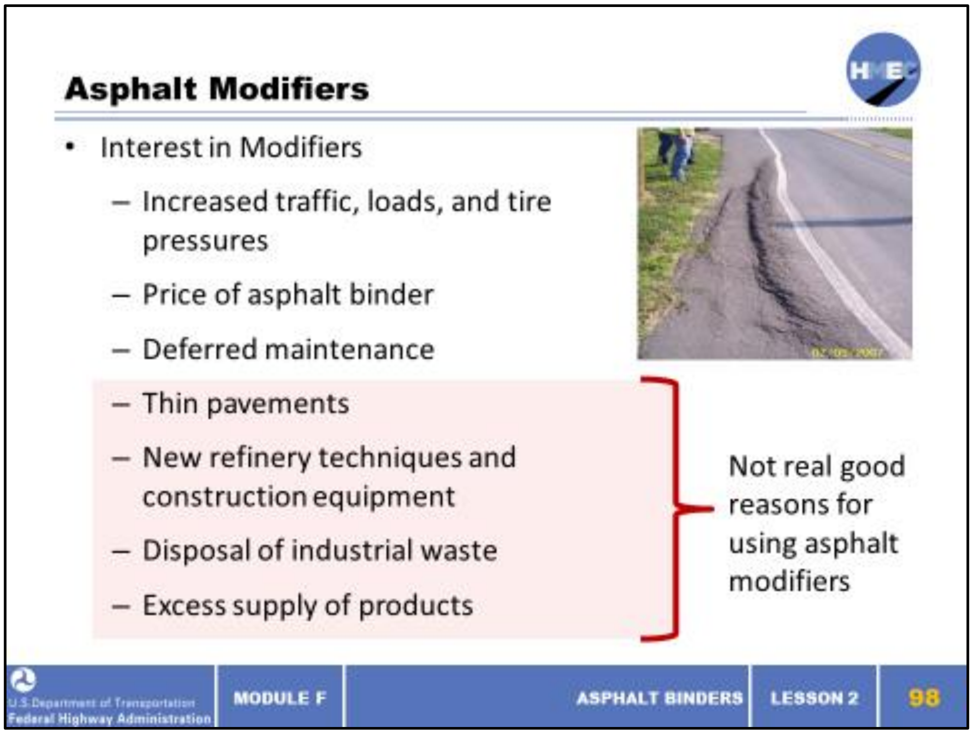
Slide 97



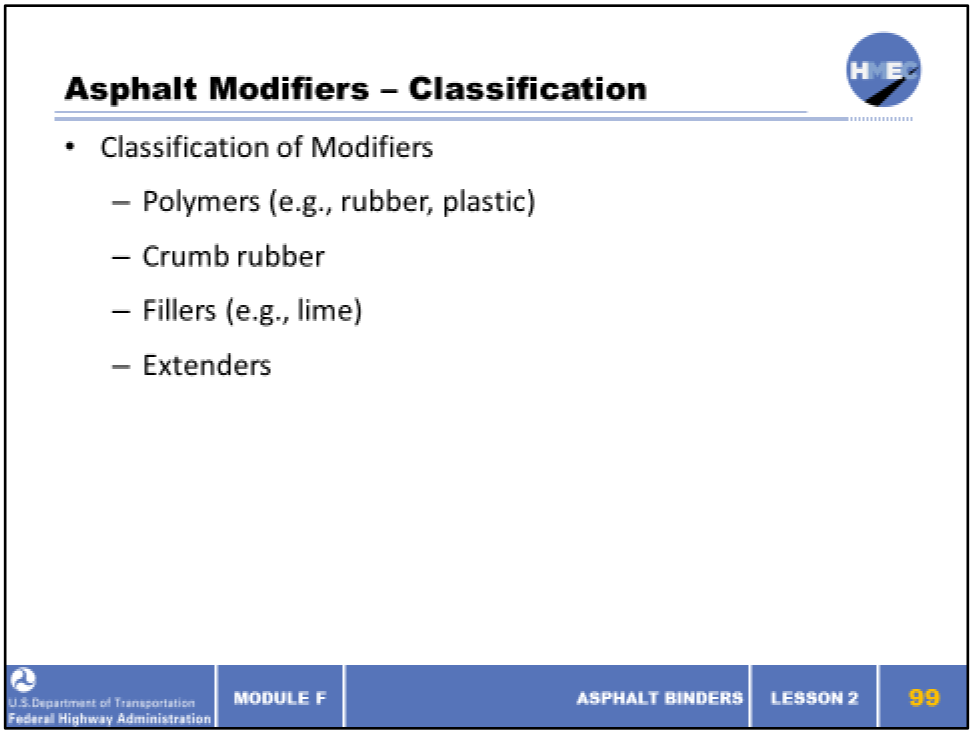
Better performance will result in lower life cycle costs. Increased initial cost is usually associated with the use of modifiers, as shown or illustrated in the previous slide.

Slide 98



Other factors that have resulted in the increased interest in modifying asphalt binders are listed.

Slide 99


Asphalt Modifiers – Classification
HMEC
• Classification of Modifiers
– Polymers (e.g., rubber, plastic)
– Crumb rubber
– Fillers (e.g., lime)
– Extenders
U.S. Department of Transportation
Federal Highway Administration
MODULE F
ASPHALT BINDERS
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Slide 100

Asphalt Modifiers – Polymer Modified

- Types of Polymer Modified Asphalt
  - Plastomers—Plastics—Thermosetting
    - High stiffness - low ductility
    - Typical kinds – EVA, EPDM, LDPE, polyolefin
  - Elastomers—Rubbers—Thermoplastic
    - Low initial stiffness - high ductility
    - Typical kinds – SB, SBS, SBR

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The term “polymer” refers to a large molecule formed by chemically reacting many (“poly”) smaller molecules (monomers) to one another in long chains or clusters. Physical properties of a specific polymer are determined by the sequence and chemical structure of the monomers from which it is made.

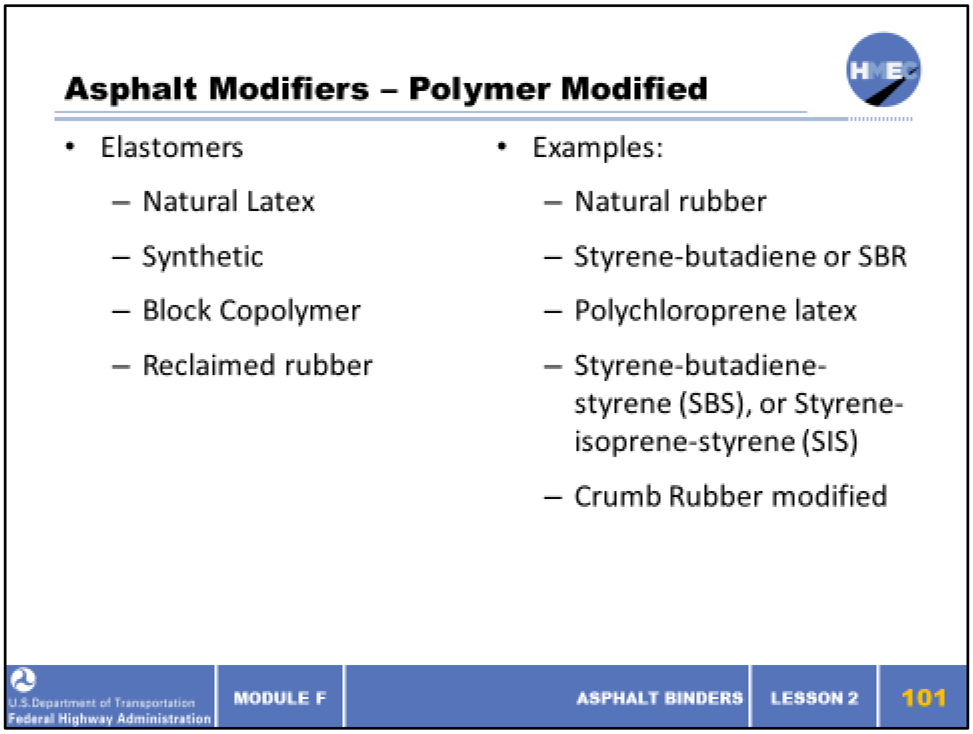
Both rubber and plastic materials are often referred to as polymers. Plastics are those polymeric materials that can be made to flow under stress. Rubbers when stretched at room temperature will return to their original shape upon release.

Many of the rubber materials that are used in asphalt binder modification are thermoplastic materials, whereas plastics are usually thermoset materials.

Thermoplastic materials soften and become plastic-like when heated but return to their hardened state upon cooling.

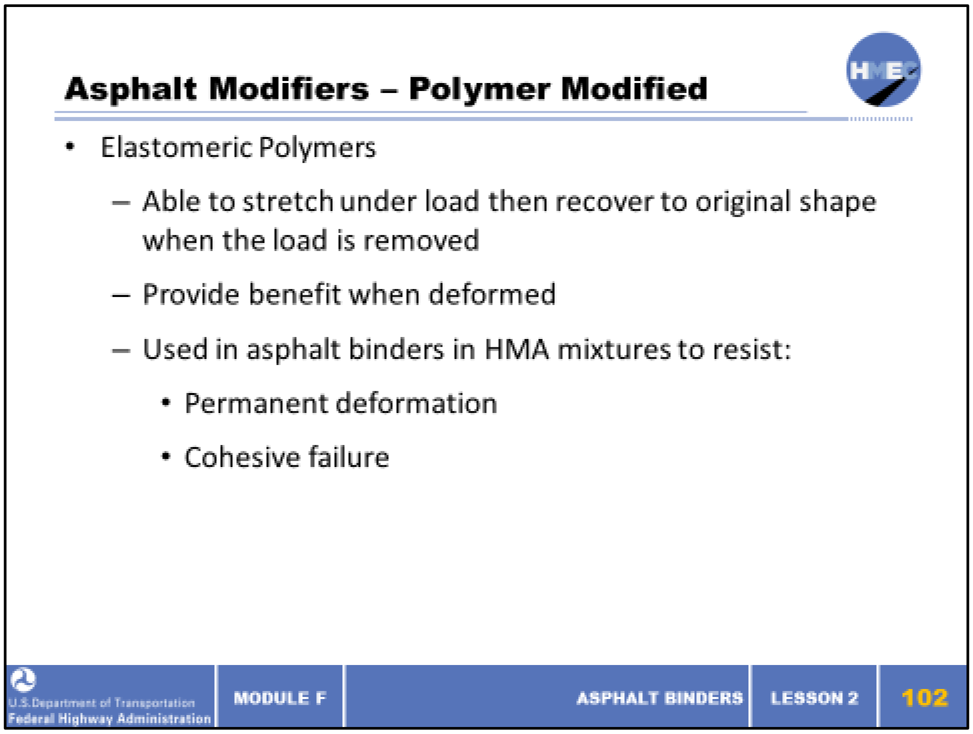
Thermosetting materials flow under stress when heated but, once cooled, cannot be re-softened by heat.

Slide 101



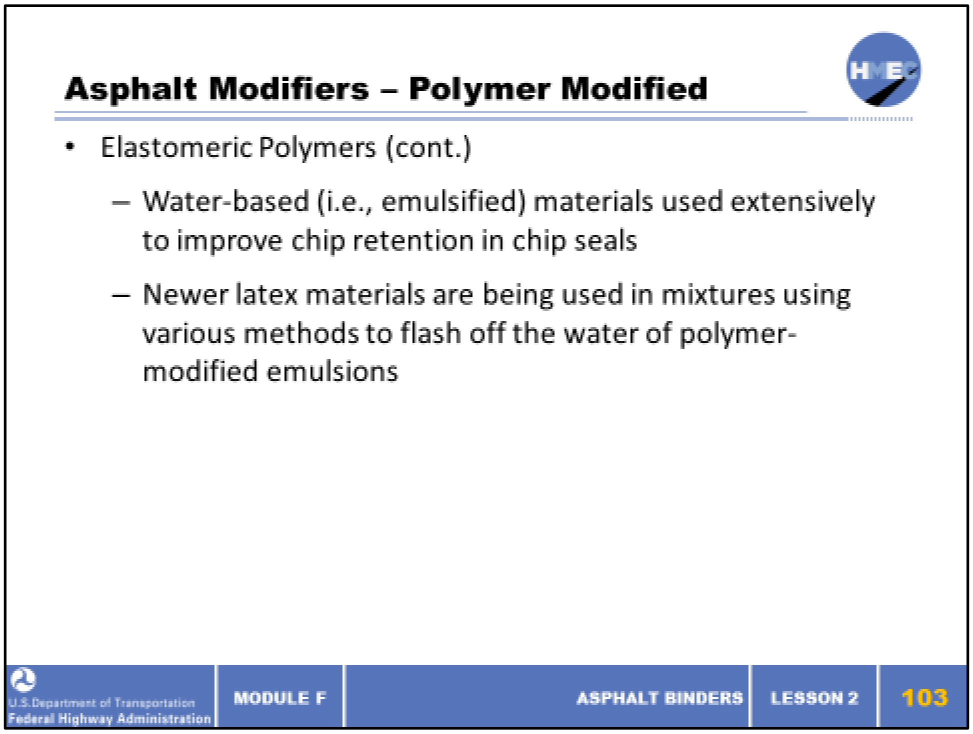
A number of examples of thermoplastic rubber materials, often referred to as elastomers in the literature, and used in asphalt binder modification, are given.

Slide 102



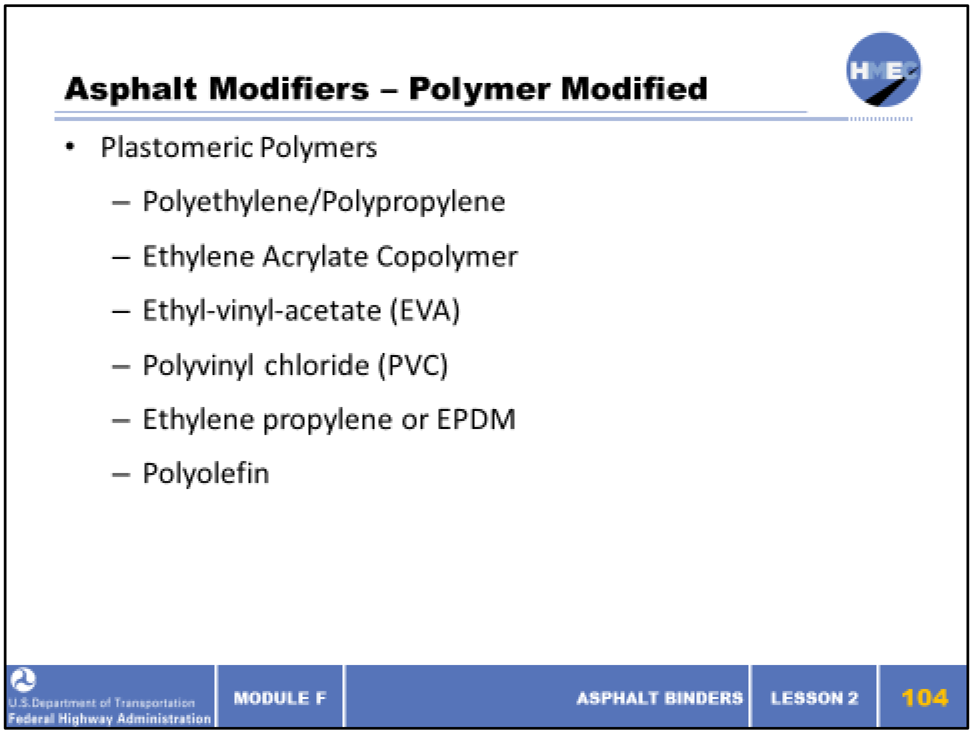
Another benefit for using the elastomers is the potential reduction in reflection cracking. A few agencies have found significant reductions in reflection cracking, but these confined to a few agencies. For example, Arizona and California are two agencies that have reported significant reductions in reflection cracking.

Slide 103



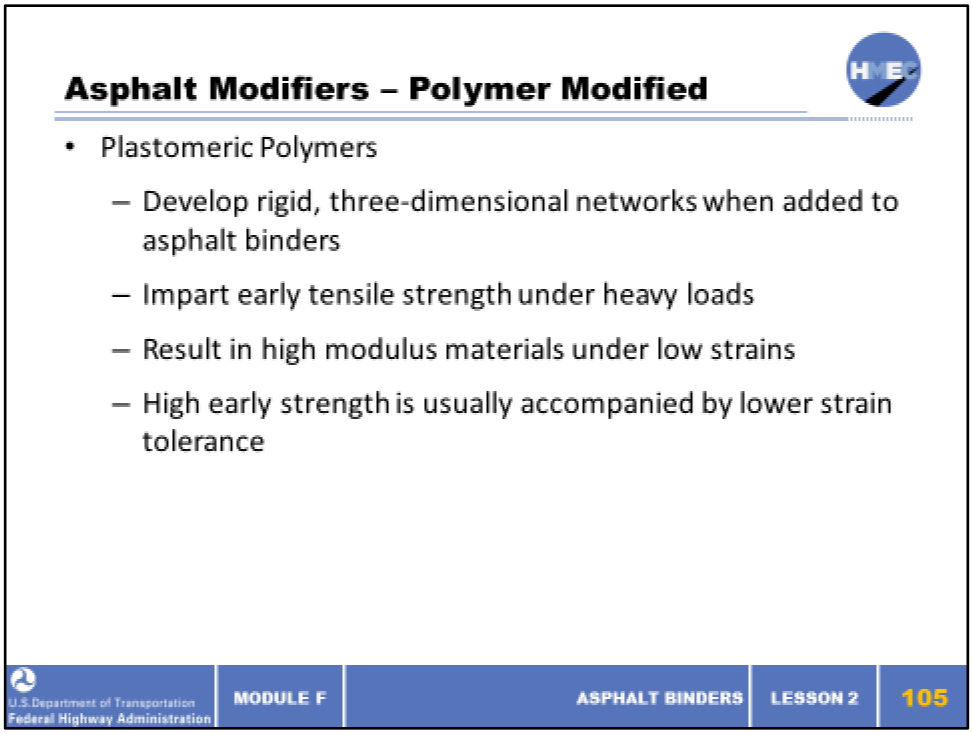
Elastomeric polymers are extensively used in emulsions to improve chip retention in chip seals. Also, newer latex materials are being used in mixtures to flash off the water of polymer-modified emulsions.

Slide 104



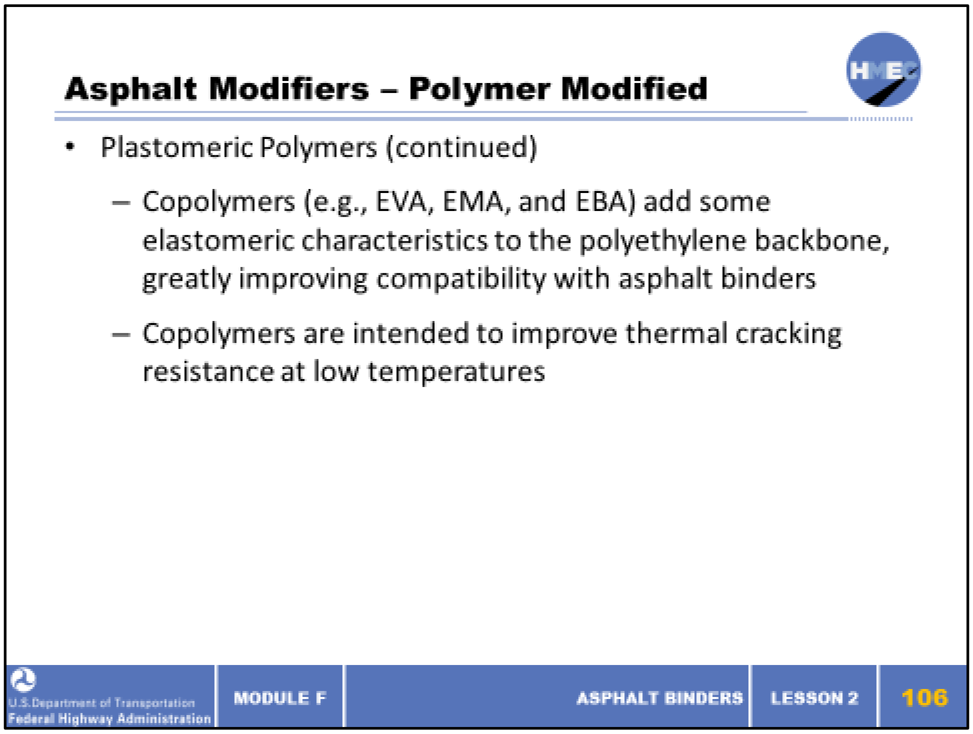
Plastic is a term usually applied to those polymeric materials that can be made to flow under stress. Plastic most often refers to the finished product including fillers, stabilizers, etc. However, it can mean the resins or homogenous polymeric starting materials.

Slide 105



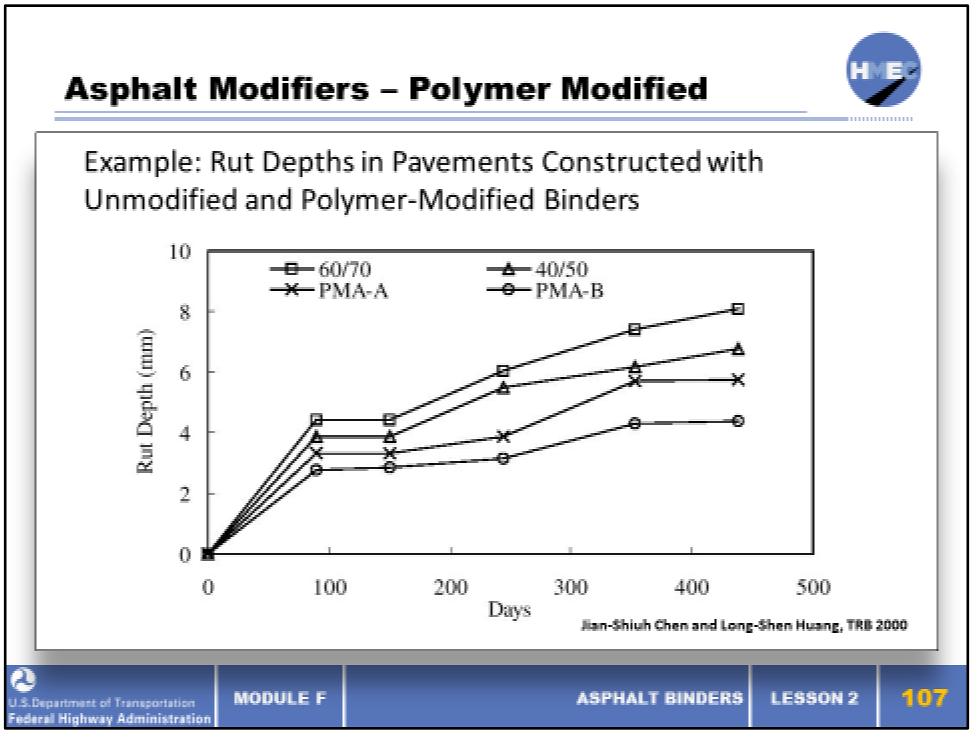
This structured network imparts early tensile strength under heavy loads and results in a high modulus material under low strains. However, the high early strength is usually accompanied by lower strain tolerance.

Slide 106



These characteristics greatly enhance the compatibility of these materials with asphalt binders and are intended to improve resistance to low temperature cracking.

Slide 107



The figure shows measured rut depths in pavements constructed with unmodified and polymer-modified binders (PMA-A and PMA-B).

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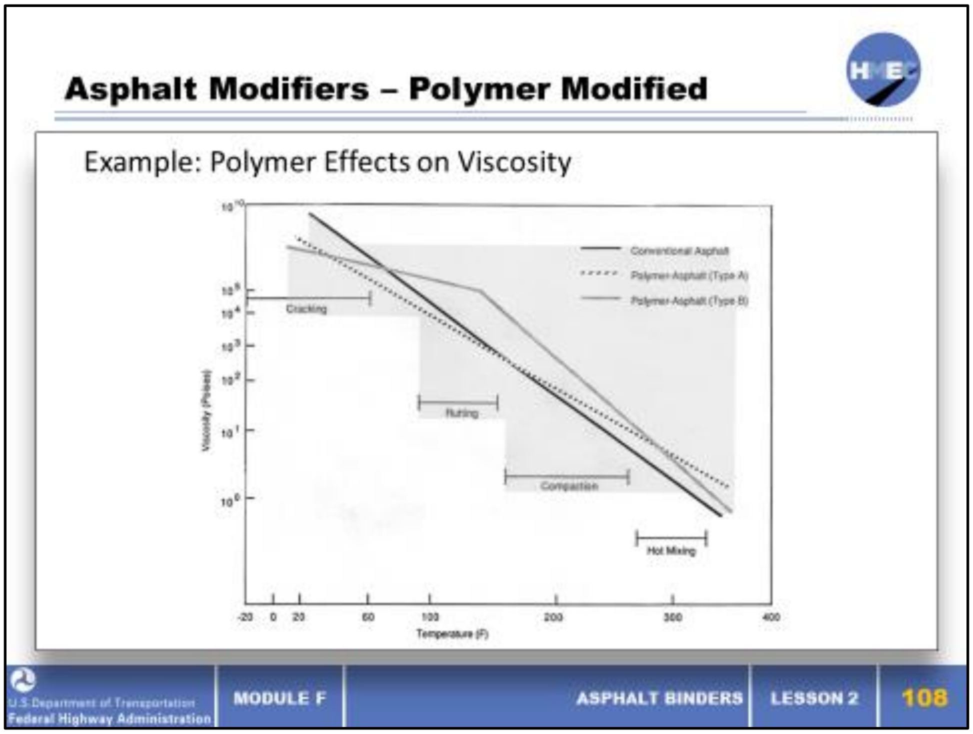
Slide 108


Asphalt Modifiers – Polymer Modified
Example: Polymer Effects on Viscosity
Conventional Asphalt
Polymer Asphalt (Type A)
Polymer Asphalt (Type B)
Cracking
Rutting
Compaction
Hot Mixing
Viscosity (Poises)
Temperature (F)
-20
0
20
60
100
200
300
400
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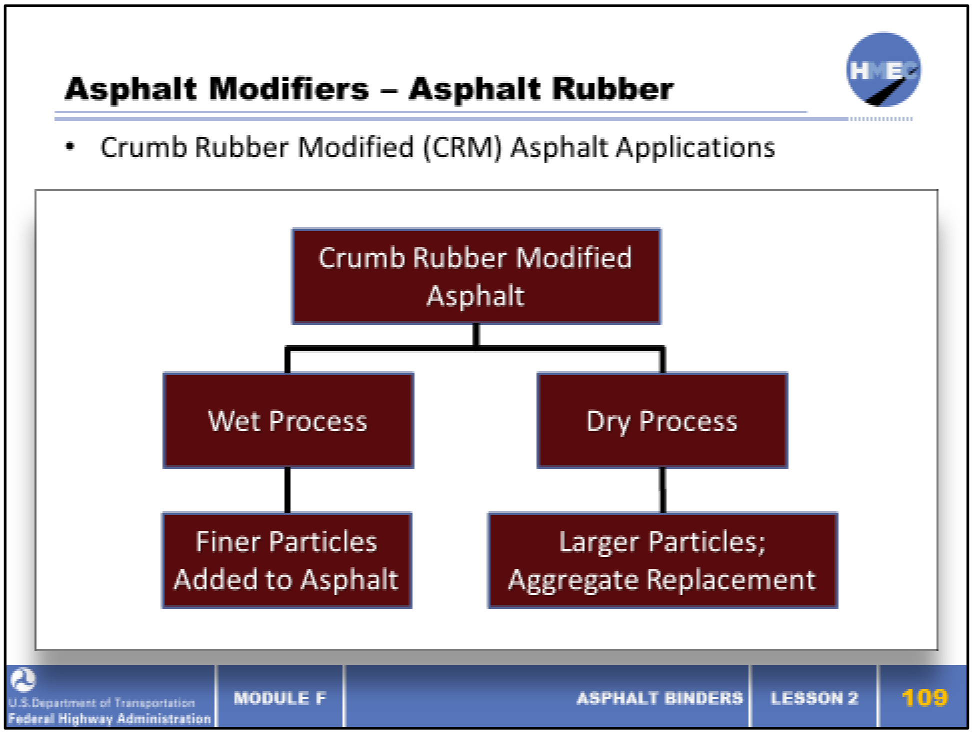
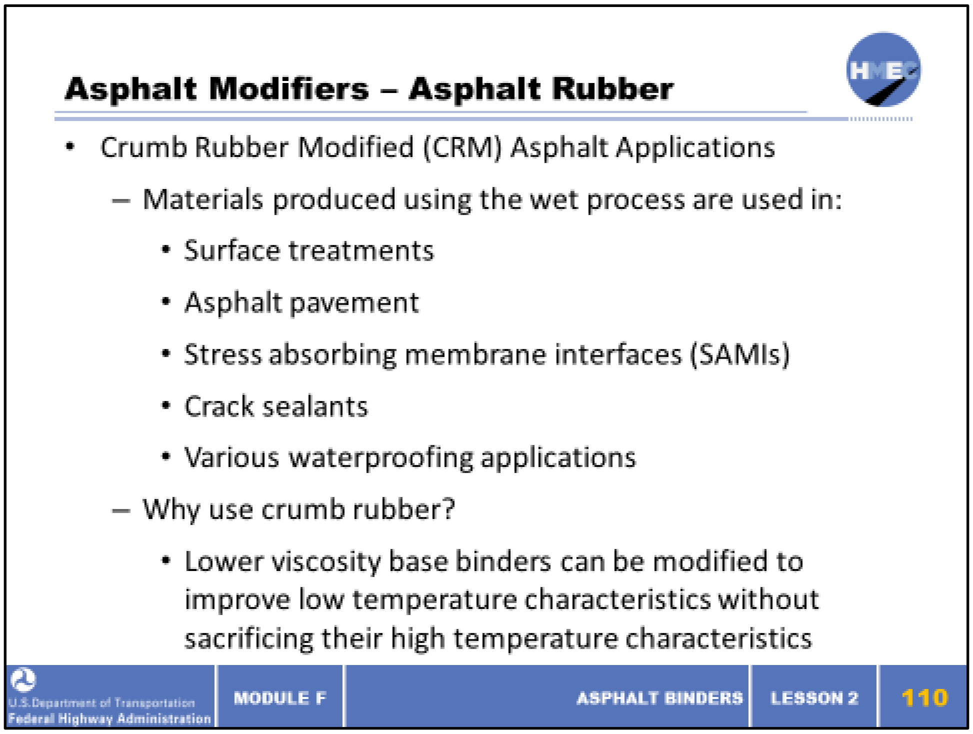
Slide 109



The wet process is typically referred to as asphalt-rubber. Generally 18 to 26% crumb rubber by weight of asphalt cement of 16 mesh or smaller is reacted with the asphalt cement at 375 to 425 °F (191 to 218 °C) for 1 to 2 hours. The blend is formulated at elevated temperatures to promote potential chemical and physical binding of the two materials. This process is also called or referred to as the "McDonald Process."

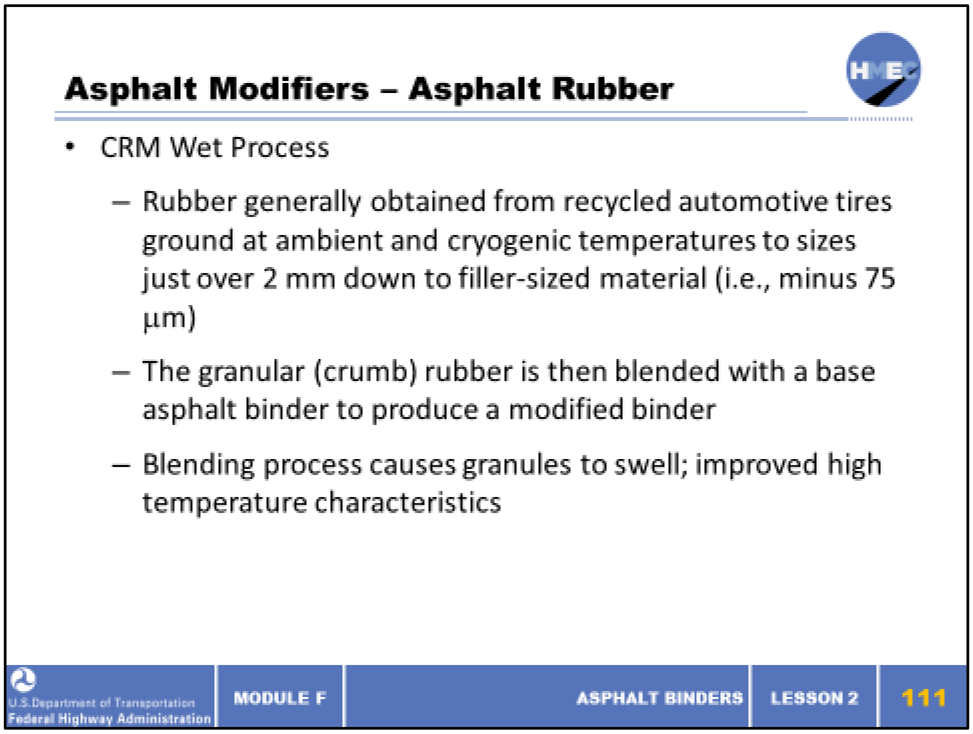
The dry process mixes the crumb rubber with aggregate before incorporating the asphalt binder. About 3 to 5% of coarse rubber particles by weight of the aggregate are used. The size is generally $^{1}/_{16}$ to ¼ inch. The natural aggregate is usually a gap graded blend to accommodate the rubber particles as aggregate in the mix. Obviously, more crumb rubber is used in the dry process, in comparison to the wet process. The dry process is also referred to in the literature as "plus ride."

Slide 110



Lower viscosity base binders can be modified to improve low temperature characteristics without sacrificing their high temperature characteristics.

Slide 111



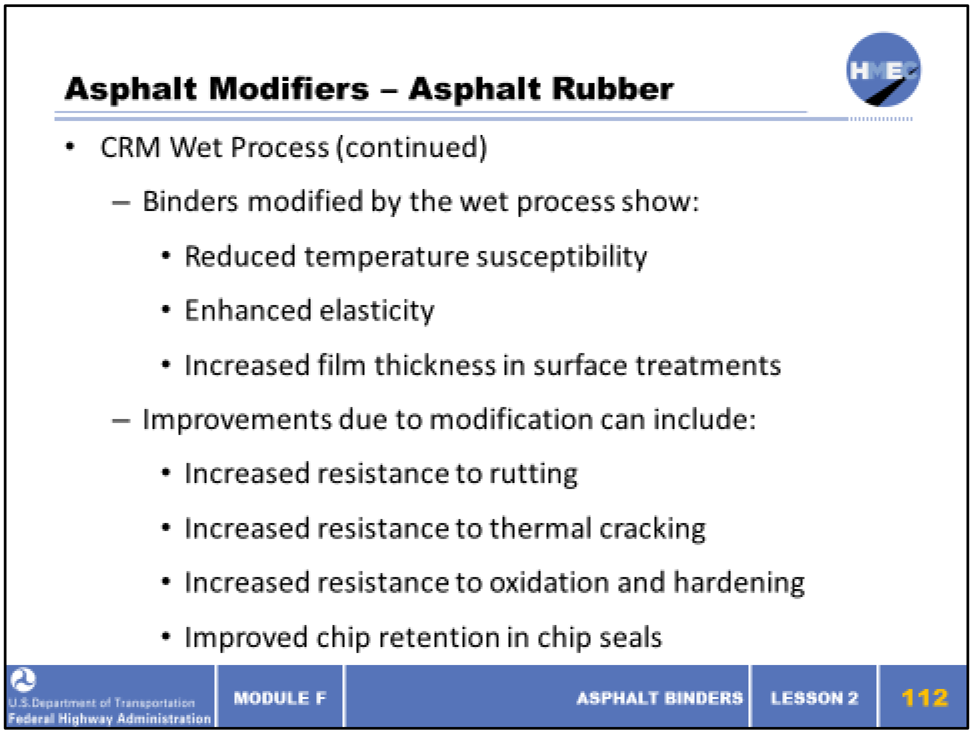
Rubber is blended with asphalt binder at high temperature to form a modified binder.

ASTM D 6114 notes that at least 15% rubber by weight of total blend is usually necessary to provide acceptable properties.

Rubber granules react with asphalt binders in the blending process causing them to swell three to five times their original volume. The reaction results in a modified binder that has improved high temperature characteristics that can improve resistance to permanent deformation.

Lower viscosity base binders can be modified to improve low temperature characteristics without sacrificing their high temperature characteristics.

Slide 112

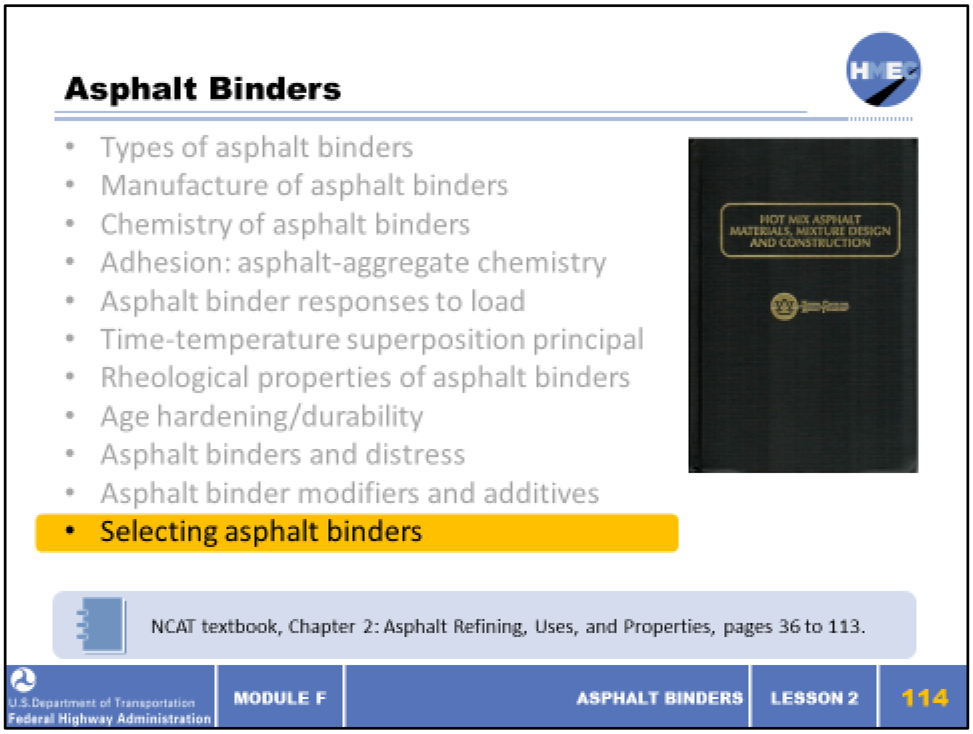


Improvements attributed to mixtures with rubber-modified binders can include increased resistance to rutting, thermal cracking, and oxidation and hardening as well as improved chip retention in chip seals. Another improvement in using crumb rubber or asphalt-rubber mixtures is the reduction in reflection cracking. As noted earlier in this lesson, however, this benefit is confined to a few agencies—primarily, Arizona and California.

Slide 113

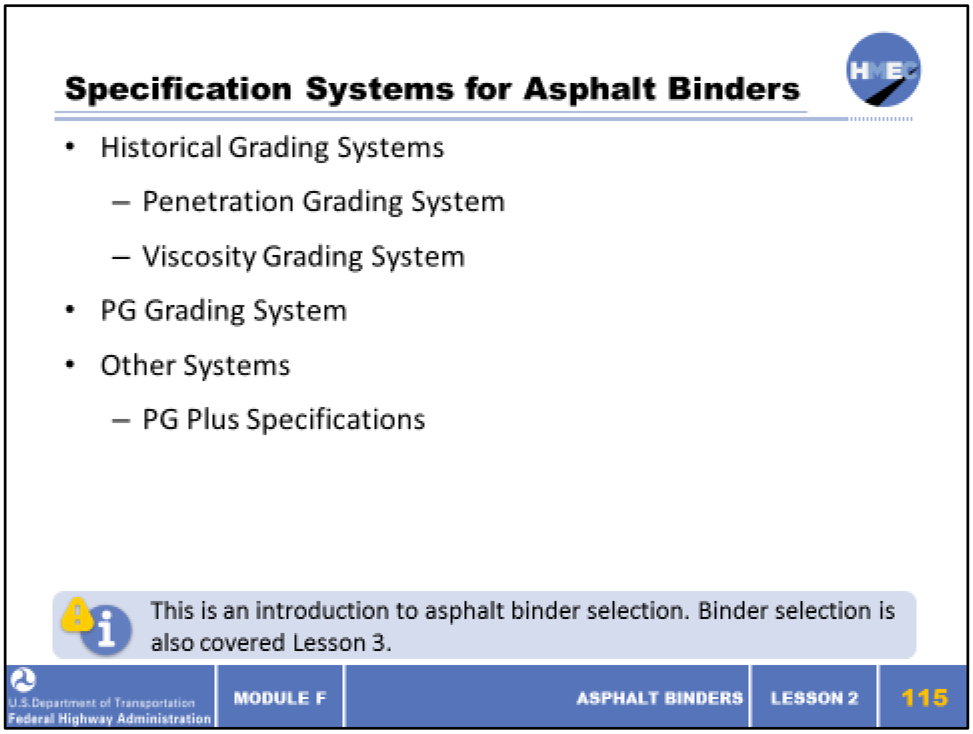
Production and construction personnel not familiar with rubber-modified binders may find working with these materials difficult.

Slide 114



This topic only represents an introduction to binder selection.

Slide 115



The first was the penetration grading system introduced in 1918 by the Bureau of Public Roads (now FHWA). The viscosity systems were developed in the 1960s to replace the penetration system and were the most commonly used systems until the Superpave PG system was developed in the 1990s. Only the Superpave PG system is used in the US now.

This section deals with specification systems for asphalt binders. It provides a brief overview of the penetration and viscosity grading systems but concentrates on the PG grading system. AASHTO and ASTM have published specifications for the penetration and viscosity grading systems, and AASHTO first published a provisional standard for the PG system in 1994.

Slide 116

Asphalt Binder Selection

Grading System	Climate	Traffic	Structural Design	Layer Location	Fundamental Properties Measured
Penetration	✓				
Viscosity	✓				✓
PG	✓	✓	✓	✓	✓

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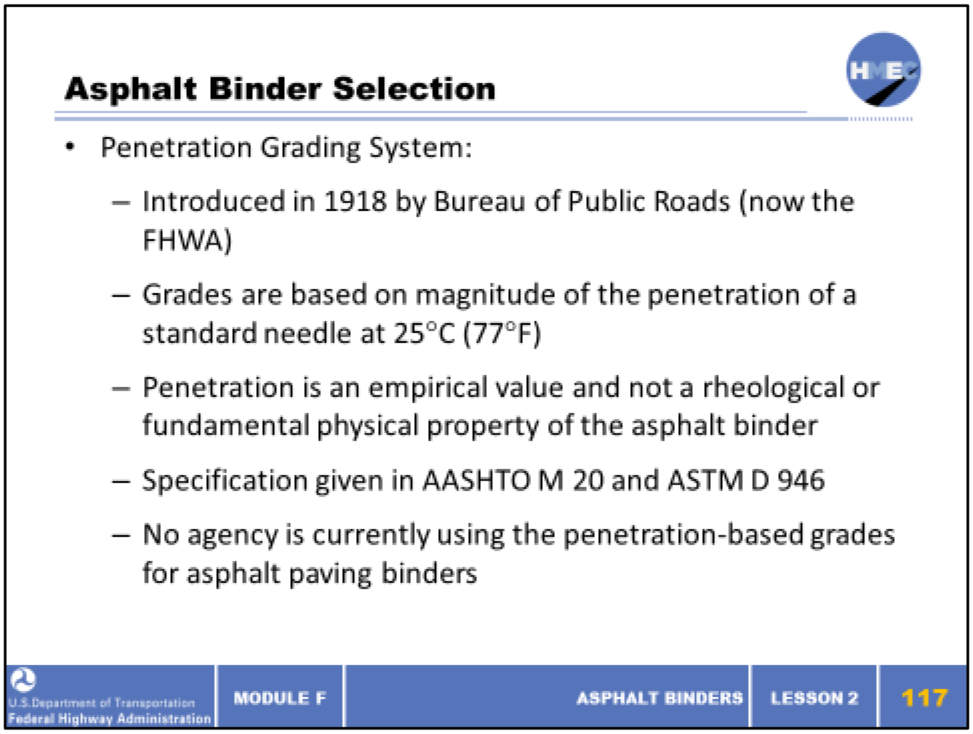
Selection of the grade of asphalt binder should be based on the factors shown in this slide/table. The selection is often based on the balancing of different requirements. As summarized in the table, the PG system is the only one that considers each factor in the table. This was one of the reasons most agencies in the US decided to switch from the other specifications to the PG system in selecting an asphalt binder.

The remainder of this topic provides a brief reviews each grading system. Lesson 3 provides additional information on the PG system because it is used by just about every agency in the US.

In addition to the factors mentioned in the table, it is also important to note that with the viscosity grading and PG systems, the fundamental properties of the material were measured. The previous system (penetration) relied upon empirical measurements.

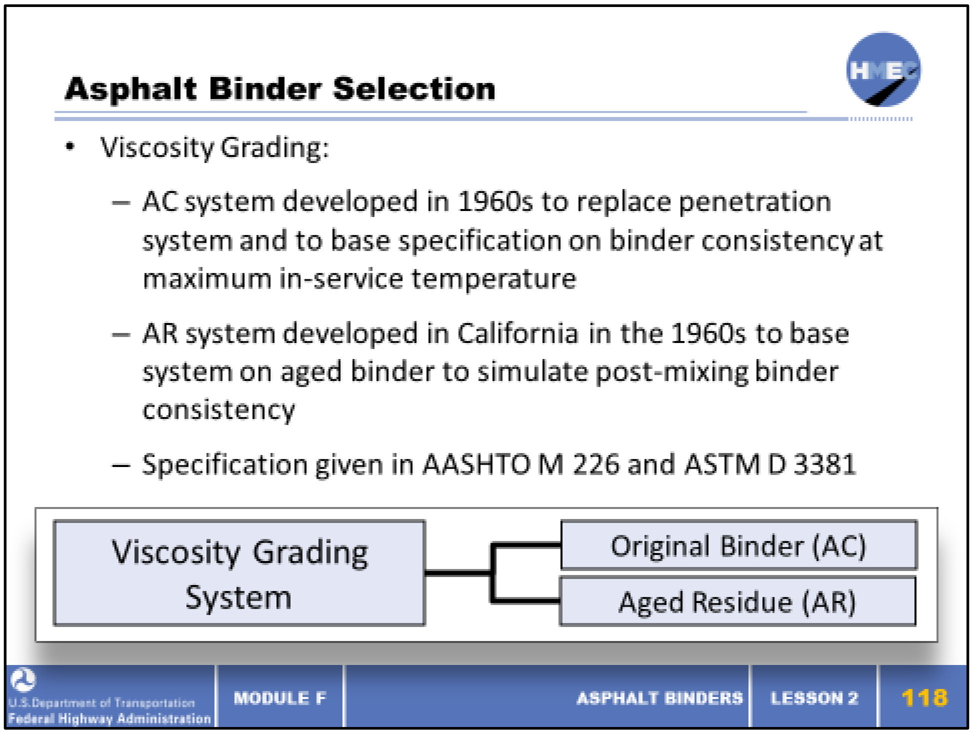
Slide 117



It is based on the penetration of a standard needle into a tin of asphalt binder at 25 °C (77 °F). This measurement does not represent a rheological property or fundamental physical property of the asphalt.

The specification is given in AASHTO M20 and ASTM D946. However, one of the major disadvantages of the penetration specification: it is based on tests conducted at a single temperature (i.e., it does not account for temperature susceptibility).

Slide 118


Asphalt Binder Selection
HMEC
• Viscosity Grading:
– AC system developed in 1960s to replace penetration system and to base specification on binder consistency at maximum in-service temperature
– AR system developed in California in the 1960s to base system on aged binder to simulate post-mixing binder consistency
– Specification given in AASHTO M 226 and ASTM D 3381
Viscosity Grading System
Original Binder (AC)
Aged Residue (AR)
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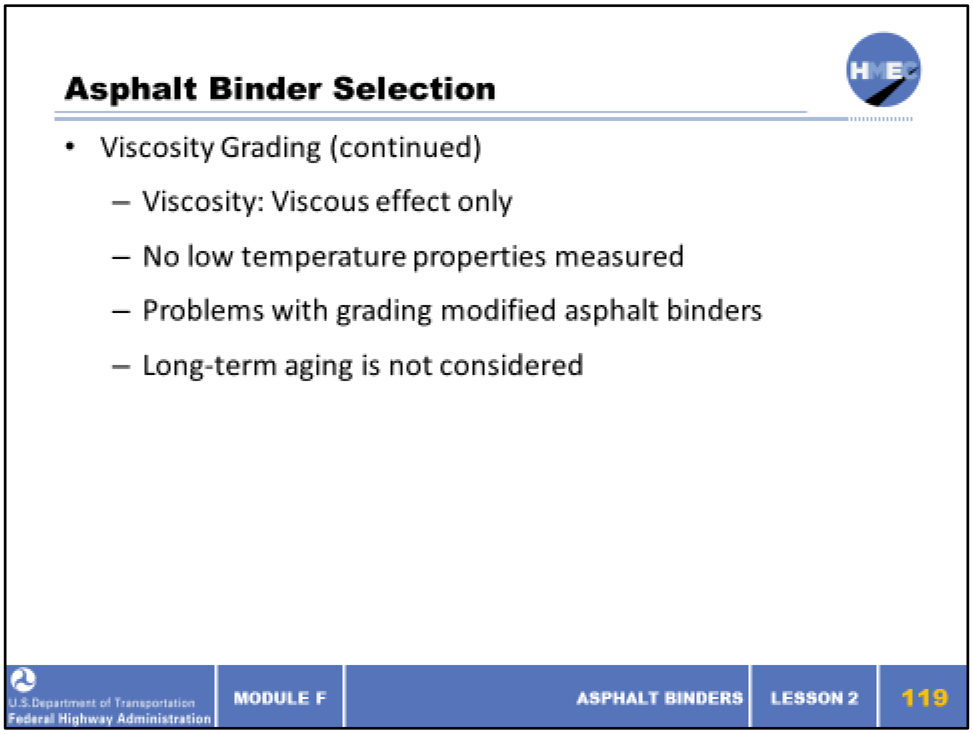
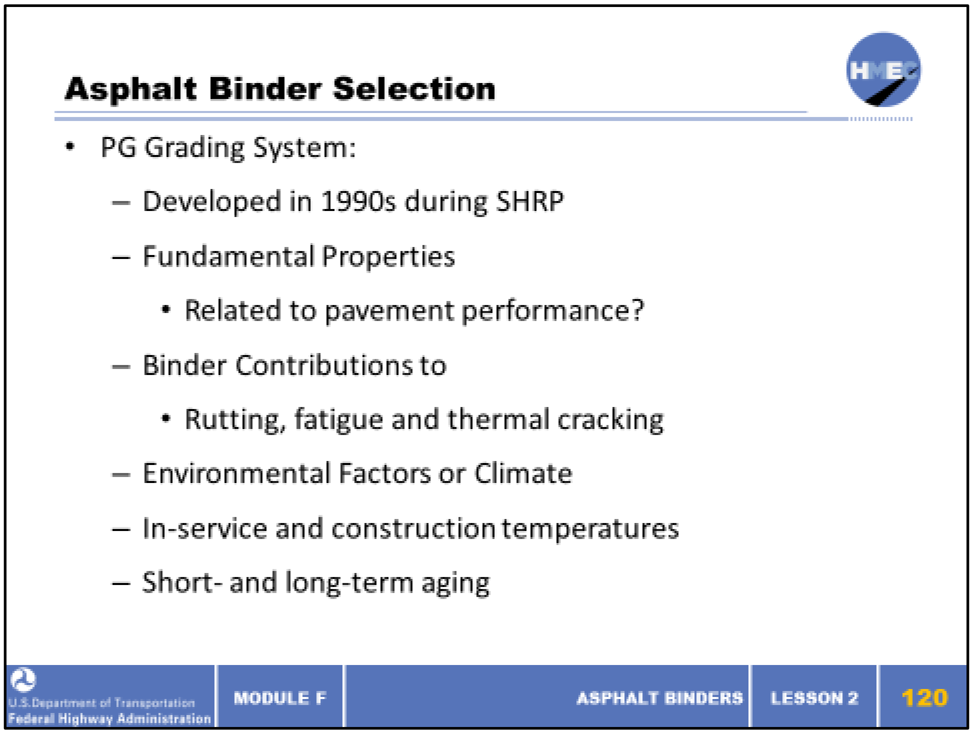
Slide 119



The principal limitations of the penetration and viscosity grading systems are listed.

• The viscosity grading systems consider only the viscous component of the binder and do not consider the elastic component.
• The penetration system is based in an empirical measurement (i.e., it does not measure a fundamental engineering property such as viscosity).
• Low temperature properties of the asphalt binder are not considered in any of the systems.
• In addition, the systems have problems with modified binders due to the significant and varied impact modifiers have on the shear flow properties of asphalt binders.
• Although the system considers aging through construction, none considers the effect of long-term aging.

Slide 120



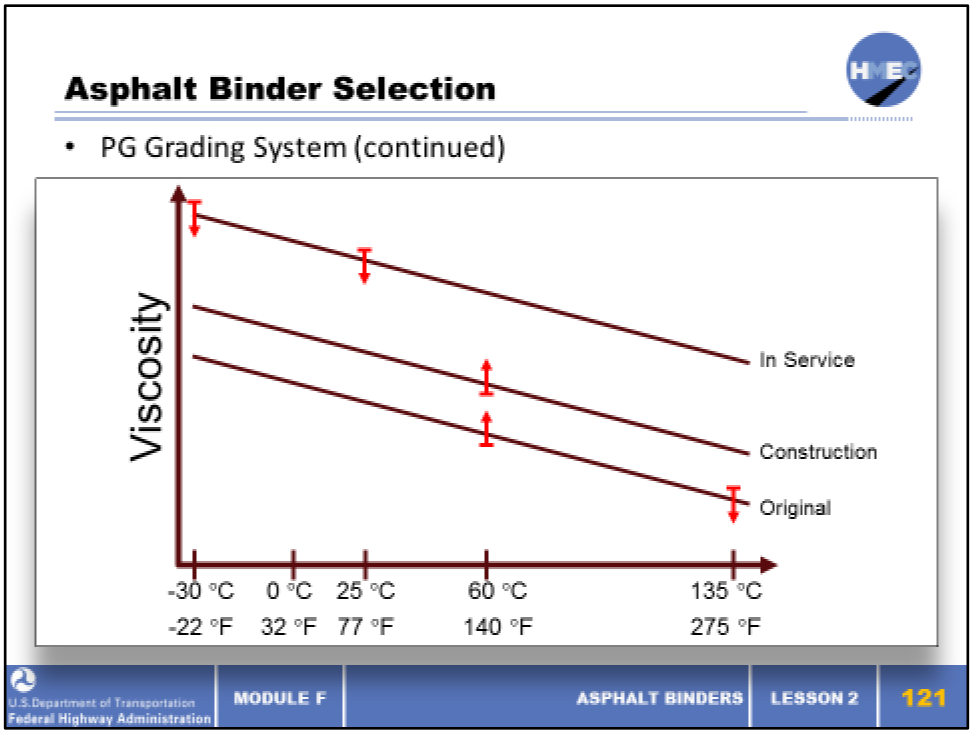
The specification attempts to specify the binder in such a way that its contribution to rutting resistance, fatigue cracking resistance, and thermal cracking resistance is maximized.

It is based on the climatic conditions expected at the site where the HMA pavement will be constructed.

It contains elements that address both in-service temperatures and construction temperatures.

It specifies procedures for simulating both short-term and long-term aging.

Slide 121



The low temperature shown (-30 °C) is for illustrative purposes only. The actual low temperature utilized for testing purposes is determined based on the expected low temperature of the pavement at the project site.

The figure schematically shows the limits on consistency (viscosity) imposed by the PG specification. The actual test methods produce other fundamental engineering properties such as modulus and failure stresses. However, viscosity is used here for illustration. As shown, the PG specification (AASHTO M 320) imposes a maximum limit on the original binder at 135 °C. Note that at low temperature of -30 °C a maximum is imposed on the viscosity (in reality it is the S value = 300 MPa maximum limit in the BBR test at that temperature).

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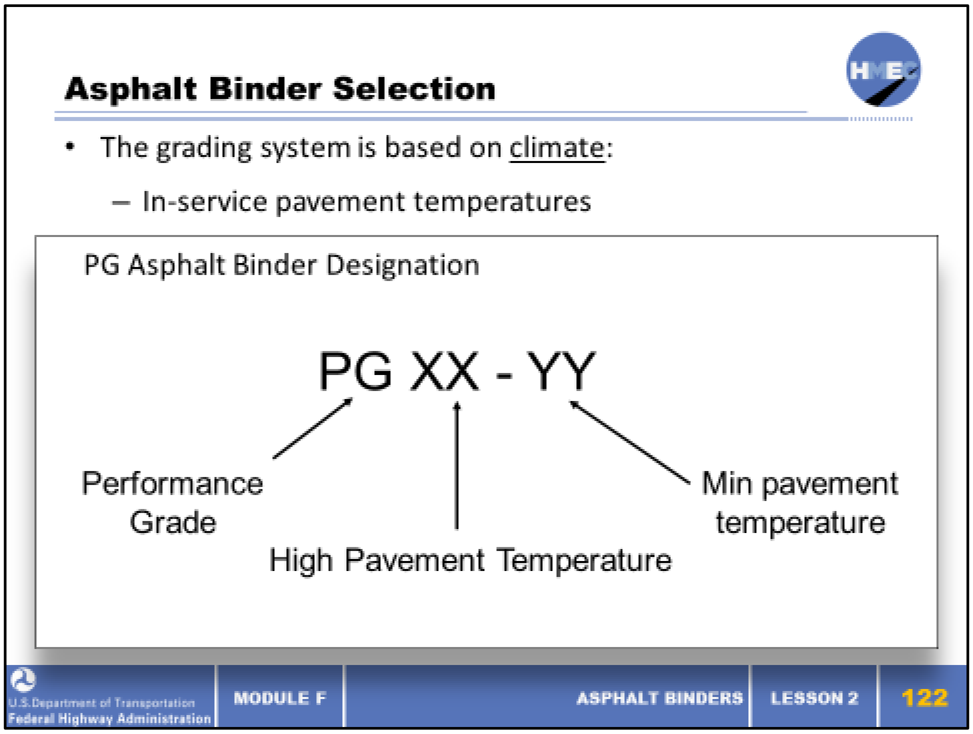
Slide 122



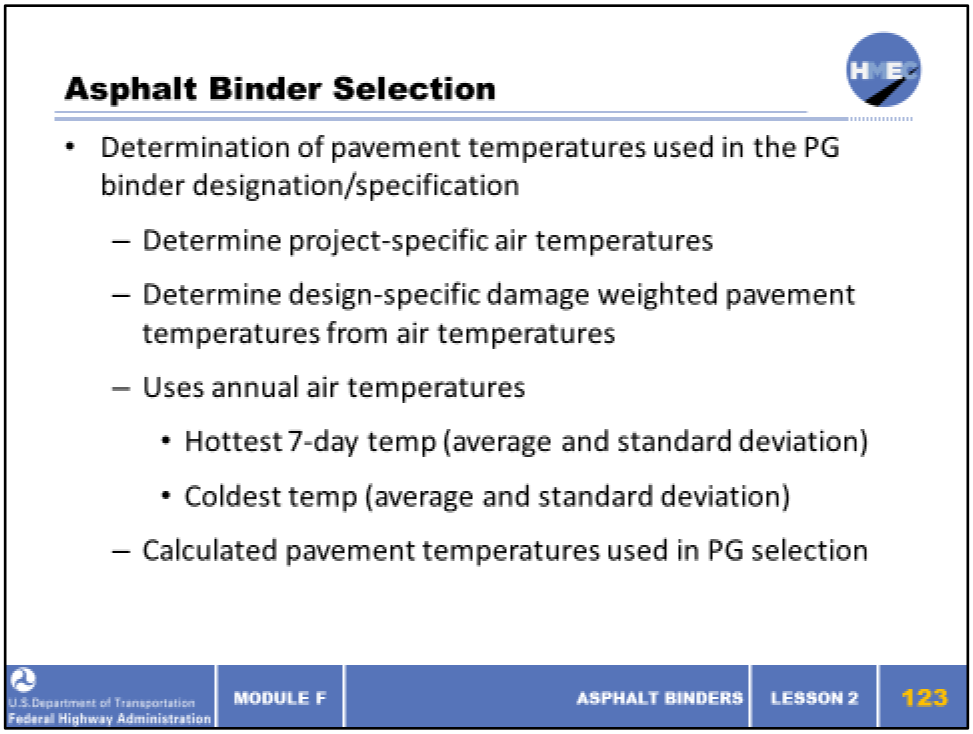
The grading system is based on the expected 7-day maximum average pavement temperature *and* the expected minimum pavement temperature. The XX shown here corresponds to the expected 7-day average maximum temperature at the site, whereas the YY corresponds to the 1-day minimum temperature.

As an example, a PG 64-22 refers is a binder grade that would be specified for a pavement with an expected 7-day maximum average temperature of 64 °C and an expected minimum pavement temperature of -22 °C.

The 7-day maximum average temperature was selected for the specification because it is simple to determine and explain. The initial term that was used early in SHRP was defined as the "Damage Weighted High Temperature" and means that the temperature was calculated based on the amount of damage created and not just merely taking the highest temperature recorded. For example if the highest temperature recorded in Montana, let us say for this discussion, is 64 ^{o}C. This temperature however only lasted for 10 hrs on one hot day in Montana. Now let us say that a lower temperature of 58 ^{o}C was also recoded but for a longer period of time say seven days. So the lower temperature will create more damage in Montana (our example) than the higher 64 ^{o}C temperature. So the damage weighted temperature in our example will be 58 ^{o}C.

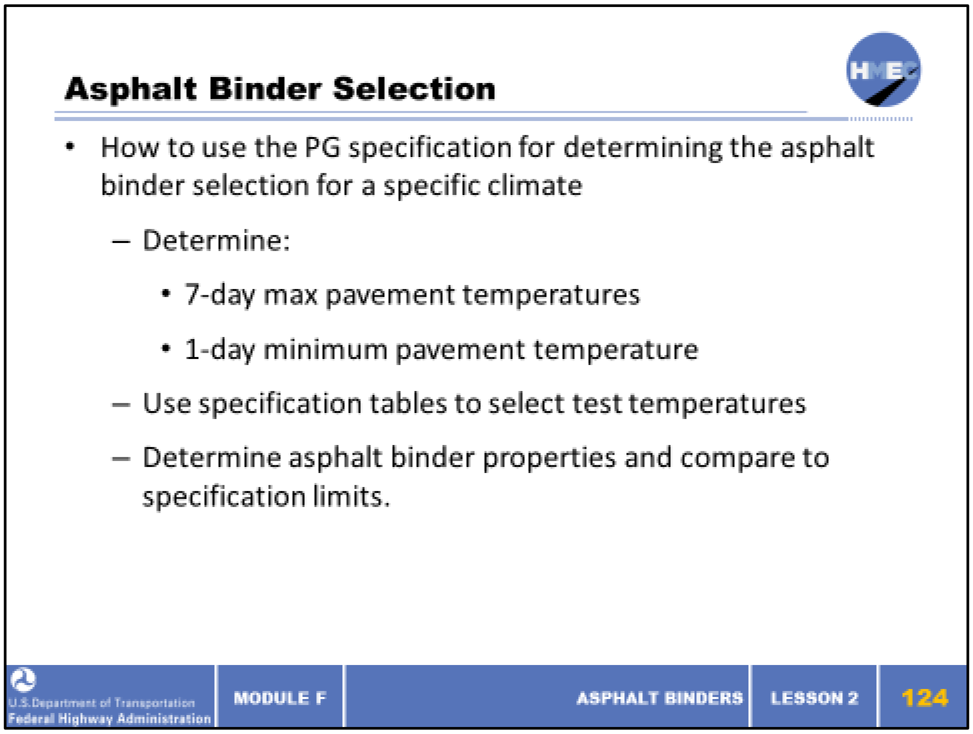
Thus, the 7-day maximum average temperature was used to determine the high pavement temperature for an asphalt grade for a specific site.

Slide 123



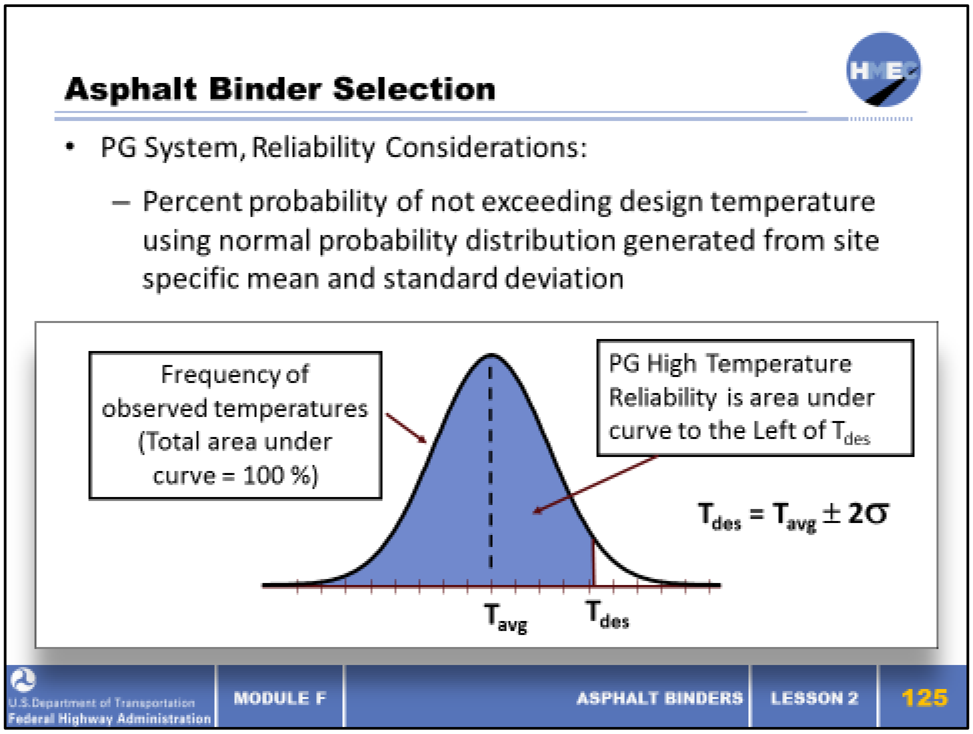
Weather information is used to estimate the extreme pavement temperatures. Data from numerous weather stations can be found by using the LTPPBind software available from the FHWA.

Slide 124



Asphalt binders are used during construction to temperatures approaching 175 °C (350 °F). In-service temperatures range from about -40 to 80 °C (-40 to 175 °F). Weather information is used to estimate the extreme pavement temperatures. Data from numerous weather stations can be found by using the LTPPBind software available from the FHWA.

Slide 125



Selecting progressively higher reliabilities (e.g., 80, 90, 95%, etc.) increases the area under the normal distribution curve and reduces the amount of risk of encountering a temperature that exceeds the design temperature.

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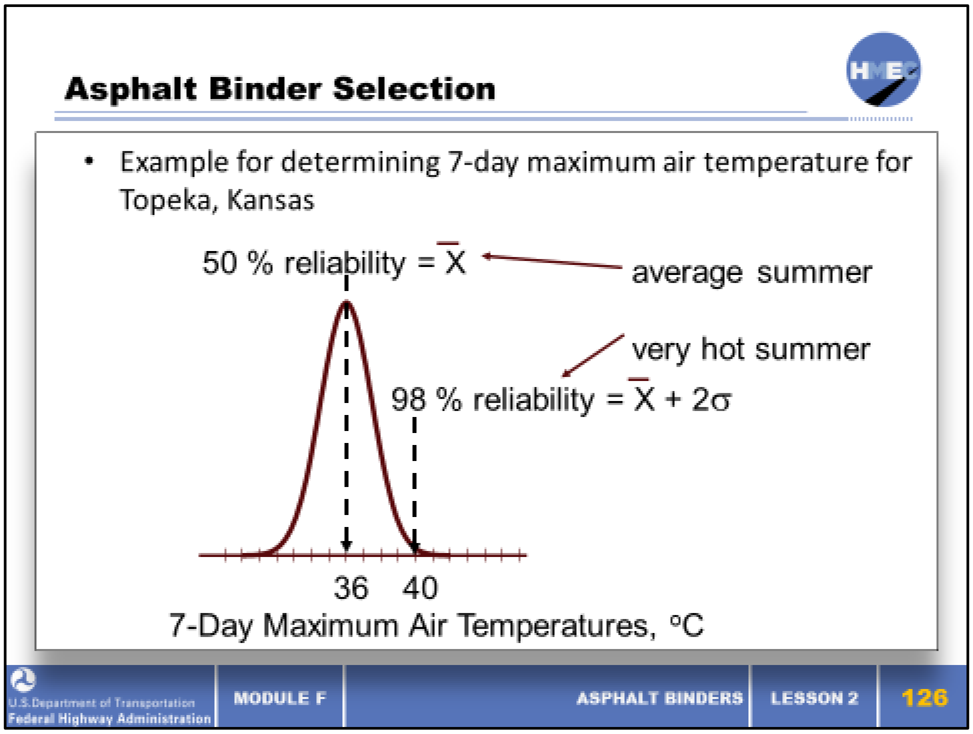
Slide 126



The mean of all values in the distribution represents 50% reliability (half of the total area under the curve). Selection of a higher reliability such as 98% (which corresponds to a very hot summer) increases the area by approximately two standard deviations above the mean value.

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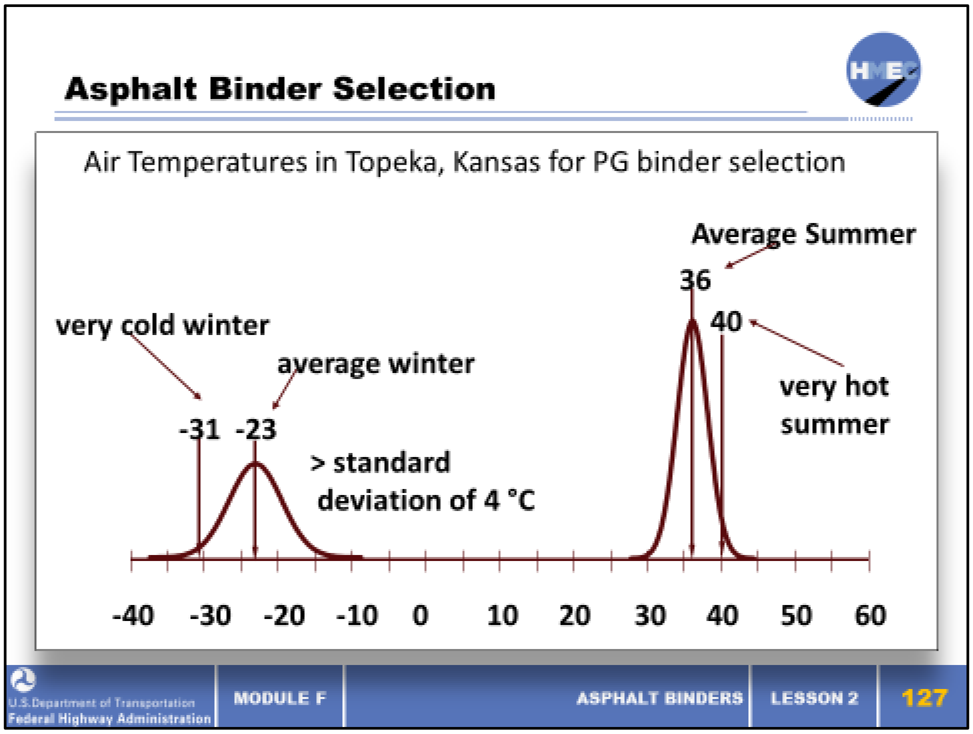
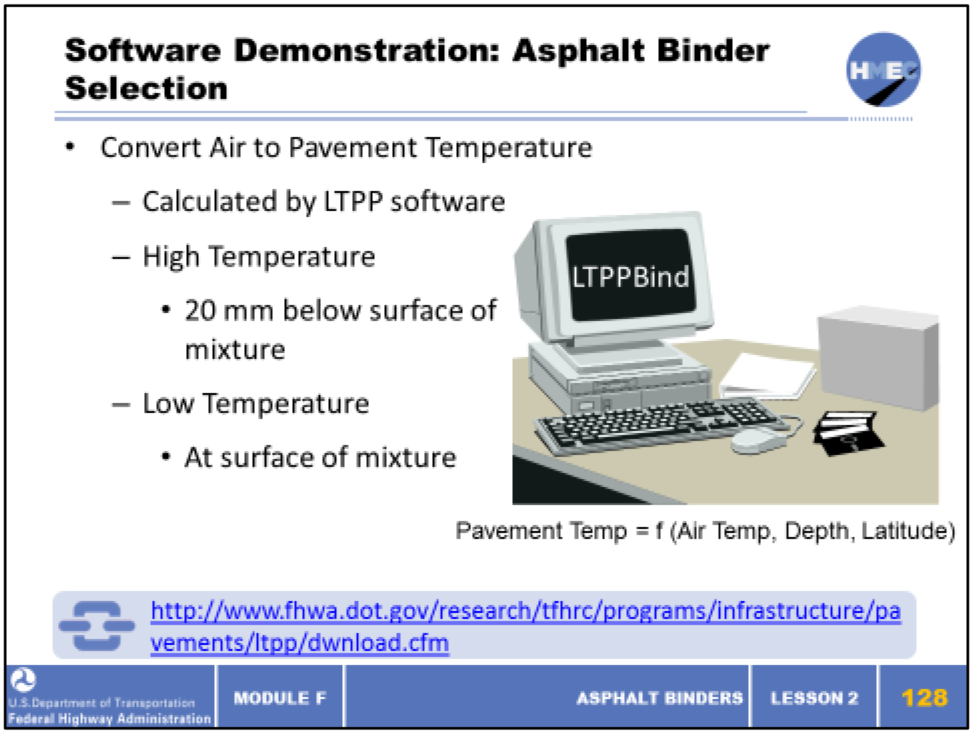
Slide 127



The standard deviation of the low temperatures is double that of the standard deviation of the high temperatures.

Slide 128



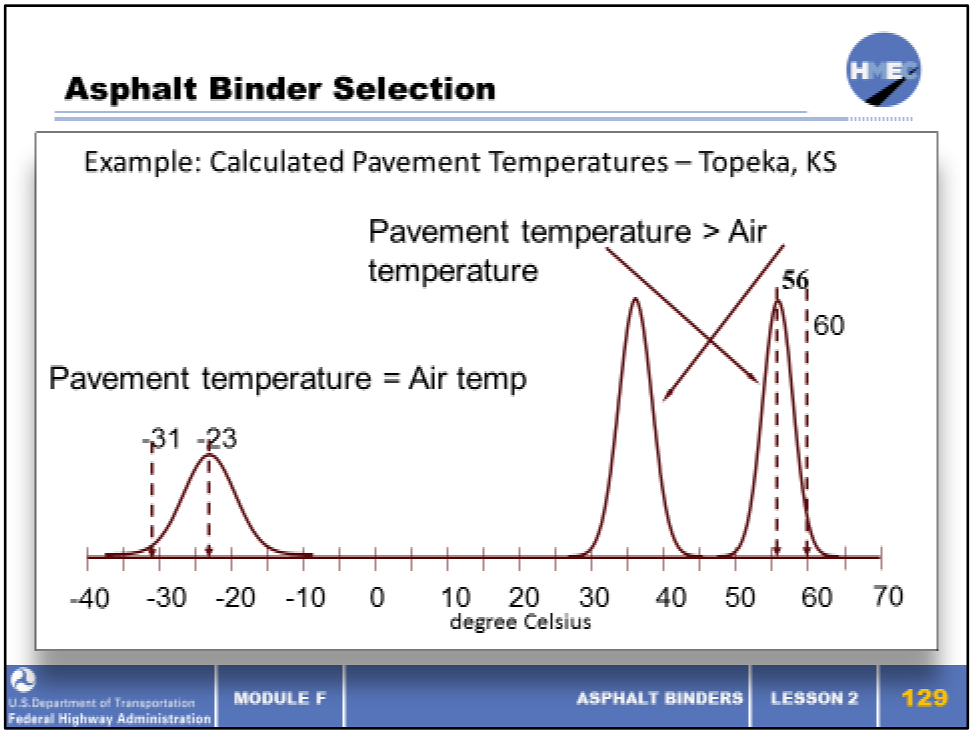
The LTPP software programs contain not only the necessary weather data but also the most current (August 2000) algorithms for converting this information into pavement temperatures.

The SHRP-A-648A report provides details of the development of the weather database, but the algorithms in this report should not be used for converting temperatures.

Several agencies have developed maps from the weather data that contain PG grading contours for easy selection of a binder grade (based on climatic conditions).

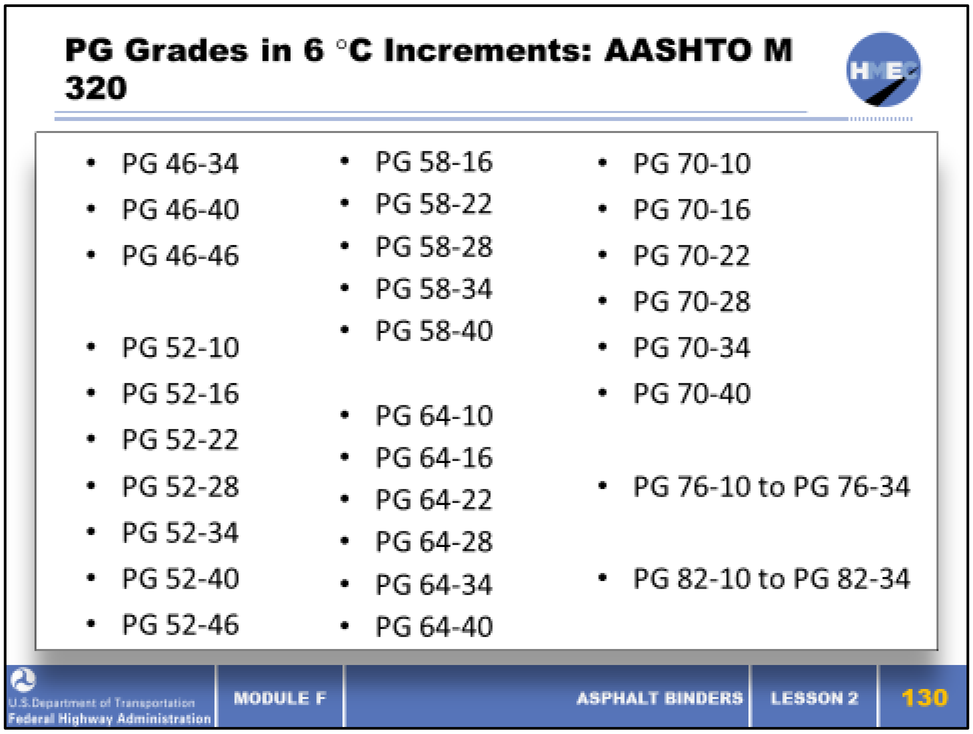
LTTP Bind software can be found at:
http://www.fhwa.dot.gov/research/tfhrc/programs/infrastructure/pavements/ltpp/dwnload.cfm

Slide 129



The software calculated pavement temperatures 20 °C greater than the air temperatures.

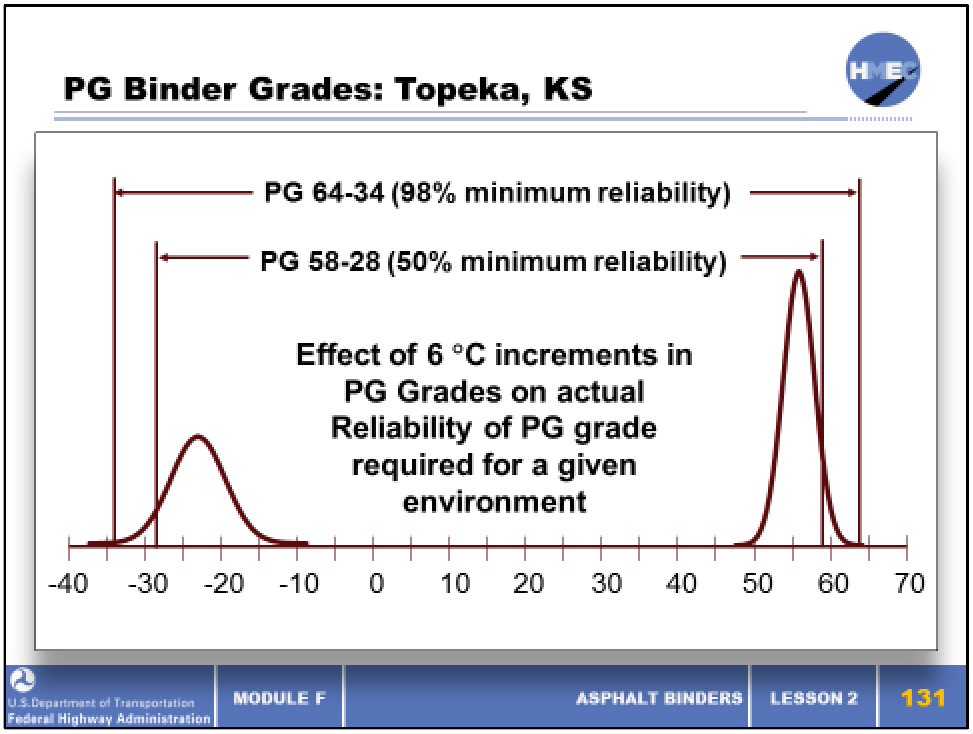
Slide 130



Why 6 degrees?

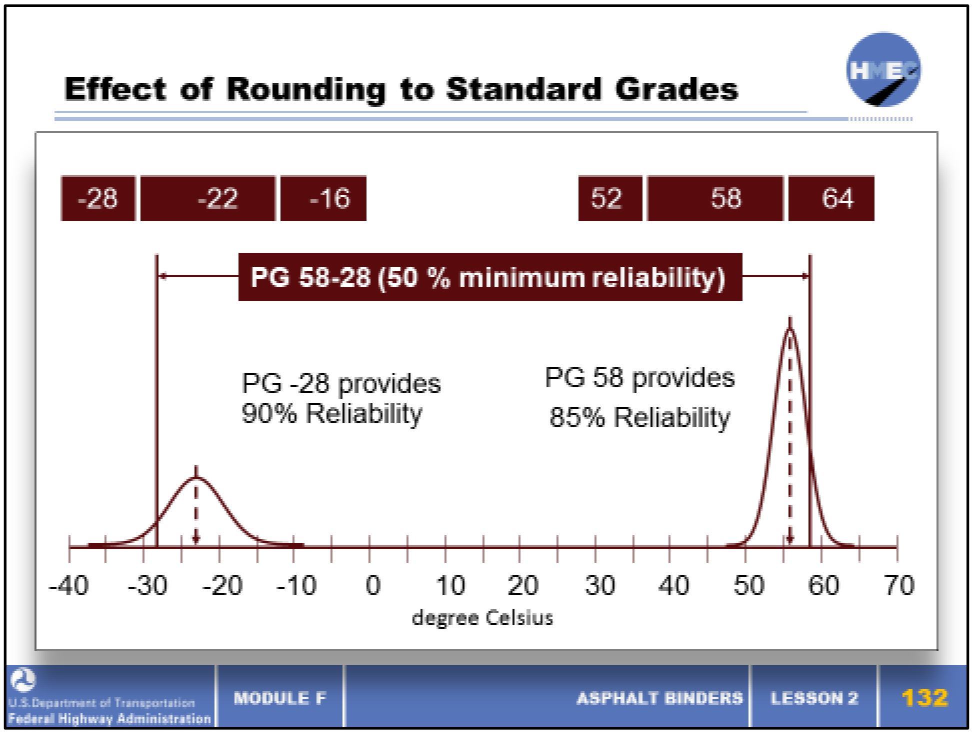
Six-degree difference between each performance grade was selected to control the number of PG grades that would be needed to cover the range of climate in the US. A 3 degree difference would have created twice as many PG grades and would require suppliers to stock many different grades for their region. This affects the tankage requirements. 6 degree difference provided the same amount of tankage that was required for earlier AC grades.

Slide 131



In this case, it is necessary to select the next highest grade for each of these temperatures, thus ensuring at least 50% or at least 98% reliability, respectively. Similarly, the next lower grade is selected for the low temperature.

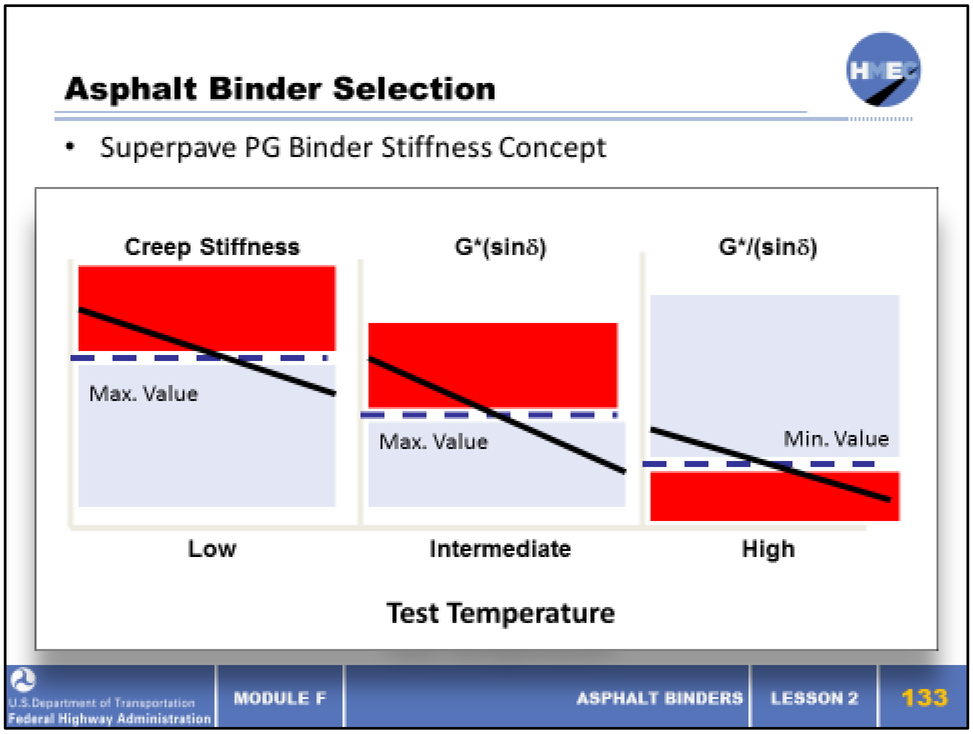
Slide 132



The effect of "rounding up" the 50% reliability temperature (i.e., 56 °C) for Topeka to the next grade (i.e., 58 °C) results in an actual reliability of 85%.

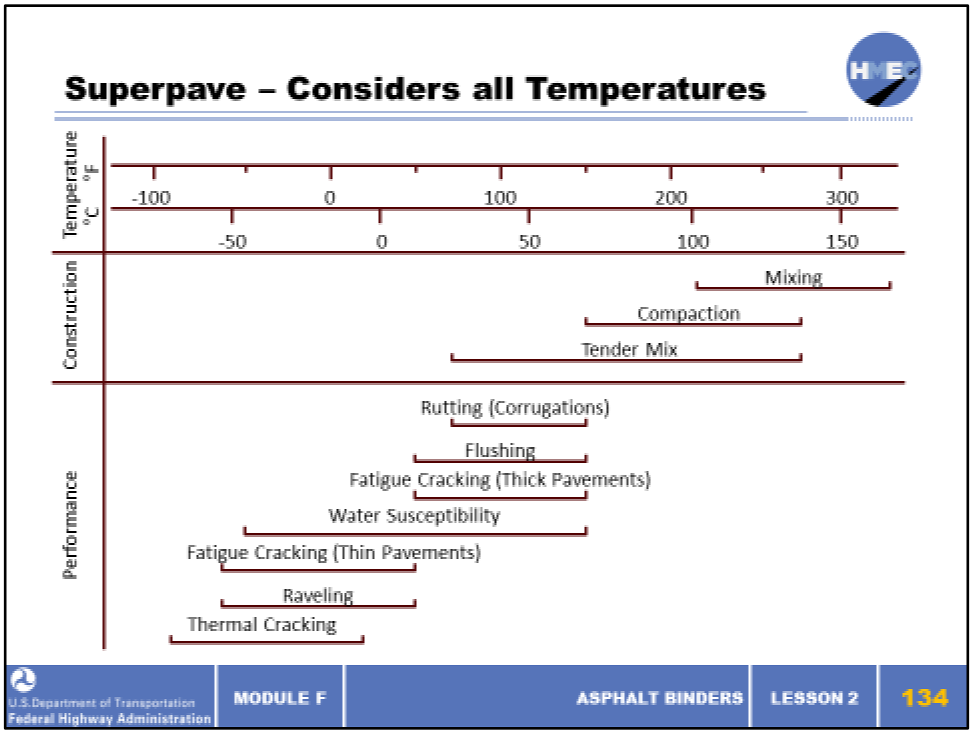
Similarly, "rounding down" the minimum temperature for 50% reliability (i.e., -23 °C) to the next lower grade (-28 °C) results in an actual reliability of 90%.

Slide 133



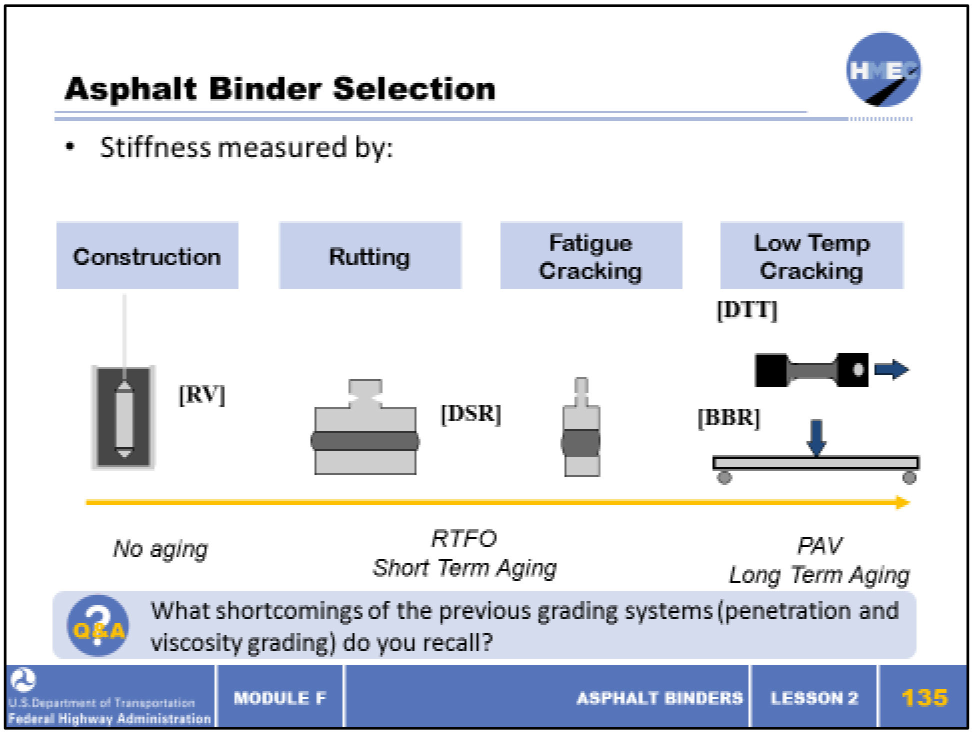
This slide provides a generic illustration of the asphalt binder stiffness concept for the specification. It is included only to illustrate the importance and background of stiffness in selecting asphalt binders. It will be discussed in greater detail in the next lesson in terms of the specification and test methods.

Slide 134



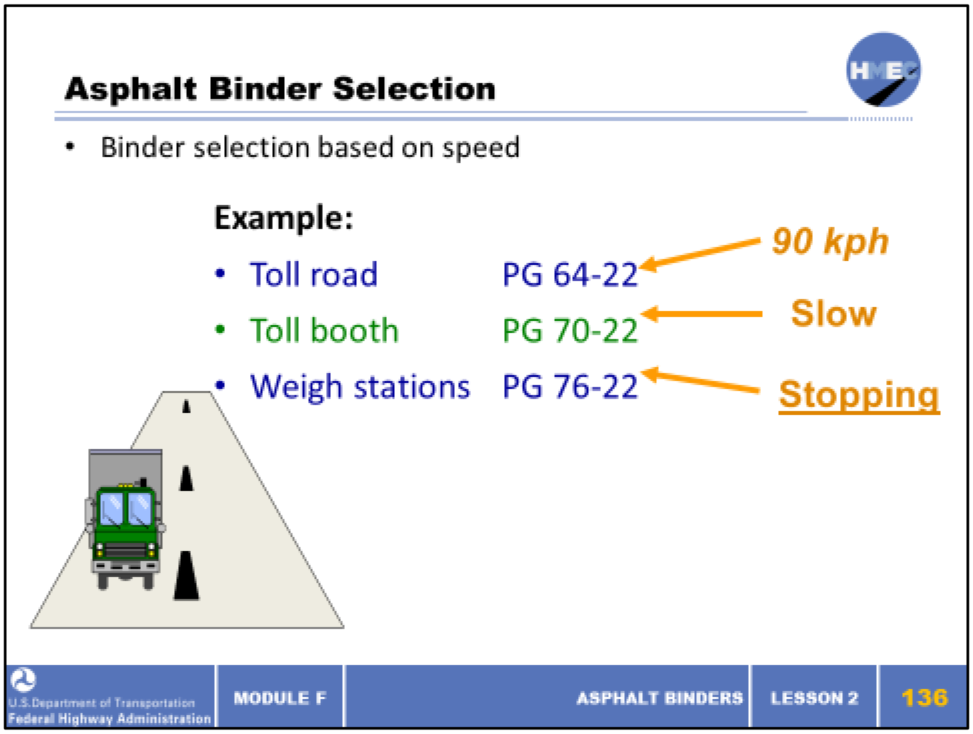
Asphalt binders are used during construction to temperatures approaching 175 °C (350 °F). In-service temperatures range from about -40 to 80 °C (-40 to 175 °F).

Slide 135



These test methods will be discussed in the next lesson and are only included here to show binder selection is not based on a single test or temperature. The next few slides will demonstrate and show the effect of traffic factors in binder selection.

Slide 136



The properties of asphalt binder depend on rate (or speed) of loading. Such materials are called viscoelastic materials. Later we will discuss more about viscoelastic properties of asphalt binder.

In general, stiffer (harder) binders should be used for projects where traffic speeds are slow to maximize permanent deformation resistance within the HMA pavement. However, this must be balanced with preserving the pavement's resistance to low temperature cracking.

Slide 137

Asphalt Binder Selection

- Binder selection based on number of applications
  - 10 to 30 x 10^6 ESAL
    - Consider increasing—one high temp. grade
  - >30 x 10^6 ESAL
    - Consider increasing—two high temp. grade

ESAL = Equivalent Single Axle Loads

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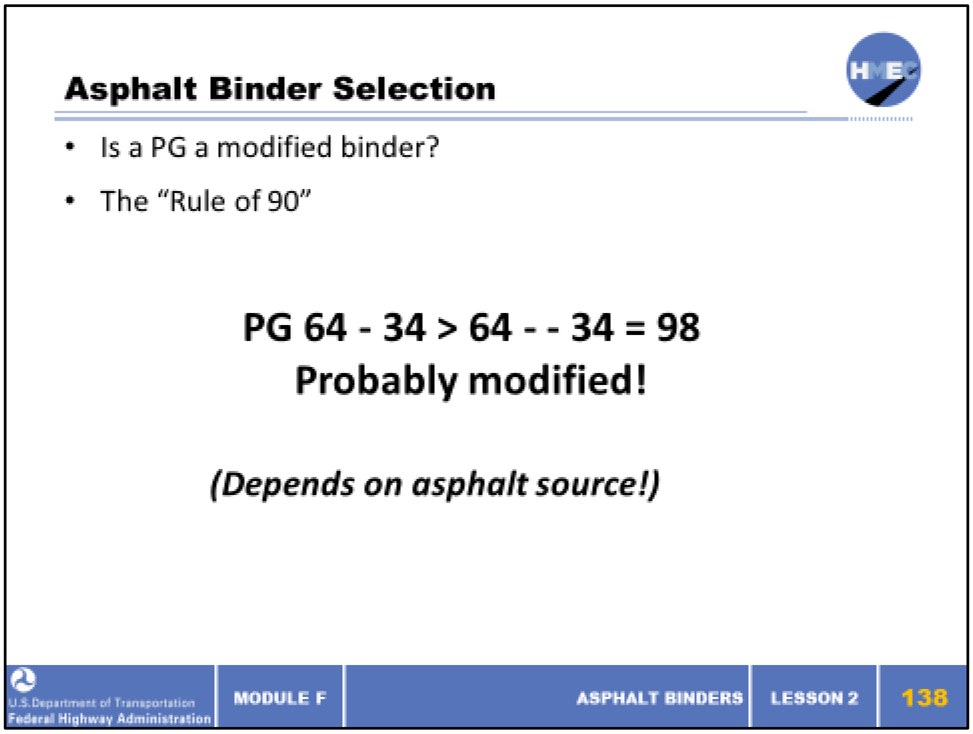
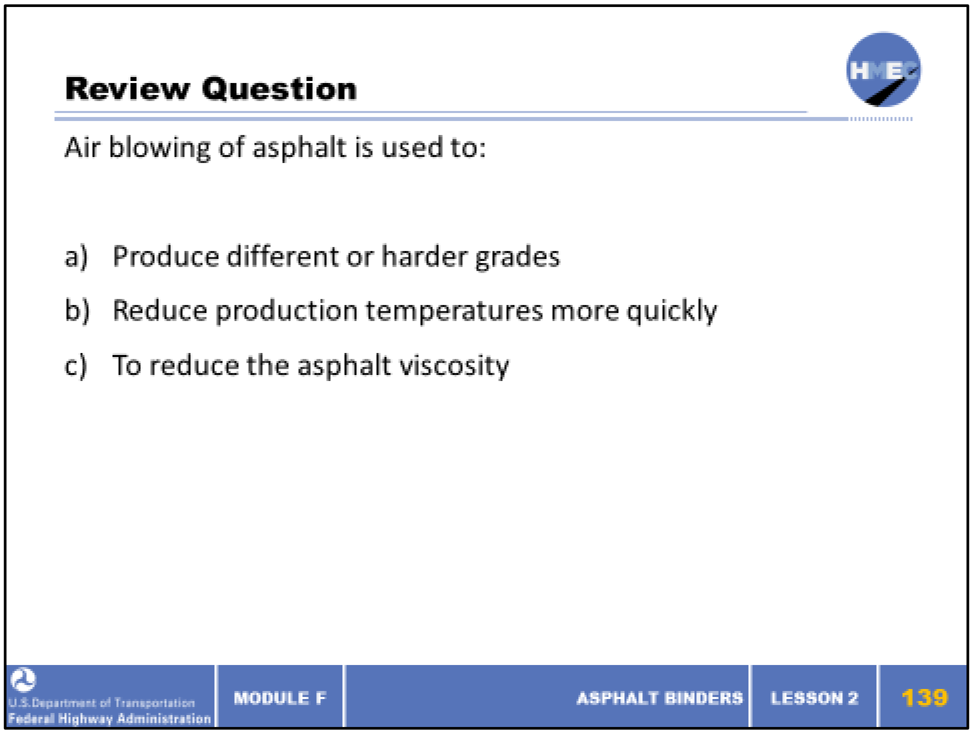
Slide 138



This slides presents the "Rule of 90." This rule is simply used to identify what binders might be modified and those not modified based on the PG or temperature values. However, whether the asphalt binder is modified based on temperature values for a specific stiffness in the binder specification depends on the asphalt source and other rheological properties of the asphalt.

Slide 139


Review Question
Air blowing of asphalt is used to:
a) Produce different or harder grades
b) Reduce production temperatures more quickly
c) To reduce the asphalt viscosity
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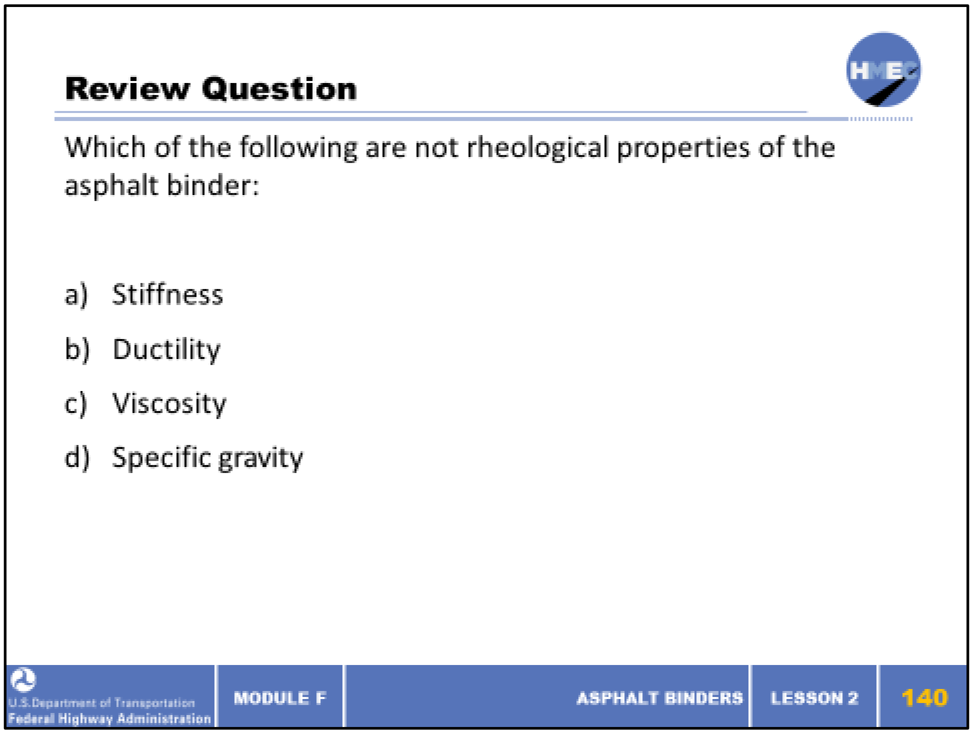
Slide 140


Review Question
HMEC
Which of the following are not rheological properties of the asphalt binder:
a) Stiffness
b) Ductility
c) Viscosity
d) Specific gravity
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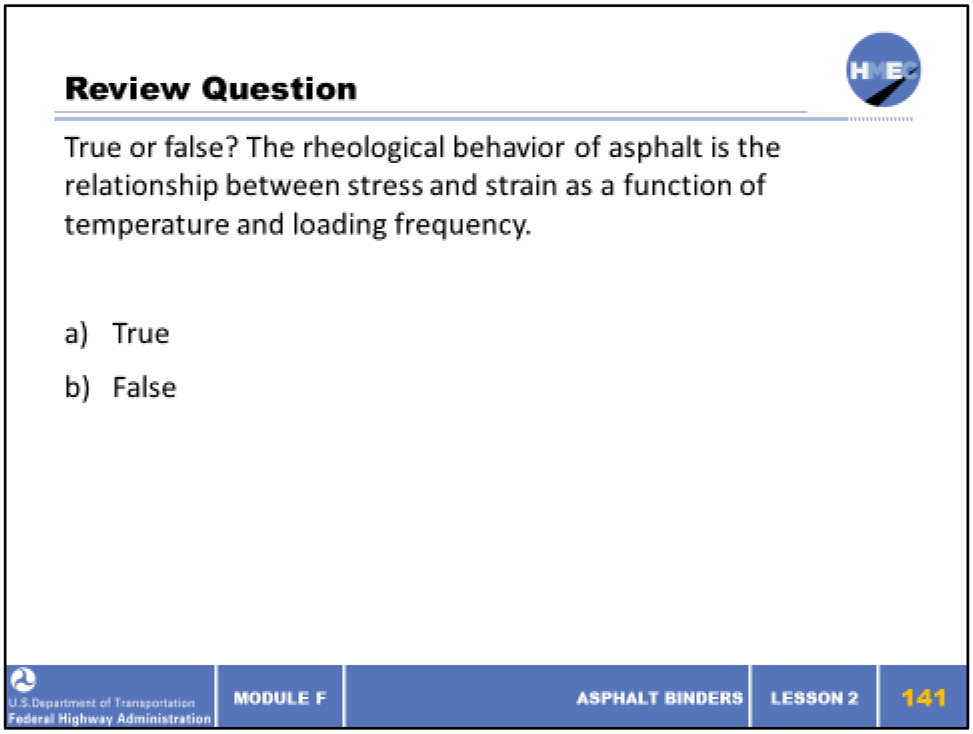
Slide 141


Review Question
HMEC
True or false? The rheological behavior of asphalt is the relationship between stress and strain as a function of temperature and loading frequency.
a) True
b) False
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Slide 142

Review Question

The fundamental consistency measurement of the resistance to flow and described by the ratio between applied shear stress and rate of shear strain is:

a) Penetration
b) Ductility
c) Viscosity
d) All of the above

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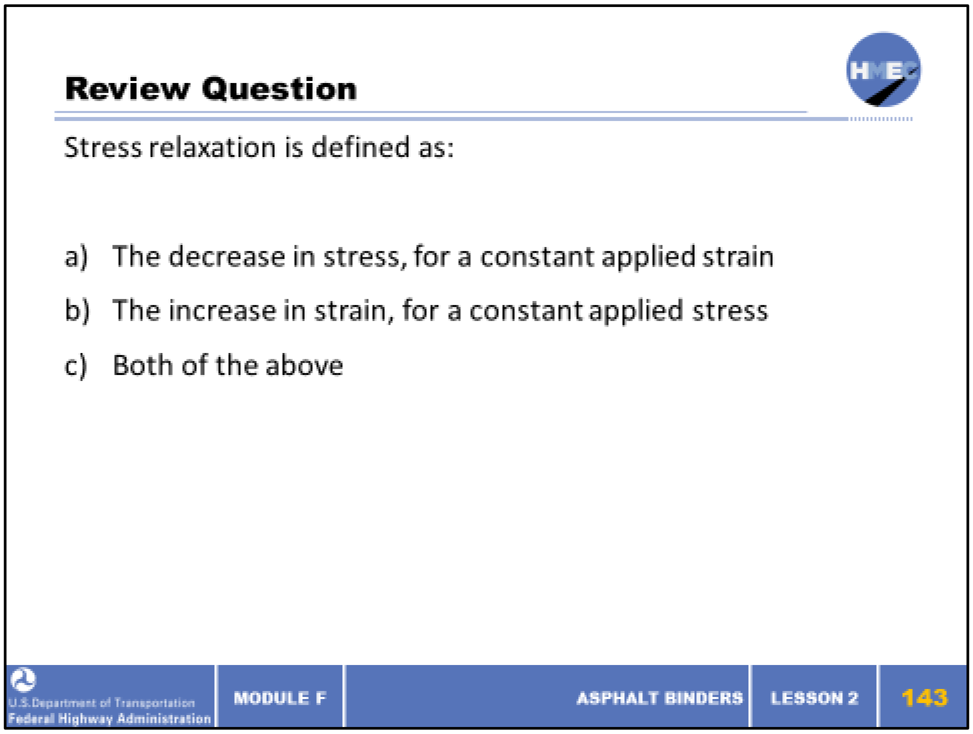
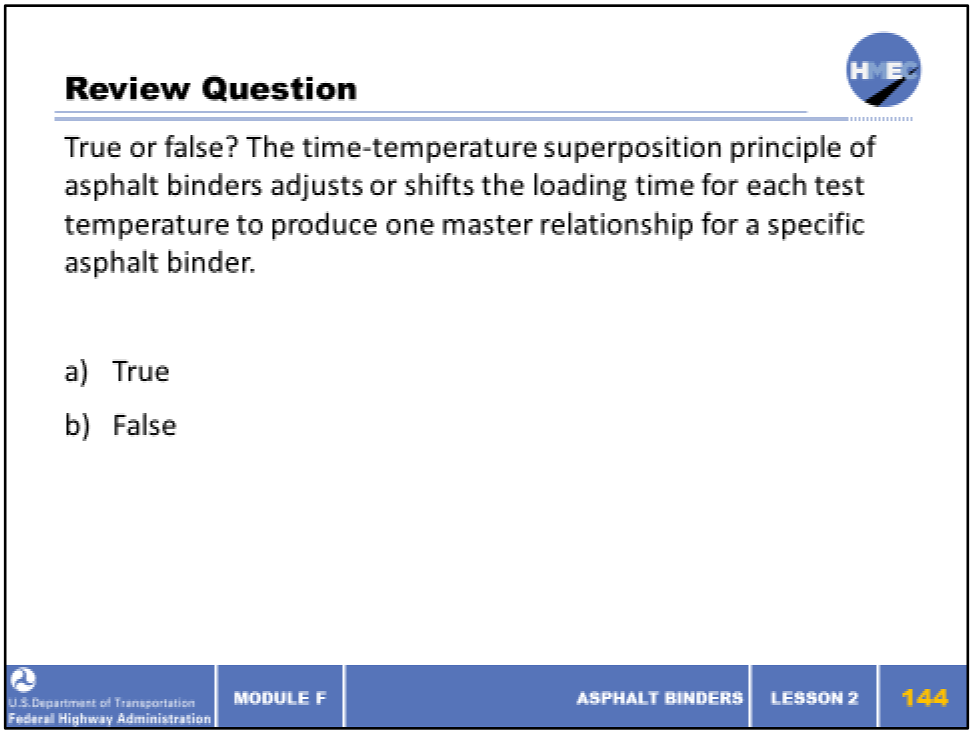
Slide 143


Review Question
HMEC
Stress relaxation is defined as:
a) The decrease in stress, for a constant applied strain
b) The increase in strain, for a constant applied stress
c) Both of the above
U.S. Department of Transportation
Federal Highway Administration
MODULE F
ASPHALT BINDERS
LESSON 2
143

Slide 144


Review Question
HMEC
True or false? The time-temperature superposition principle of asphalt binders adjusts or shifts the loading time for each test temperature to produce one master relationship for a specific asphalt binder.
a) True
b) False
U.S.Department of Transportation
Federal Highway Administration
MODULE F
ASPHALT BINDERS
LESSON 2
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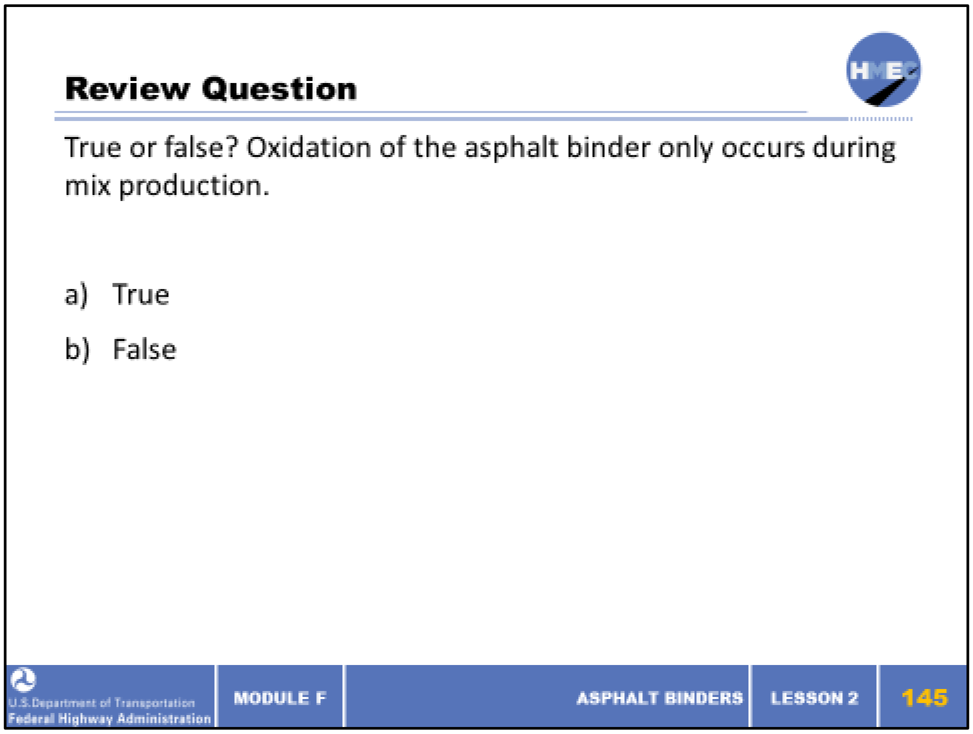
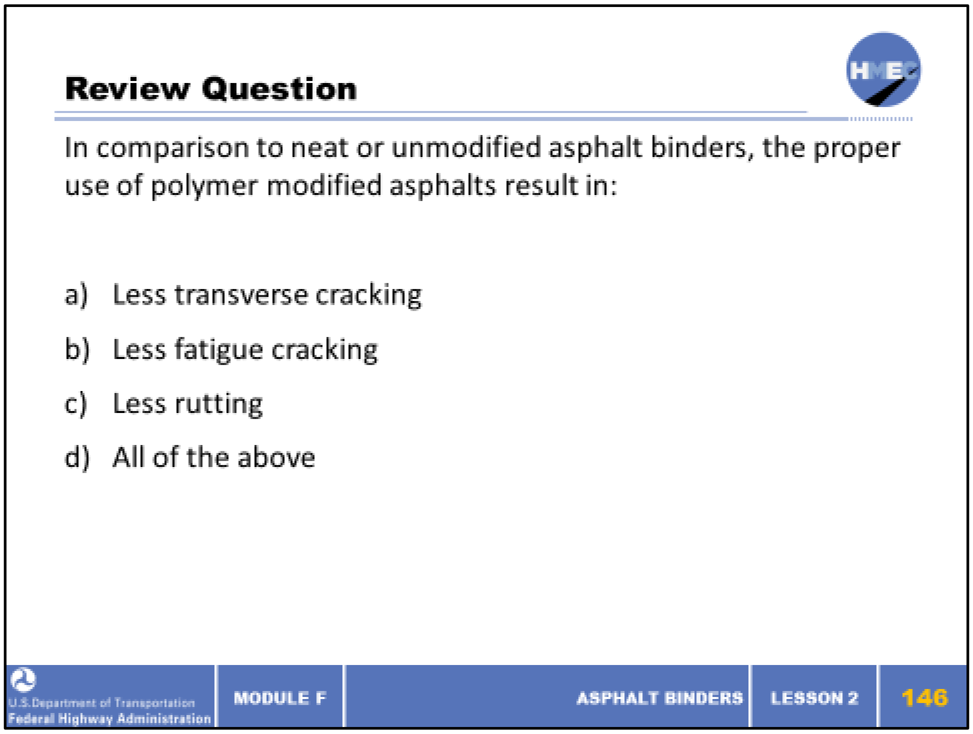

Review Question
True or false? Oxidation of the asphalt binder only occurs during mix production.
a) True
b) False
U.S. Department of Transportation
Federal Highway Administration
MODULE F
ASPHALT BINDERS
LESSON 2
145

Slide 146


Review Question
HMEC
In comparison to neat or unmodified asphalt binders, the proper use of polymer modified asphalts result in:
a) Less transverse cracking
b) Less fatigue cracking
c) Less rutting
d) All of the above
U.S. Department of Transportation
Federal Highway Administration
MODULE F
ASPHALT BINDERS
LESSON 2
146

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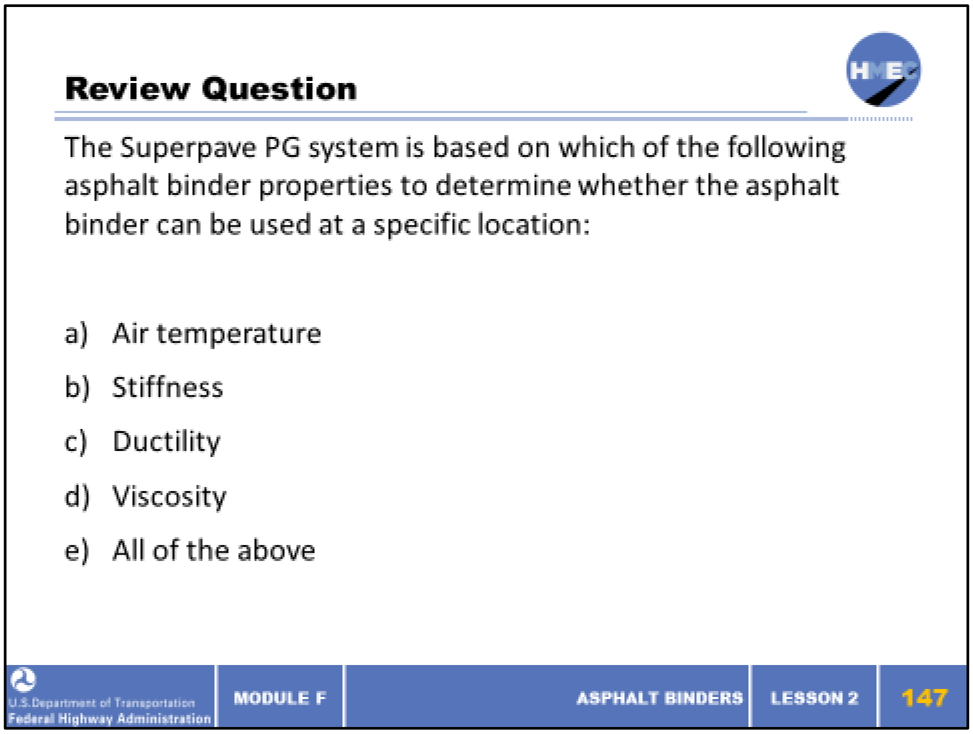
Review Question
HMEC
The Superpave PG system is based on which of the following asphalt binder properties to determine whether the asphalt binder can be used at a specific location:
a) Air temperature
b) Stiffness
c) Ductility
d) Viscosity
e) All of the above
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ASPHALT BINDERS
LESSON 2
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Review Question

Which of the following crumb rubber modified asphalt application processes relates to adding fine rubber particles to the asphalt prior to mixing with the aggregate?

a) Wet process

b) Dry process

c) Both processes

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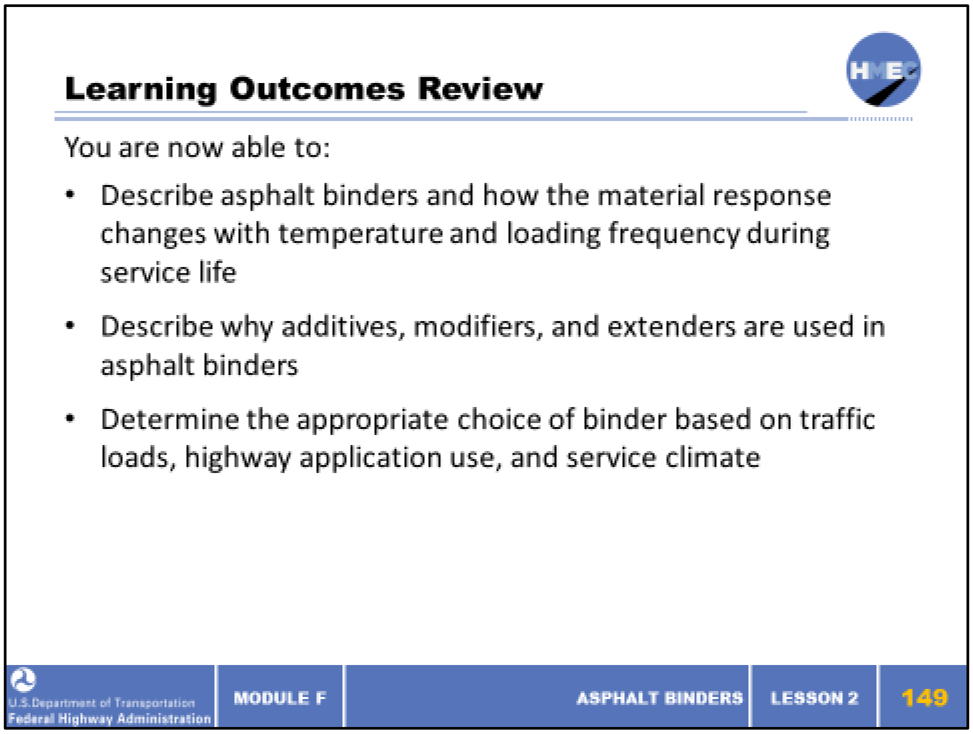
Learning Outcomes Review
You are now able to:
• Describe asphalt binders and how the material response changes with temperature and loading frequency during service life
• Describe why additives, modifiers, and extenders are used in asphalt binders
• Determine the appropriate choice of binder based on traffic loads, highway application use, and service climate
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ASPHALT BINDERS
LESSON 2
149

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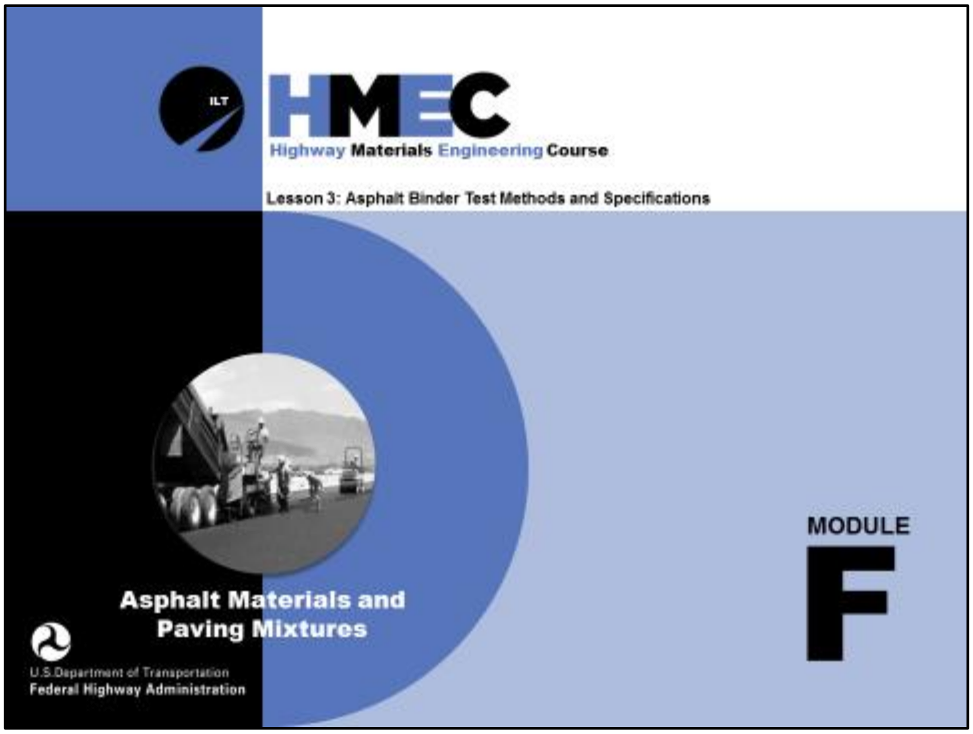
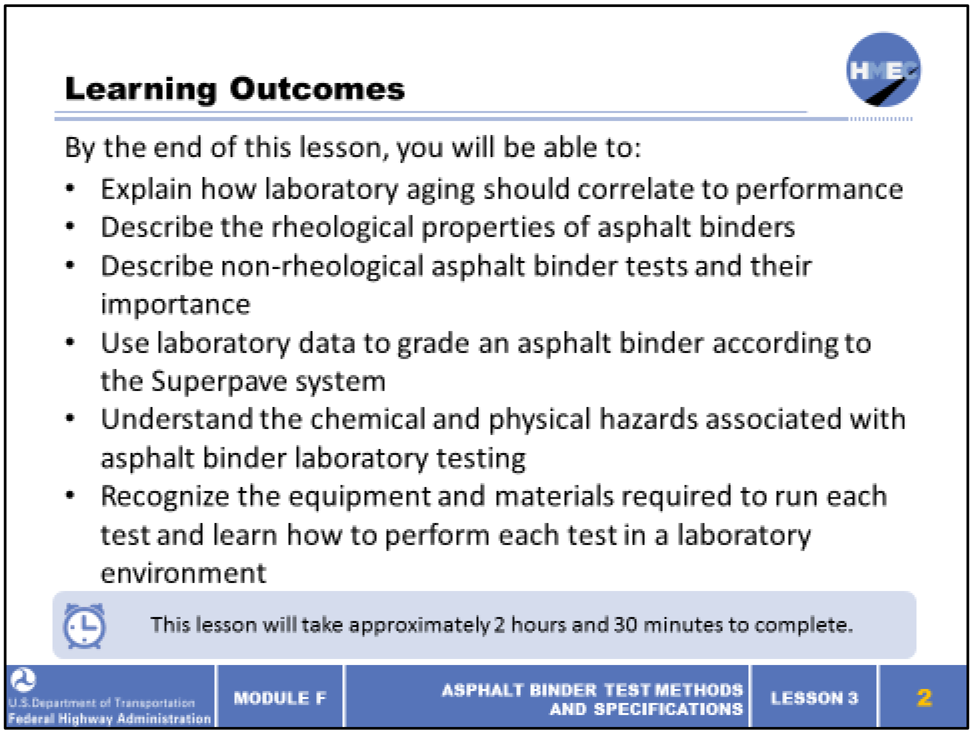
Slide 1


ILT
HMEC
Highway Materials Engineering Course
Lesson 3: Asphalt Binder Test Methods and Specifications
Asphalt Materials and Paving Mixtures
U.S.Department of Transportation
Federal Highway Administration
MODULE
F

Slide 2


Learning Outcomes
By the end of this lesson, you will be able to:
• Explain how laboratory aging should correlate to performance
• Describe the rheological properties of asphalt binders
• Describe non-rheological asphalt binder tests and their importance
• Use laboratory data to grade an asphalt binder according to the Superpave system
• Understand the chemical and physical hazards associated with asphalt binder laboratory testing
• Recognize the equipment and materials required to run each test and learn how to perform each test in a laboratory environment
This lesson will take approximately 2 hours and 30 minutes to complete.
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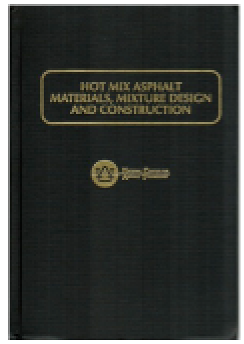
MODULE F
ASPHALT BINDER TEST METHODS AND SPECIFICATIONS
LESSON 3
2

Slide 3

Lesson 3 – Asphalt Binder Tests Methods and Specifications

- Laboratory simulation of age-hardening for asphalt binders
- Rheological property and consistency test methods
- Physical (non-rheological) property test methods
- Superpave performance grade specification
- Laboratory safety procedures and standards
- Laboratory testing and demonstration



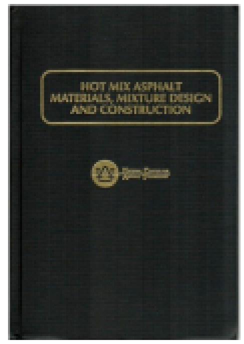
NCAT textbook, Chapter 2: Asphalt Refining, Uses, and Properties, pages 7 to 127.

U.S. Department of Transportation Federal Highway Administration | MODULE F | ASPHALT BINDER TEST METHODS AND SPECIFICATIONS | LESSON 3 | 3

Slide 4

Lesson 3 – Asphalt Binder Tests Methods and Specifications

- **Laboratory simulation of age-hardening for asphalt binders**
- Rheological property and consistency test methods
- Physical (non-rheological) property test methods
- Superpave performance grade specification
- Laboratory safety procedures and standards
- Laboratory testing and demonstration



NCAT textbook, Chapter 2: Asphalt Refining, Uses, and Properties, pages 7 to 127.

U.S. Department of Transportation Federal Highway Administration | MODULE F | ASPHALT BINDER TEST METHODS AND SPECIFICATIONS | LESSON 3 | 4

Slide 5

Discussion: Laboratory Simulation of Age Hardening

1. Why do we try to simulate the age hardening of asphalt binders in the laboratory?
2. What parameters affect the simulation of age hardening?

Q&A Let's discuss the questions on the screen.

U.S. Department of Transportation Federal Highway Administration | MODULE F | ASPHALT BINDER TEST METHODS AND SPECIFICATIONS | LESSON 3 | 5

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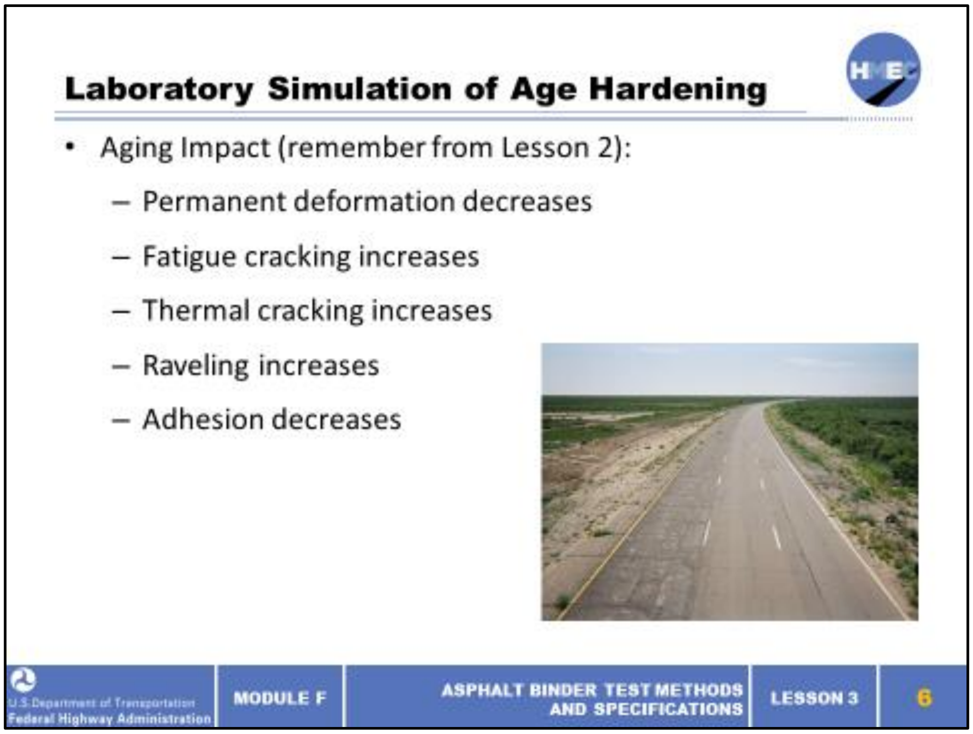


Aging of the asphalt binder impacts the resistance of the HMA mixture to permanent deformation, fatigue cracking, thermal cracking, and moisture sensitivity. The factors that affect how an asphalt binder will harden and age are oxidation, volatilization, polymerization, thixotropic or steric hardening, syneresis, and separation.

Slide 7

The viscosity of asphalt binders significantly increases during mixture production and continually increases over time but at a much slower rate with the age of pavement. This age hardening can affect the properties of an asphalt binder significantly to cause deterioration.

A variety of tests have been developed to determine the durability characteristics of asphalt binders. Those that are specified in the PG binder specification include the rolling thin film oven (RTFO) test and the pressure aging vessel (PAV). The rolling thin film oven test was developed to simulate the hardening that results during hot mixing. The pressure aging vessel was developed to simulate 7 to 10 years of in-service aging.

Remember back to the earlier lesson topic on asphalt chemistry:

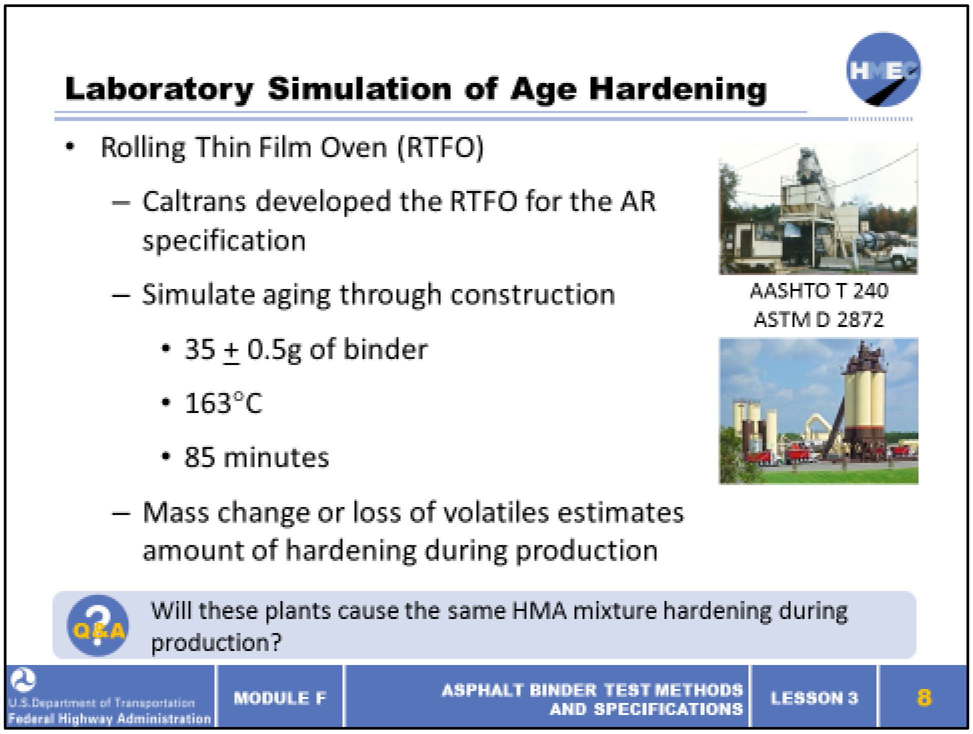
• Asphaltenes are the component of asphalt primarily responsible for the viscosity (consistency) of the asphalt.
• Maltenes are the oily medium that act as dispersants (peptizers) for the asphaltenes.

As asphalt ages, more asphaltenes are formed (large molecular weight agglomerate) due to increasing associations formed during oxidation. Asphaltenes are hard brittle solids when isolated and consist primarily of hydrocarbons.

Maltenes are comprised of oils and resins.

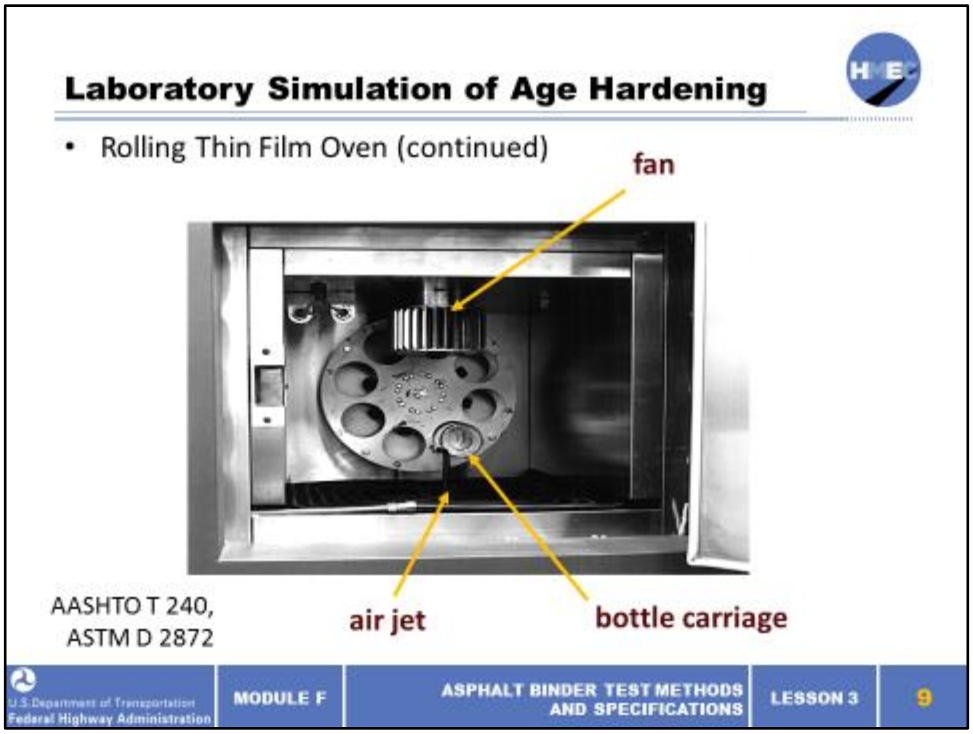
A number of factors contribute to this hardening with time, including oxidation, volatilization, polymerization, thixotropy, separation, and syneresis.

Slide 8



A modification of this test (99 °C (210 °F) for 168 hours) may represent field hardening to periods of 5 years.

Slide 9



A fan circulates air inside the oven and that a jet of air is blown over the binder while it is being aged.

Also note that the bottle carriage allows aging of up to 8 samples at a time. The carriage turns around at a factory set fixed rate. There is an air jet positioned directly at the center of each bottle as it passes the air jet location. This allows a predetermined and consistent amount of pre-heated air into each bottle each time it passes the air jet location.

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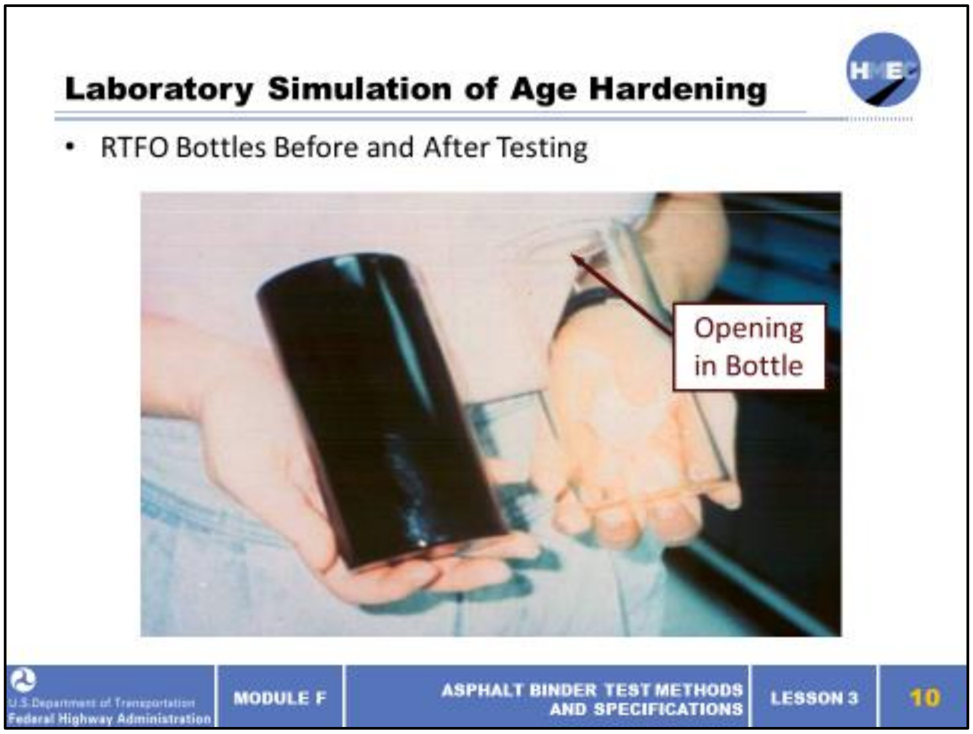
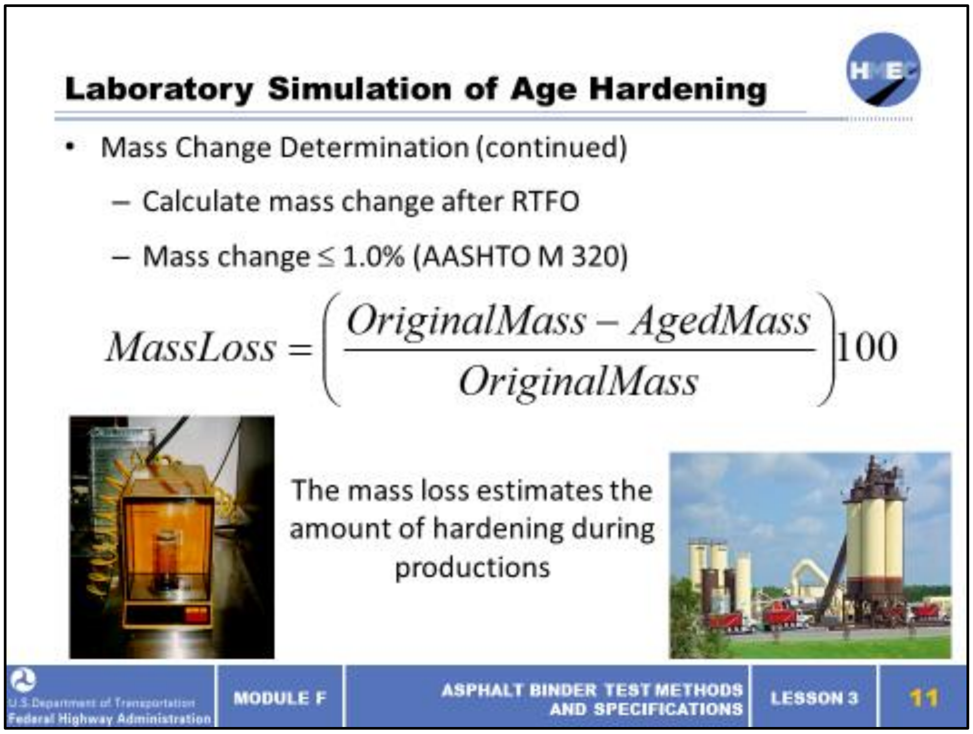
Slide 10



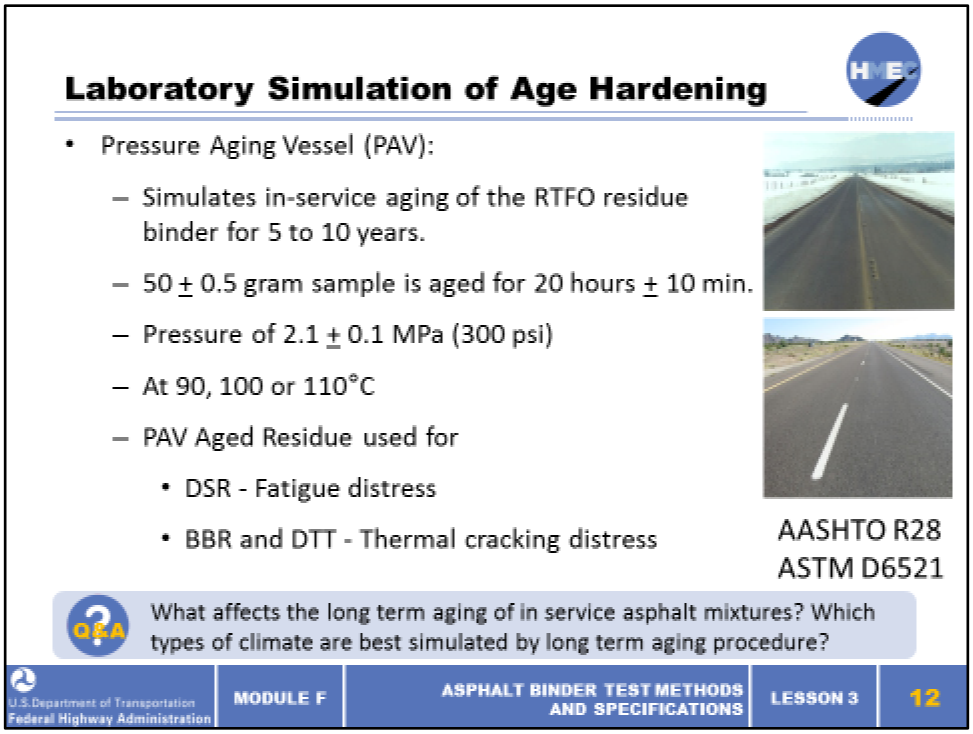
This may be obtained from the asphalt binder laboratory where the course is being conducted for the purpose of demonstration and then returned.

Slide 11



The equation shows how to determine the percent change in mass. According to the PG binder specification, this cannot exceed 1.0% (or 0.35 g if a 35 g sample was used). While most asphalt binders will lose mass during this simulated aging process, occasionally some asphalt binders will gain mass while aged in the RTFO.

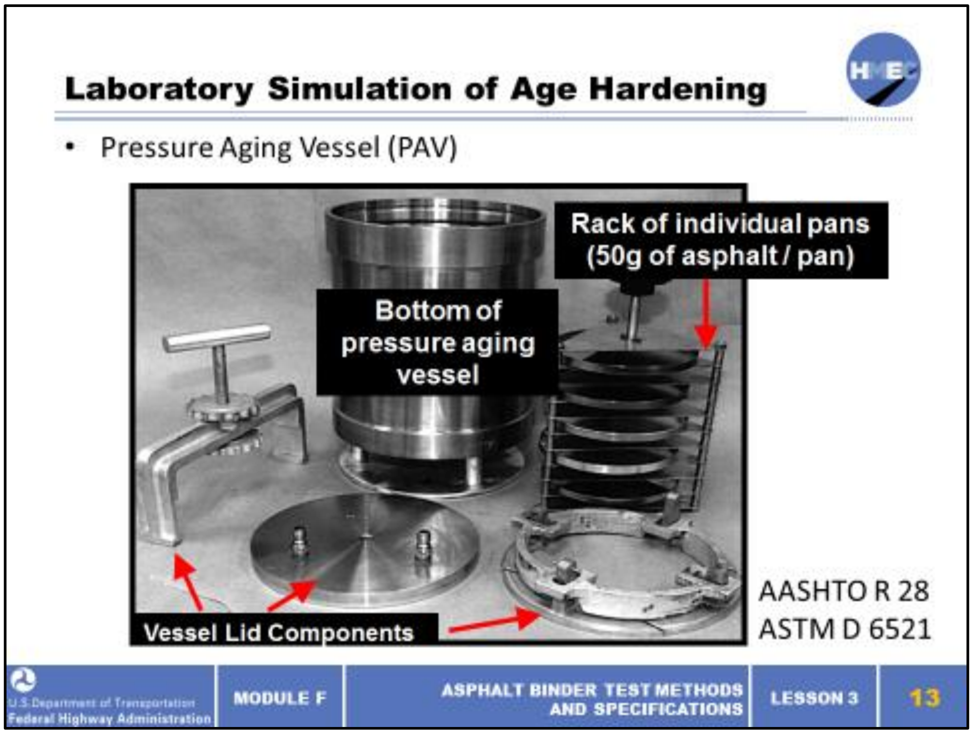
Slide 12



In this procedure a 50 +/- 0.5 g sample is aged for 20 hours +/- 10 min. under a pressure of 2.1 +/- 0.1 MPa (300 psi) and at an elevated temperature of 90, 100, or 110°C (194, 212, or 230°F), depending on the mean maximum weekly pavement temperature expected for the HMA pavement.

Reason for the PAV Test: The PAV test simulates field aging of 5 to 10 years and produces sufficient amount of PAV aged binder to perform DSR, BBR, and the DTT tests.

Slide 13



Approximately 50 +/- 0.5 g of binder that has been aged in the RTFO are placed on shallow pans that fit into a rack. Once loaded, the rack is placed in the pressure vessel, charged with air pressure, and heated at the desired aging temperature for 20 hours +/- 10 min.

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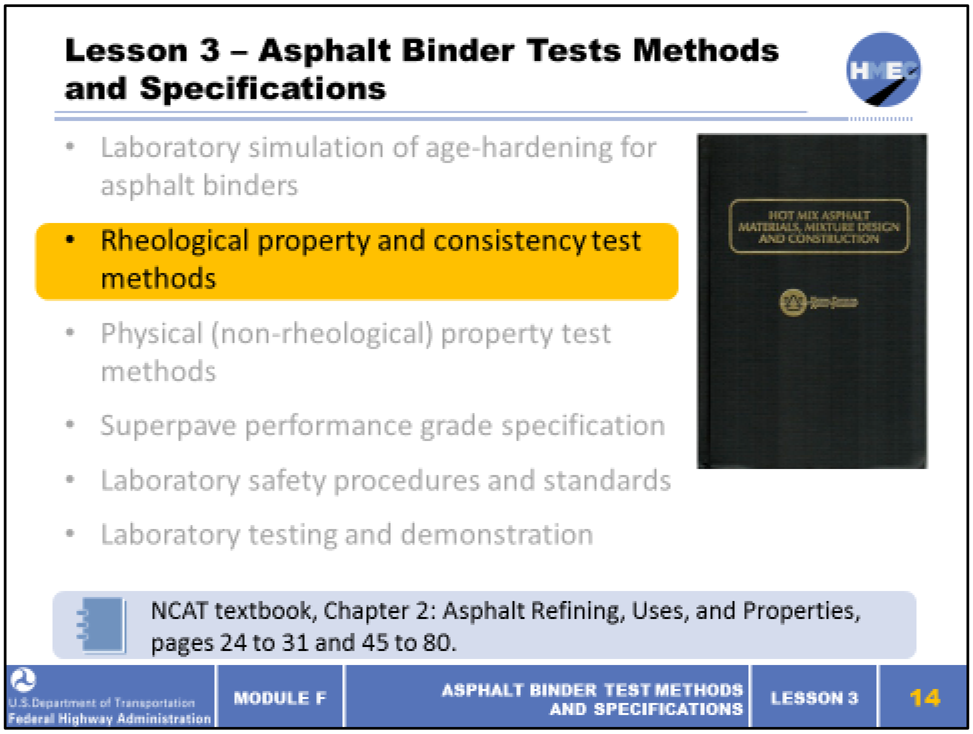
Lesson 3 – Asphalt Binder Tests Methods and Specifications
HMEC
Laboratory simulation of age-hardening for asphalt binders
Rheological property and consistency test methods
Physical (non-rheological) property test methods
Superpave performance grade specification
Laboratory safety procedures and standards
Laboratory testing and demonstration
HOT MIX ASPHALT MATERIALS, MIXTURE DESIGN AND CONSTRUCTION
NCAT textbook, Chapter 2: Asphalt Refining, Uses, and Properties, pages 24 to 31 and 45 to 80.
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MODULE F
ASPHALT BINDER TEST METHODS AND SPECIFICATIONS
LESSON 3
14

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Rheological Property and Consistency Tests

- Consistency Tests
  – Rotational viscometer; viscosity
- Rheological Property Tests
  – Dynamic shear rheometer; shear modulus
  – Bending beam rheometer; creep modulus
- Strength Property Test
  – Direct tension test

U.S. Department of Transportation Federal Highway Administration | MODULE F | ASPHALT BINDER TEST METHODS AND SPECIFICATIONS | LESSON 3 | 15

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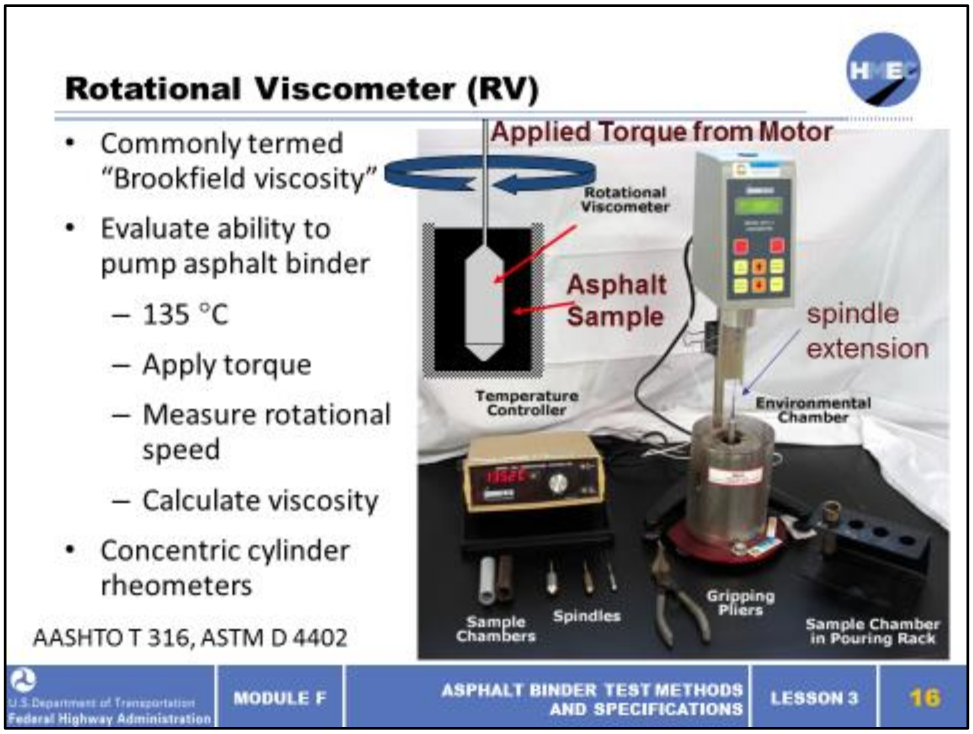


The temperature control chamber, also called the "thermosel" is specially designed to maintain the temperature of the sample at the desired temperature.

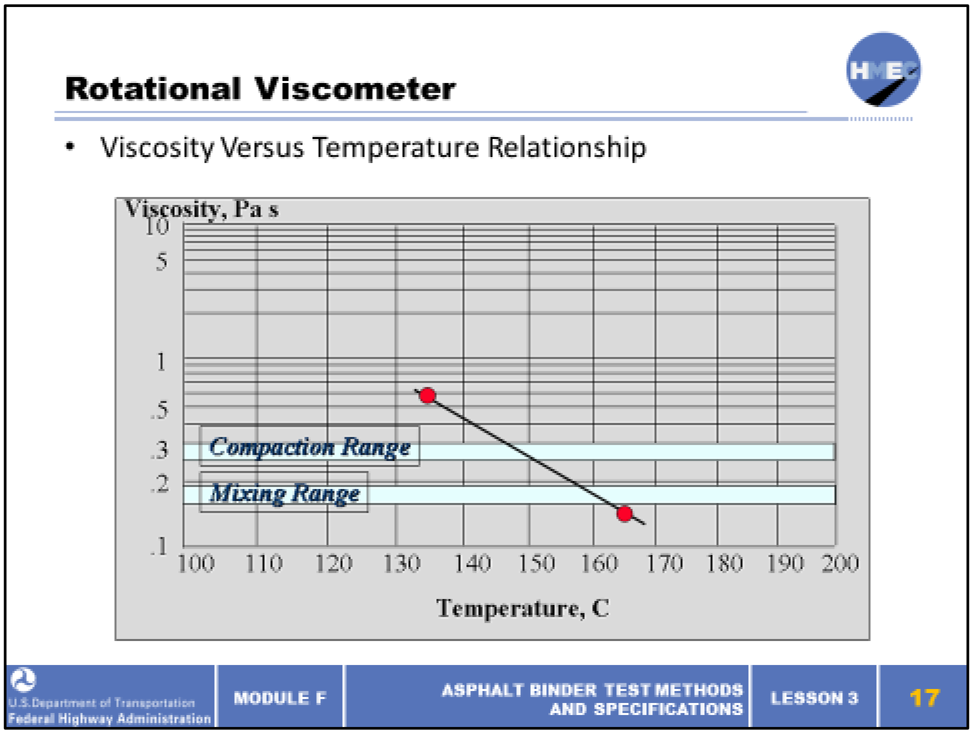
Digital readout – reads test results and parameters digitally
- Control keys – keys that control the functioning of the Rheometer
- Thermo container – controls temperature of the sample
- Spindle extension – attaches the spindle to lower into the chamber
- Temperature controller – controls temperature

AASHTO T316 Definition: As defined in Lesson 2, the ratio between the applied shear stress and the rate of shear is called the coefficient of viscosity. This coefficient is a measure of the resistance to flow of the liquid. It is commonly called the viscosity. The Système International d'Unités or International System of Units (SI) unit of viscosity is the Pascal second (Pa·s).

The PG binder specification requires testing at 135 °C (275 °F) and specifies a maximum viscosity of 3 Pa·s at this temperature. Binders with values greater than 3 Pa·s indicate that it would be difficult to pump the binder and construct a hot mix asphalt (HMA) pavement layer using such a binder.

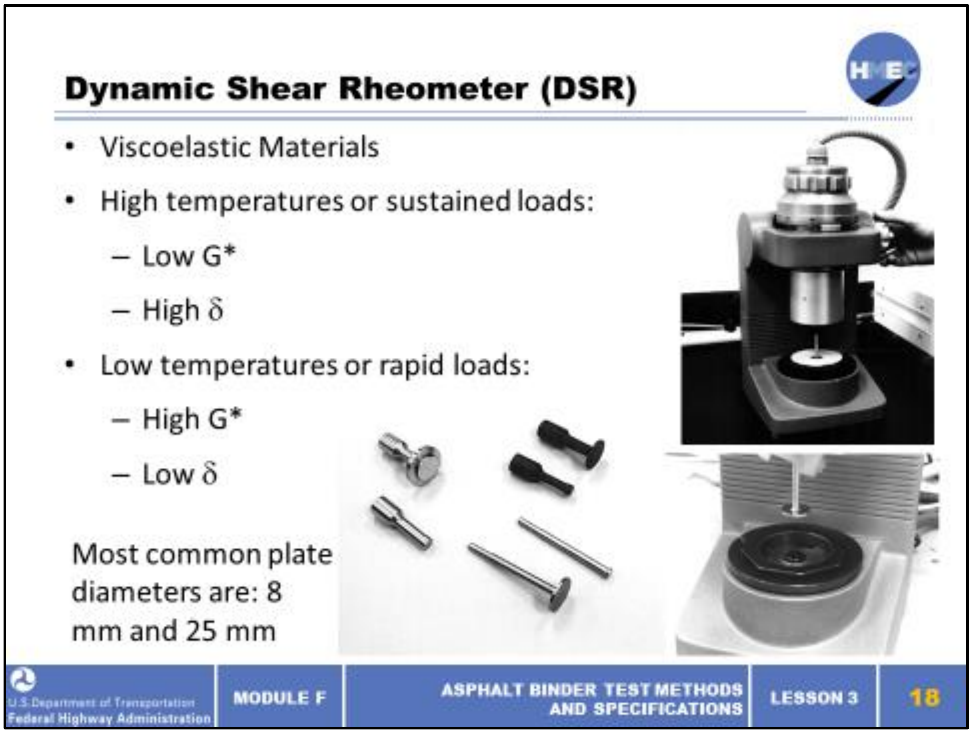
Relationship to performance: This test (RV) is used to satisfy the pumpability requirements in the performance graded (PG) asphalt binder specification.

Slide 17



Two popular units of viscosity include Stoke and Poise. The cgs unit for viscosities determined using a viscometer and only gravitational forces is cm^2 and called a stoke (St). The SI unit of viscosity is a Pascal-second and is equivalent to 10 poises. The units are necessary on this plot because this straight line relationship is only true when data is plotted in the units shown. However, when polymer modifiers are added to asphalt binder, the behavior may not be linear. In this case, further testing should be done by the supplier to determine suitable mixing and compacting ranges.

Slide 18



The dynamic shear rheometer has a parallel plate configuration. The stress and strain measurements are based on the assumption of a cylindrical geometry. This is why a great deal of effort is expended in preparing and trimming the specimen prior to starting the test. Not shown are the control unit, the temperature-controlled water circulation unit, and the computer and software used to command the DSR and obtain test data.

The asphalt binder sample is submerged in water or ethylene glycol for very accurate temperature control; hence the need for the water circulation unit. This is specific to this particular DSR. Some newer models of rheometers utilize an air bath for temperature control.

At high temperatures or sustained (slowly applied) loads, the binder stiffness is relatively low, whereas the phase angle is relatively high.

At low temperatures or rapid loads, the opposite is true.

Viscoelasticity is the property of materials that exhibit both viscous and elastic characteristics when undergoing deformation. Viscous materials, like honey, resist shear flow and strain linearly with time when a stress is applied. Elastic materials strain instantaneously when stretched and just as quickly return to their original state once the stress is removed.

Viscoelastic materials have elements of both of these properties and, as such, exhibit time dependent strain. Whereas elasticity is usually the result of bond stretching along crystallographic planes in an ordered solid, viscoelasticity is the result of the diffusion of atoms or molecules inside of an amorphous material.

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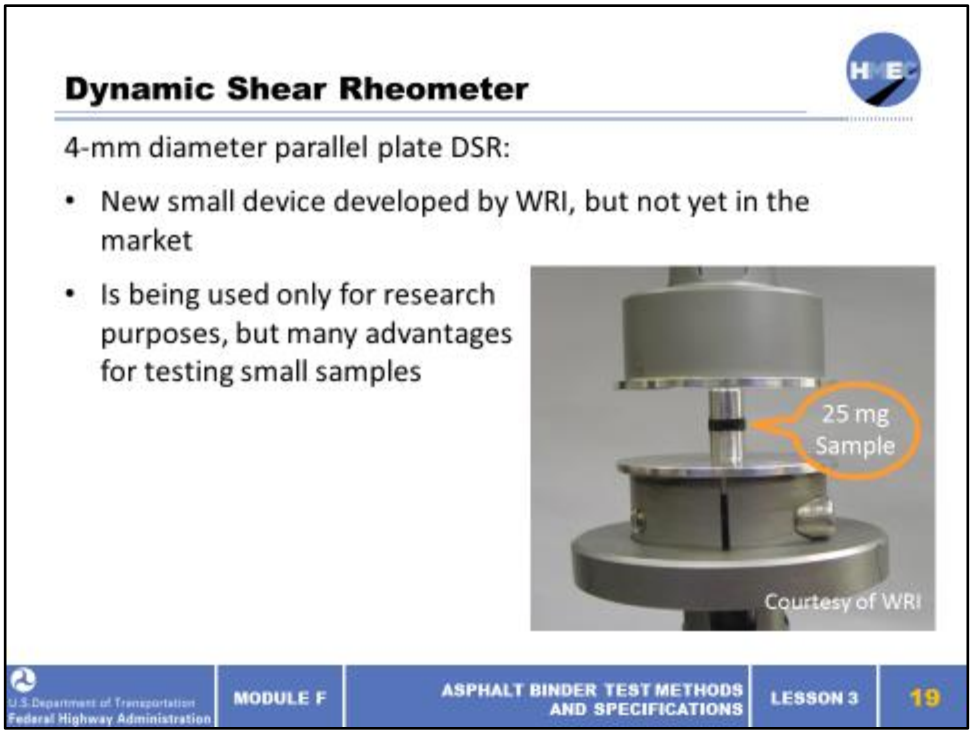
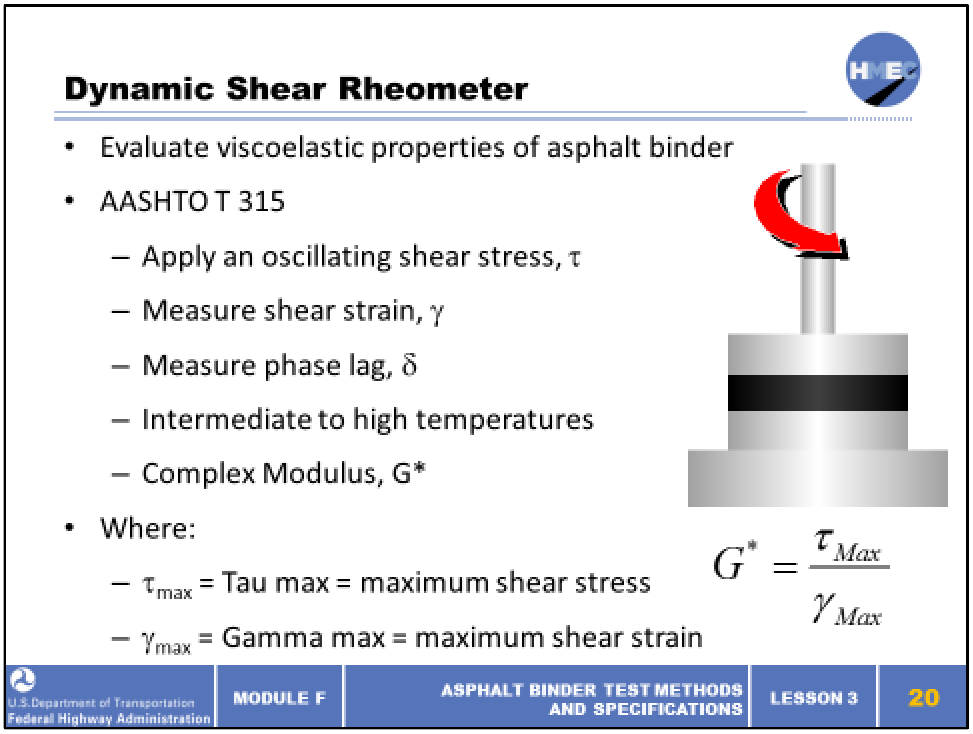


The new DSR is termed the 4-mm diameter parallel plate, and was developed by the Wyoming Research Institute. Currently, the 4-mm diameter parallel plate DSR is only being used for research purposes, but has many advantages and capabilities because of the small sample sizes. The small DSR has the same test capabilities as the traditional DSRs and is being refined. The small samples allow the complex shear modulus to be measured on binders recovered from very thin layers, such as seal coats, surface treatments, and other thin layers.

Slide 20

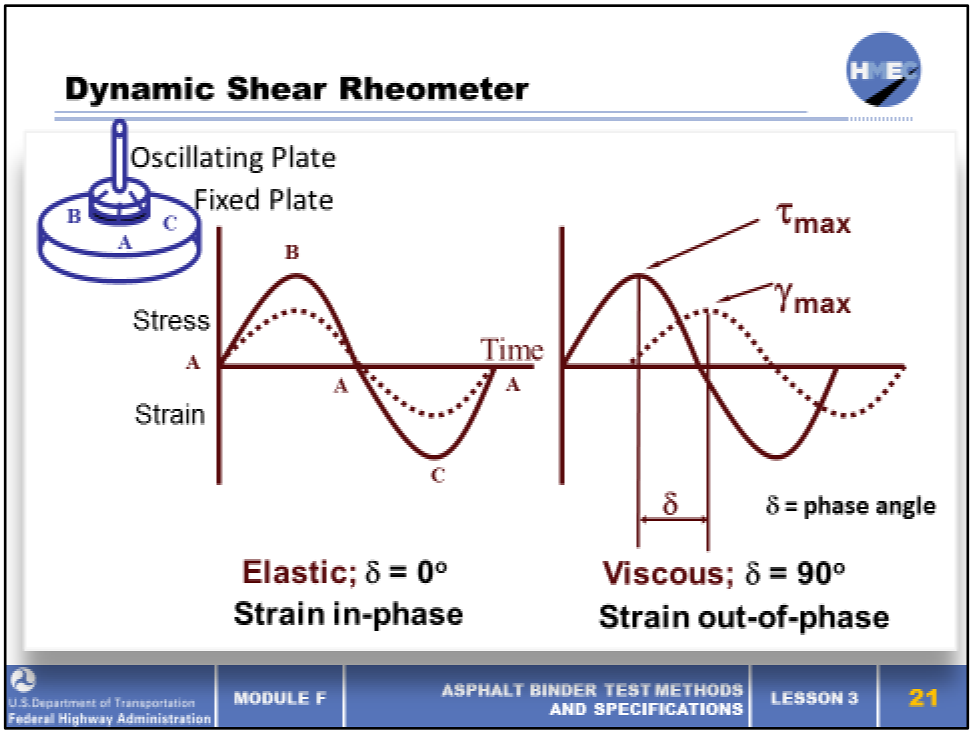


This device applies an oscillating shear stress to an asphalt binder sandwiched between a fixed base plate and a spindle and is used to measure the resultant shear strain and phase lag between the applied stress and resultant shear strain.

It is used to determine the dynamic shear modules and phase angle of asphalt binder when tested in dynamic (oscillatory) shear using parallel plate geometry.

The absolute value of the complex shear modulus is determined by dividing the maximum applied shear stress by the maximum resultant shear strain.

Slide 21



Recall the definitions for viscoelastic materials. Recall the definitions for viscoelastic materials. Viscoelasticity is the property of materials that exhibit both viscous and elastic characteristics when undergoing deformation. Viscous materials, like honey, resist shear flow and strain linearly with time when a stress is applied. Elastic materials strain instantaneously when stretched and just as quickly return to their original state once the stress is removed.

Viscoelastic materials have elements of both of these properties and, as such, exhibit time dependent strain. Whereas elasticity is usually the result of bond stretching along crystallographic planes in an ordered solid, viscoelasticity is the result of the diffusion of atoms or molecules inside of an amorphous material.

This figure schematically explains the following: If a material is purely elastic, then the strain response will be in phase with the applied stress. If a material is purely viscous, then the response will be 90° out of phase.

• Test operates at 10 rad/s or 1.59 Hz
• 360° = 2π radians per circle; 1 rad = 57.3°

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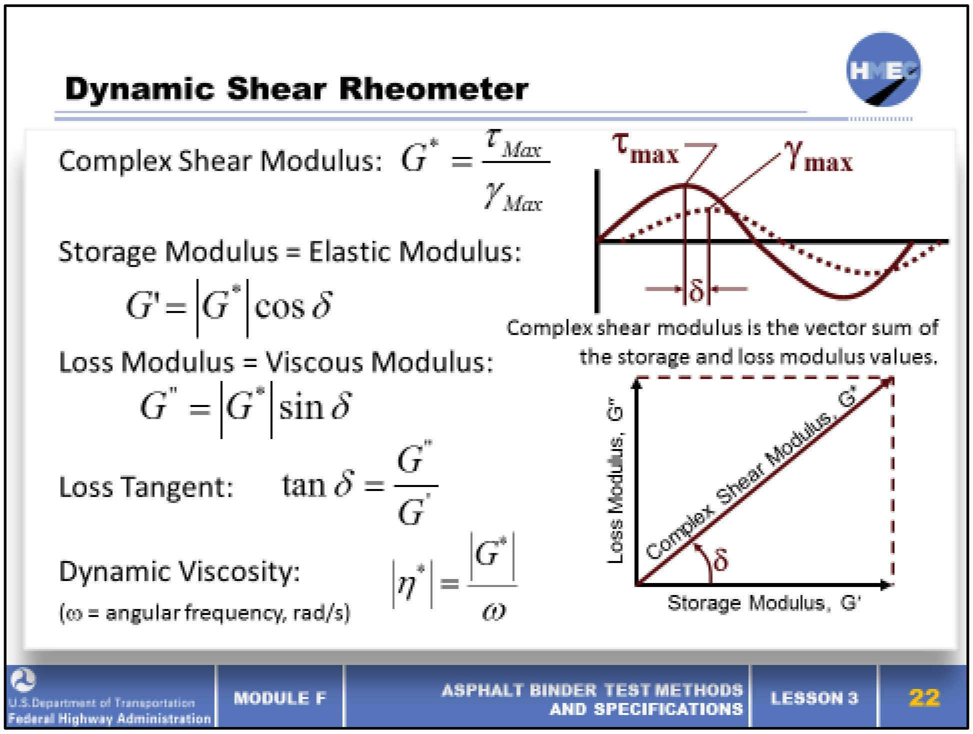
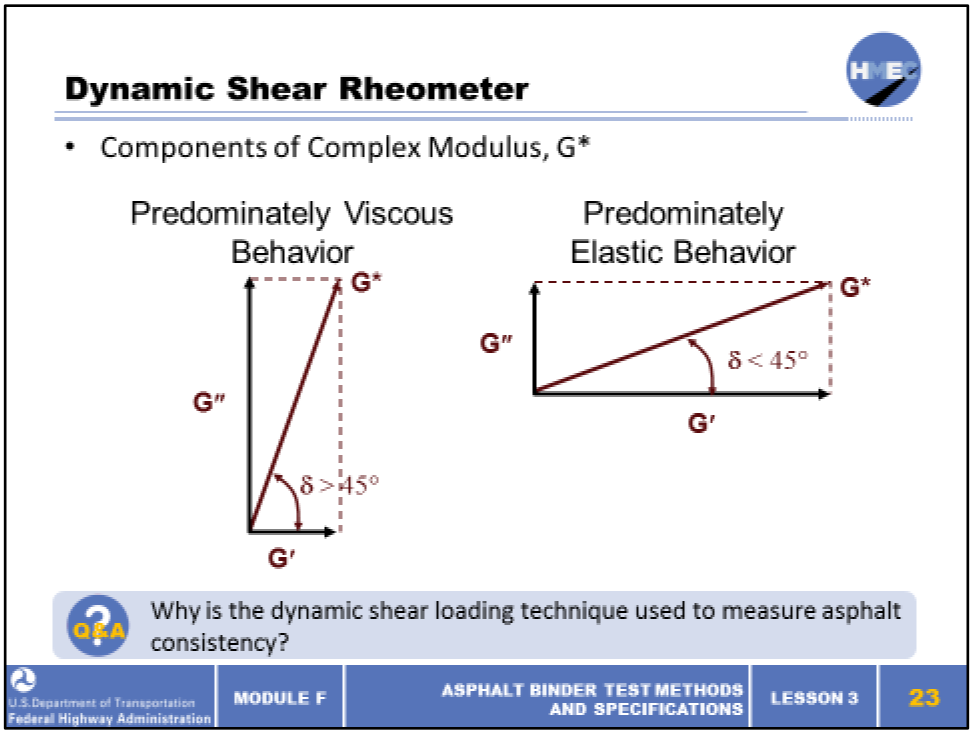
Slide 22



The figure shows a popular way of schematically showing the relationship between the viscoelastic moduli of asphalt binders (G*. G', and G").

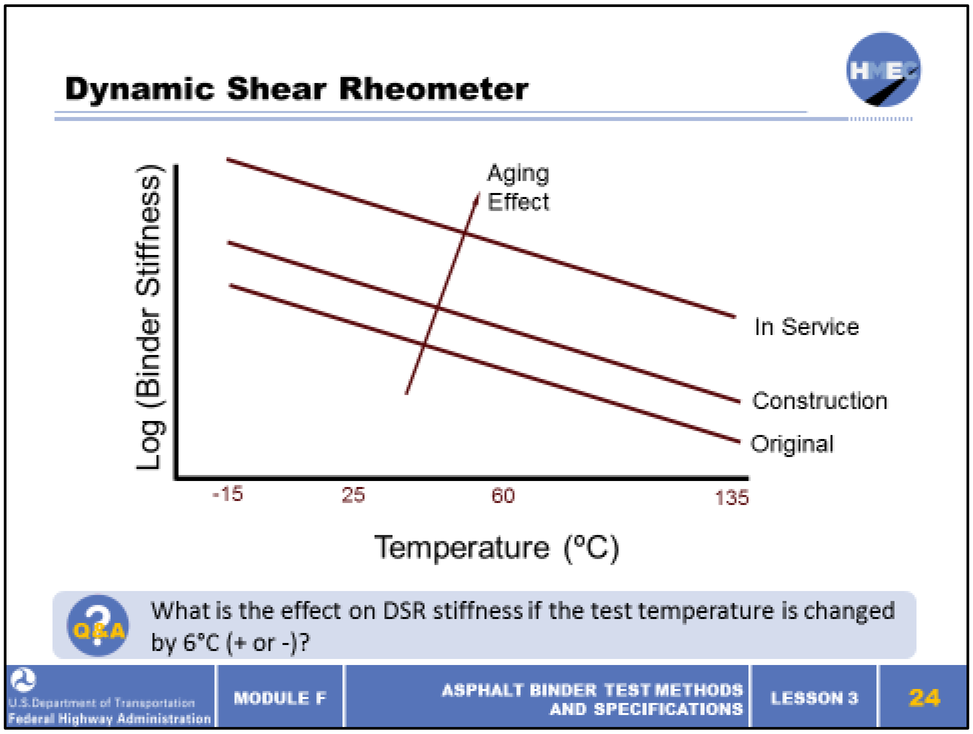
The complex modulus, G*, is the vector sum (Pythagorean's theorem). If delta is 0, the G* equals the storage modulus. In other words, the response is all elastic. If delta is 90°, then the response is all viscous (G* = viscous component).

Slide 23



The value of the phase angle determines the proportion of elastic behavior versus viscous behavior at a given temperature. Values greater than 45° indicate behavior that is predominately viscous (left diagram). Values less than 45° indicate behavior that is predominately elastic (right diagram).

Slide 24



Dynamic Shear Rheometer
HMEC
Log (Binder Stiffness)
Aging
Effect
In Service
Construction
Original
-15
25
60
135
Temperature (°C)
Q&A
What is the effect on DSR stiffness if the test temperature is changed by 6°C (+ or -)?
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ASPHALT BINDER TEST METHODS AND SPECIFICATIONS
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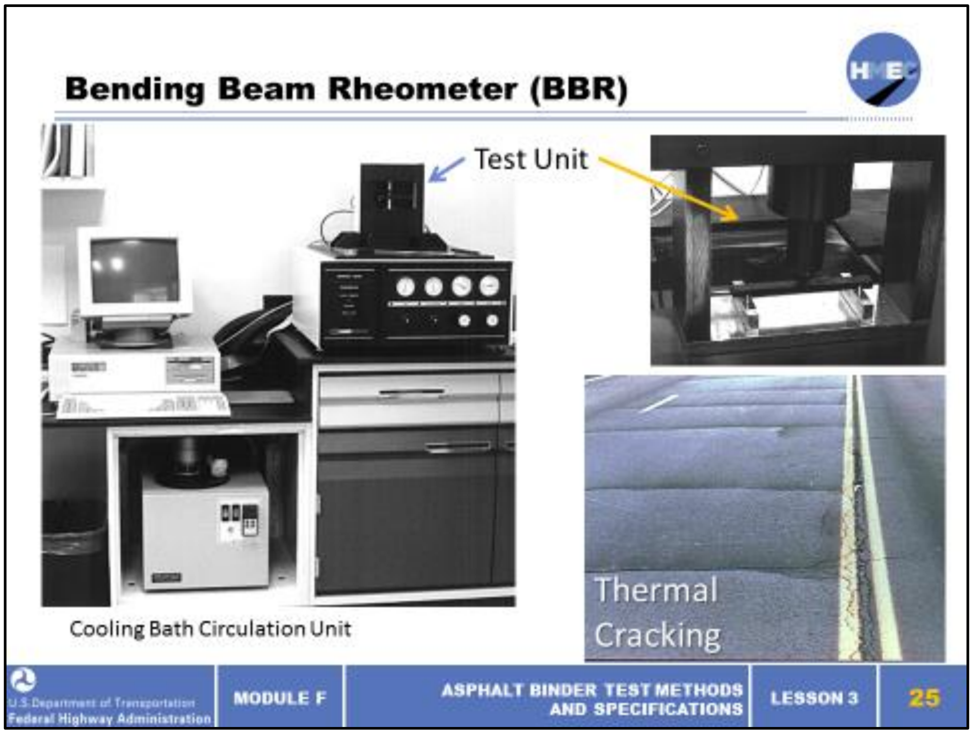
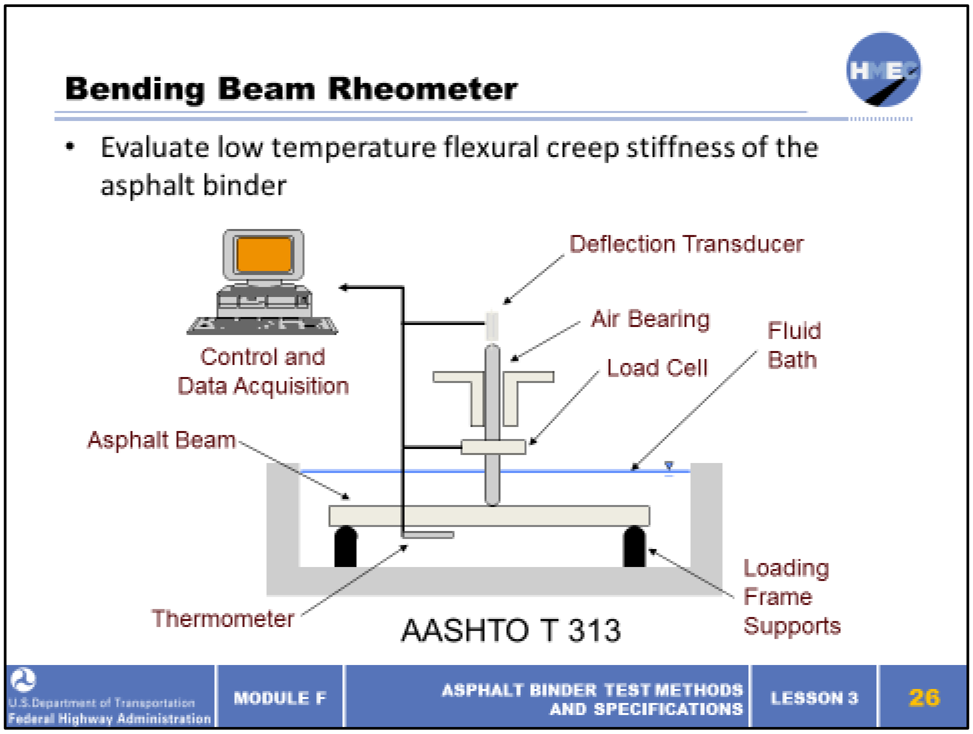


The photo shows the components of the BBR. The test unit is in the upper-right portion of the photo. The cooling bath circulation unit is beneath the computer. As noted previously, the BBR is used to determine and evaluate the low temperature resistance of asphalt binders.

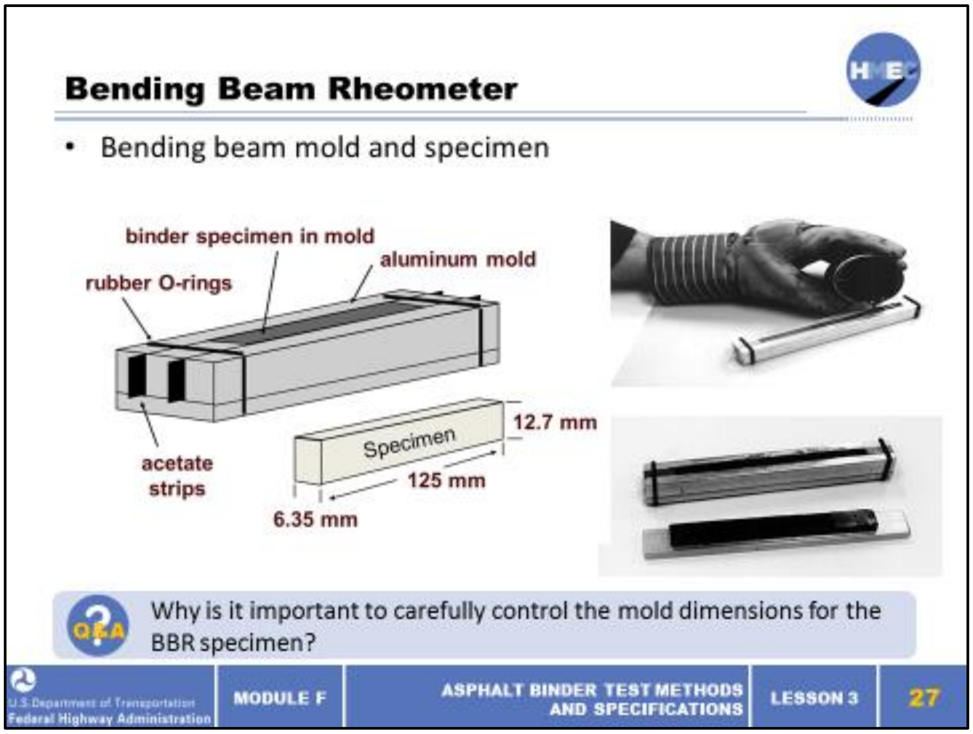
Slide 26



The asphalt binder beam is submerged in a cooling bath to facilitate accurate temperature control. Note also that the asphalt binder beam is simply supported and loaded at mid-span.

Loading is controlled and data is collected from the deflection transducer, load cell and thermometer utilizing computer software in combination with a control unit.

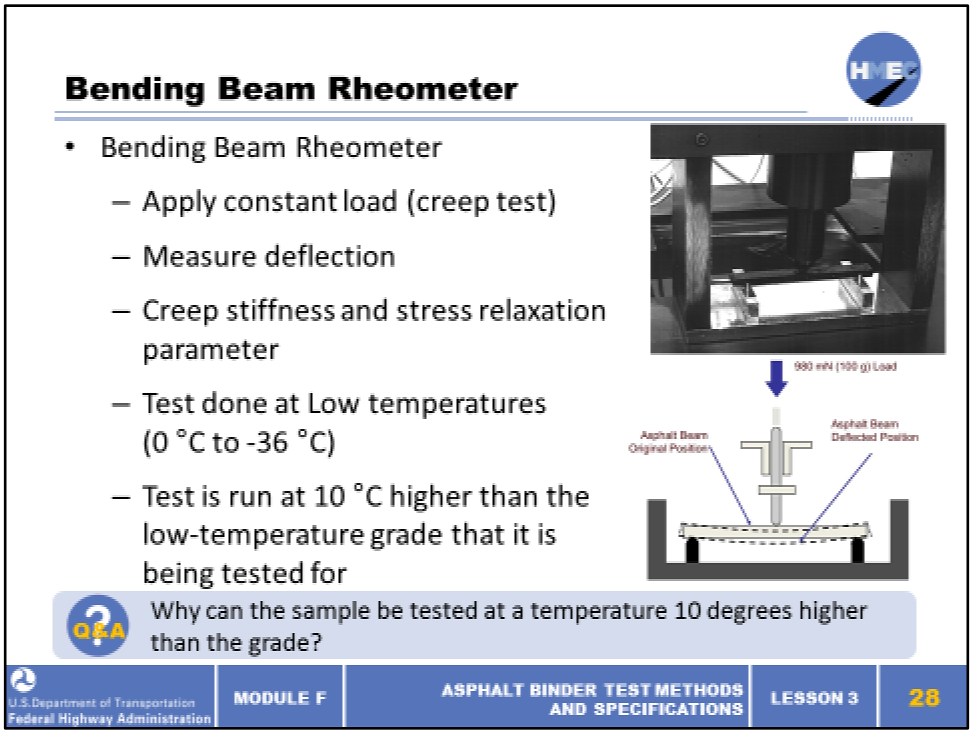
Slide 27



Note that the dimensions of the beam are 6.35 mm (0.25 in) wide by 12.7 mm (0.5 in) high by 125 mm (4.92 in) long. The acetate strips prevent the asphalt binder from adhering to the aluminum frame and are easily peeled off the beam without damaging the sample when the mold is disassembled.

It is important to produce consistent specimens which have dimensions as close to those specified in the AASHTO T313 as possible. Doing this ensures less variability from specimen to specimen and also better reproducibility between different labs.

Slide 28



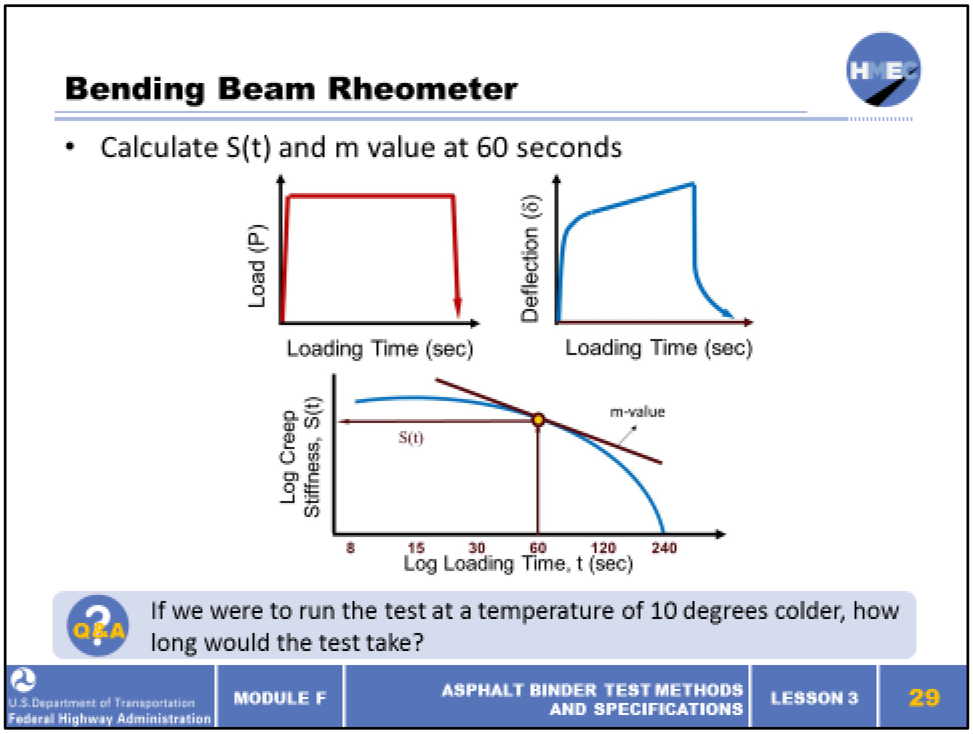
After this conditioning period, the specimen is placed on the test supports and a 30 mN preload is applied to the test specimen.

Following this, a test load of 980 mN is applied for 1 second then removed for 20 seconds, after which the test load is reapplied for 240 seconds. Test data is collected only during this latter 240-second period.

After this conditioning period, the specimen is placed on the test supports and a 30 mN preload is applied to the test specimen.

Following this, a test load of 980 mN is applied for 1 second then removed for 20 seconds, after which the test load is reapplied for 240 seconds. Test data is collected only during this latter 240-second period.

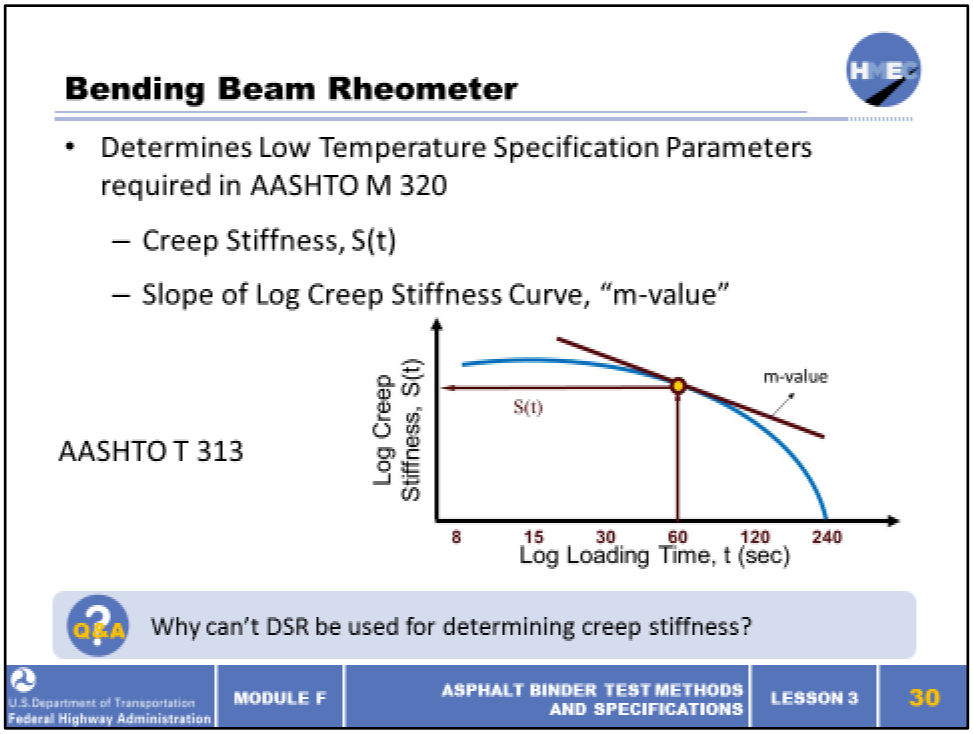
Slide 29



Upon initial loading, the rate of deflection increase is substantial but much less during the majority of the loading duration. Upon unloading, the beam recovers, almost instantaneously, the majority of the deformation (elastic response) followed by a delayed recovery (viscous response).

Since deformation increases the longer the load is left on the sample (and deformation is in the denominator of the preceding equation), stiffness decreases with time.

Slide 30

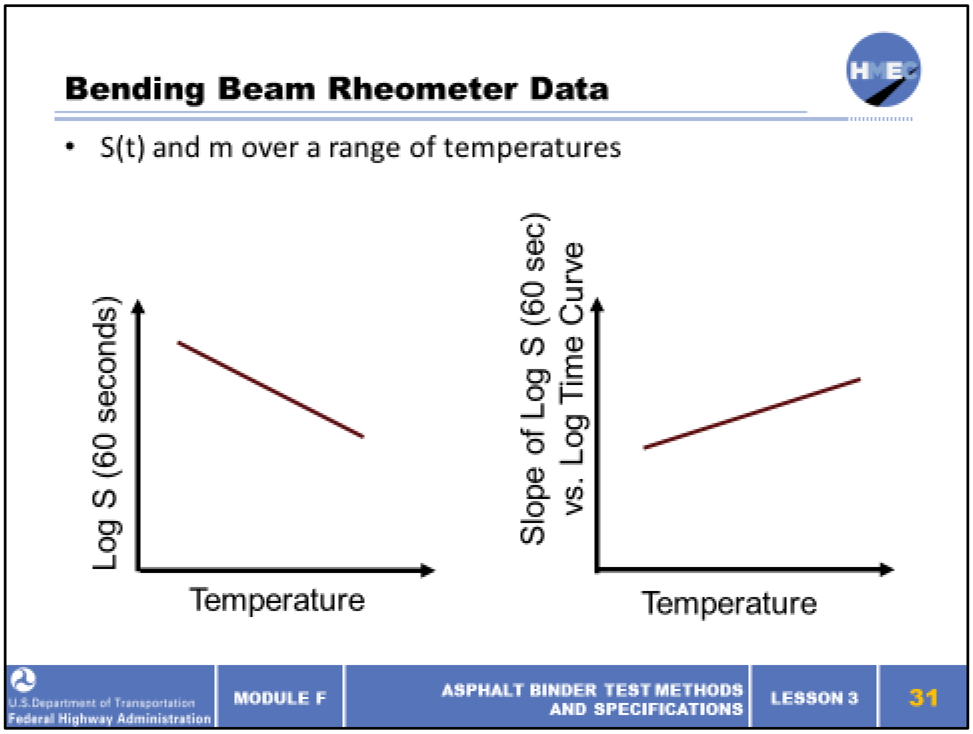


Reason for BBR test: This test (BBR) is used to satisfy the low temperature thermal cracking requirements in the performance graded asphalt binder specification.

There is a maximum requirement on the binder stiffness (S = max. 300 MPa) to ensure that the binder is sufficiently soft at cold temperatures.

In addition, there is a minimum set on the slope requirement (m = min. 0.3) to ensure that the material can relax (deform) quickly enough to prevent thermal cracking.

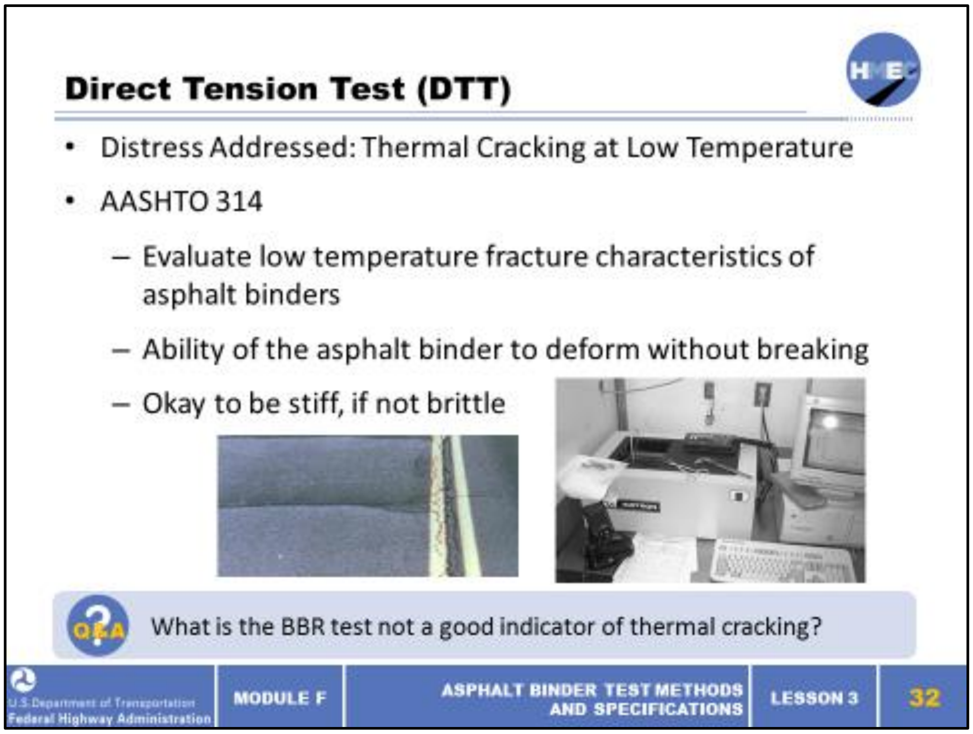
Slide 31



The chart on the left shows creep stiffness at 60 seconds versus temperature and indicates a higher stiffness at lower temperatures.

The chart on the right indicates that the m-value increases (the slope of the stiffness versus time curve gets steeper at 60 seconds) with increased temperature.

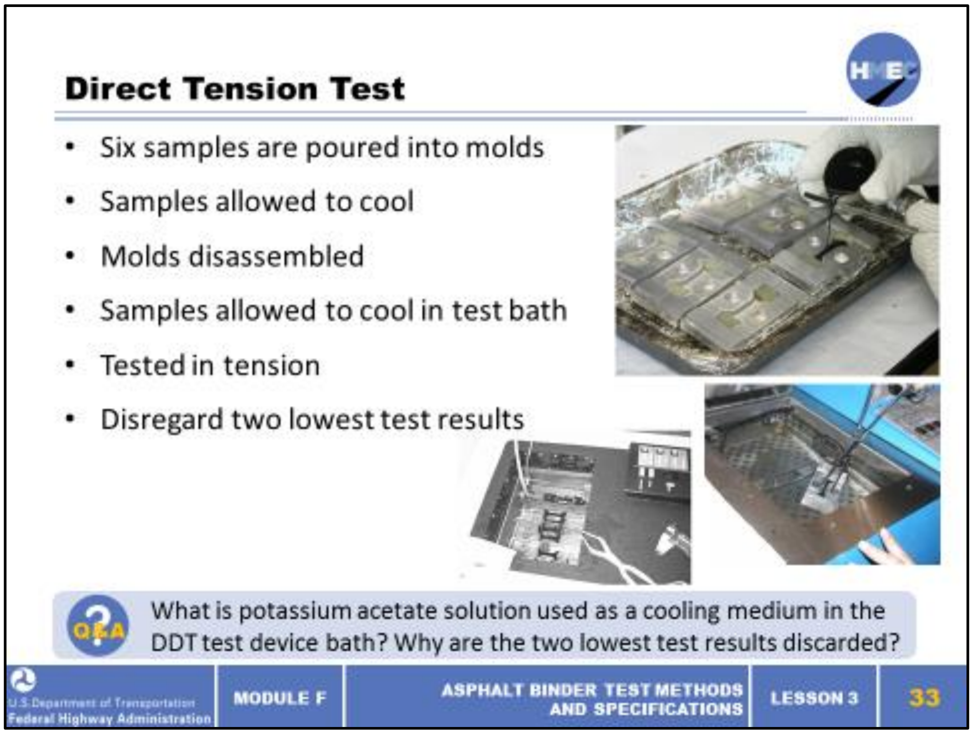
Slide 32



Increased stiffness of asphalt binders at lower temperatures is not considered a problem as long as the binder does not become brittle (i.e., easily fractured under load). Note that the unit is controlled by computer software and companion electronic hardware that also collects data during the test.

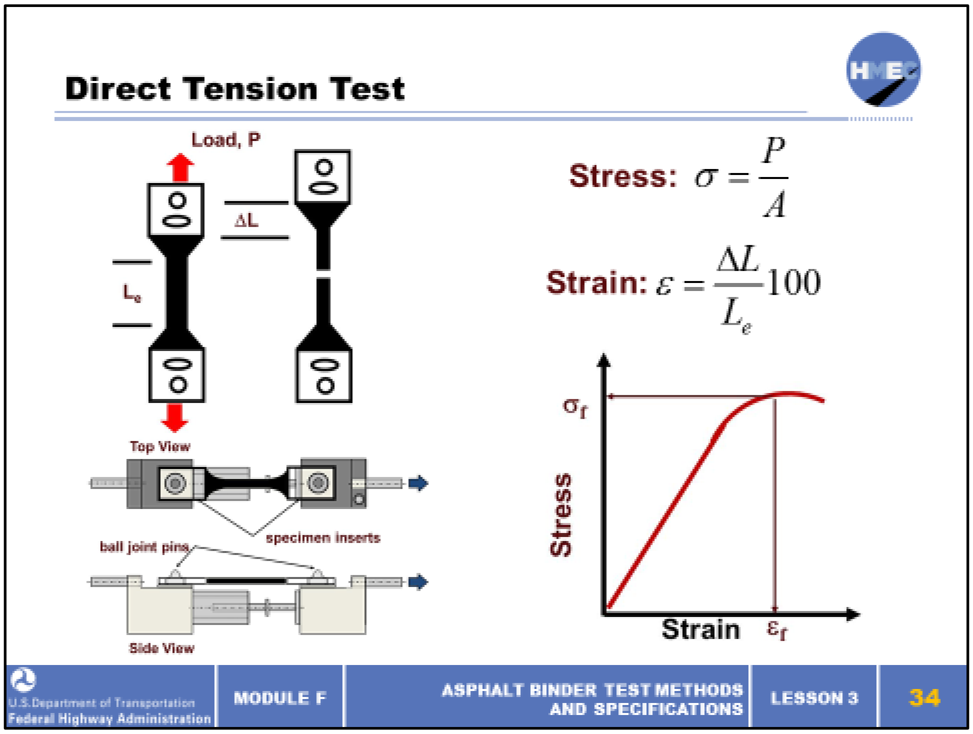
Reason for DTT test: This test (DTT) is used to satisfy the low temperature thermal cracking requirements in the performance graded asphalt binder specification. It can also be used in conjunction with the BBR to determine the critical cracking temperature.

Slide 33



Six test samples are always made and tested when performing the Direct Tension Test. After the samples are poured into the molds, cooled, and disassembled, they are placed in the test bath to condition for 1 hour. The bath uses potassium acetate solution as a cooling medium while testing. After the condition period, each sample is tested. The two lowest test values are disregarded when calculating the average values.

Slide 34

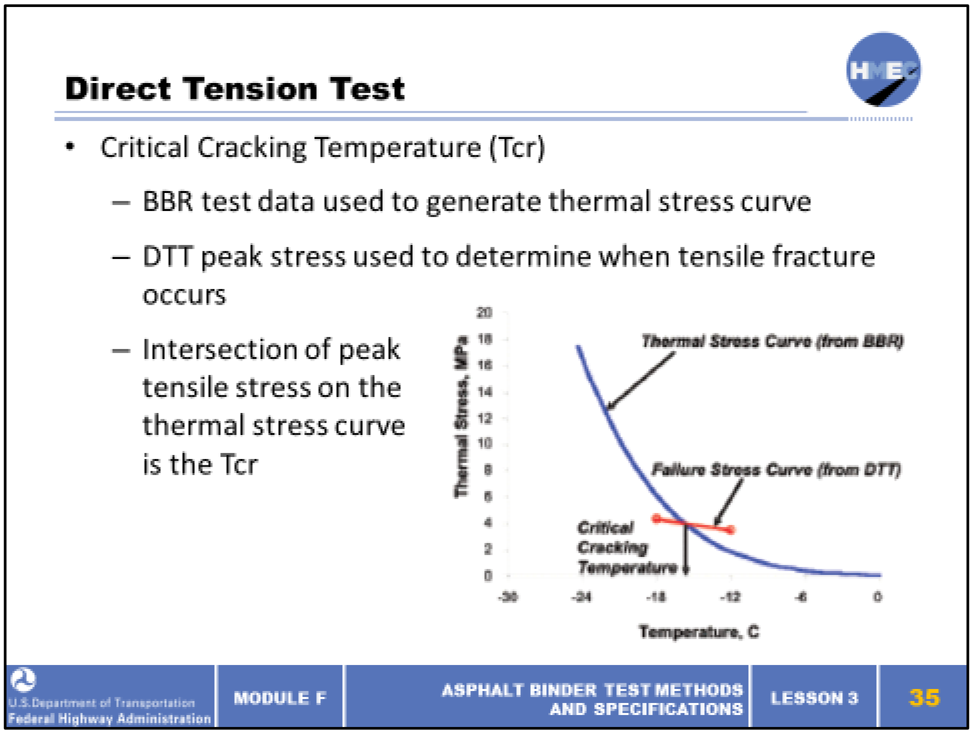


The stress at failure is determined from the maximum load divided by the original cross sectional area, whereas the strain at failure is determined from the change in length of the effective gauge length divided by the gauge length.

The PG binder specification requires a minimum strain at failure of 1.0% at the test temperature for a given binder grade. However, the direct tension test is not required if the creep stiffness (from the bending beam rheometer) is less than 300 MPa. If the creep stiffness is between 300 and 600 MPa, the DTT failure strain requirement can be used in lieu of the creep stiffness requirement. In either case, the m-value requirement (from the bending beam rheometer test) must be satisfied.

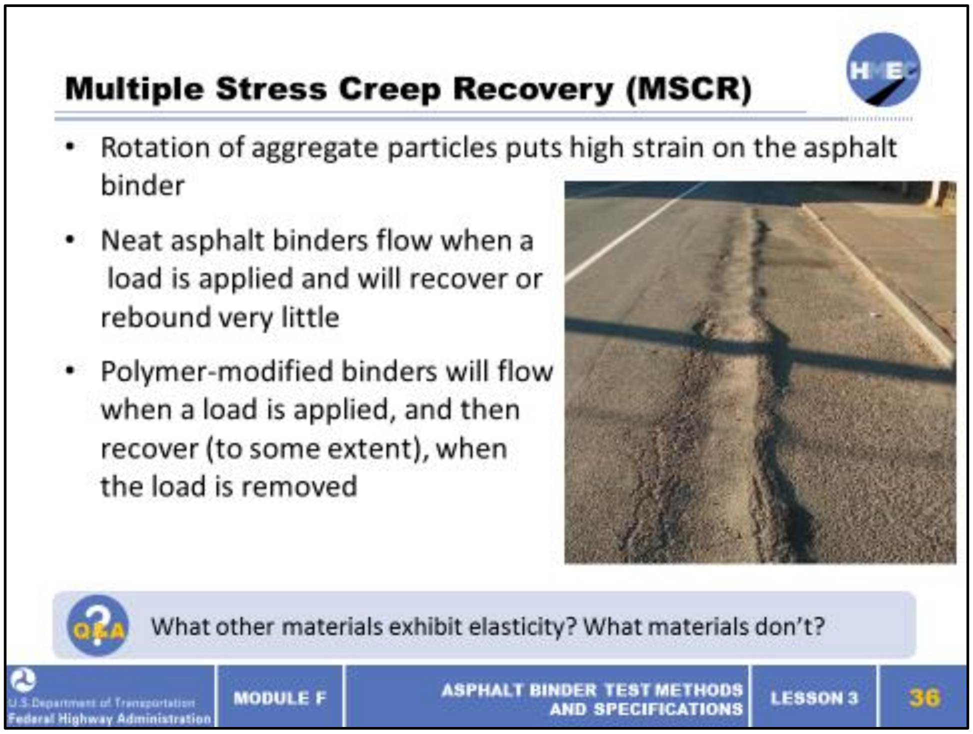
The photo on the lower left shows schematically the top and side views of an asphalt binder specimen arranged in the testing device. For the schematic, the left portion of the arrangement is fixed, while the right portion is able to move (i.e., apply a constant rate of elongation to the test specimen).

Slide 35



The CCT is an alternative method for determining low temperature use of a binder. It is used in conjunction with results from the BBR. The BBR data is used to generate a thermal stress curve, and then the DTT peak stress is used to determine when tensile fracture occurs.

Slide 36



As traffic volume and heavy loads on our roadways are on the rise, there has become a need for stronger and more durable pavements. As a wheel passes over the pavement surface, aggregate particles can rotate, putting high strain on the asphalt binder. If the binder has a high elasticity, it may recover from the deformation. However, if the strain exceeds the binders elastic capacity, permanent deformation (such as rutting) can occur. Modifiers are commonly added to the binder to enhance the material's recovery under strain.

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Slide 37

The elastic recovery test is time-consuming (can take 4 or 5 hours to complete). Many highway agencies are using modified test methods, not the AASHTO or ASTM versions, and producers who sell their product to more than one state often have to run several different procedures on the same material. The current asphalt binder specifications (like M 320) are based on the study of unmodified asphalts.

Tests like Elastic Recovery indicate the presence of modifiers, but have a poor correlation to performance. To address these problems, research was conducted to develop a test method that would characterize modified binder, and more importantly, is correlated to performance. This test would replace current PG+ standards like Elastic Recovery. Current PG binder tests consider asphalt to have a linear stress-strain relationship.

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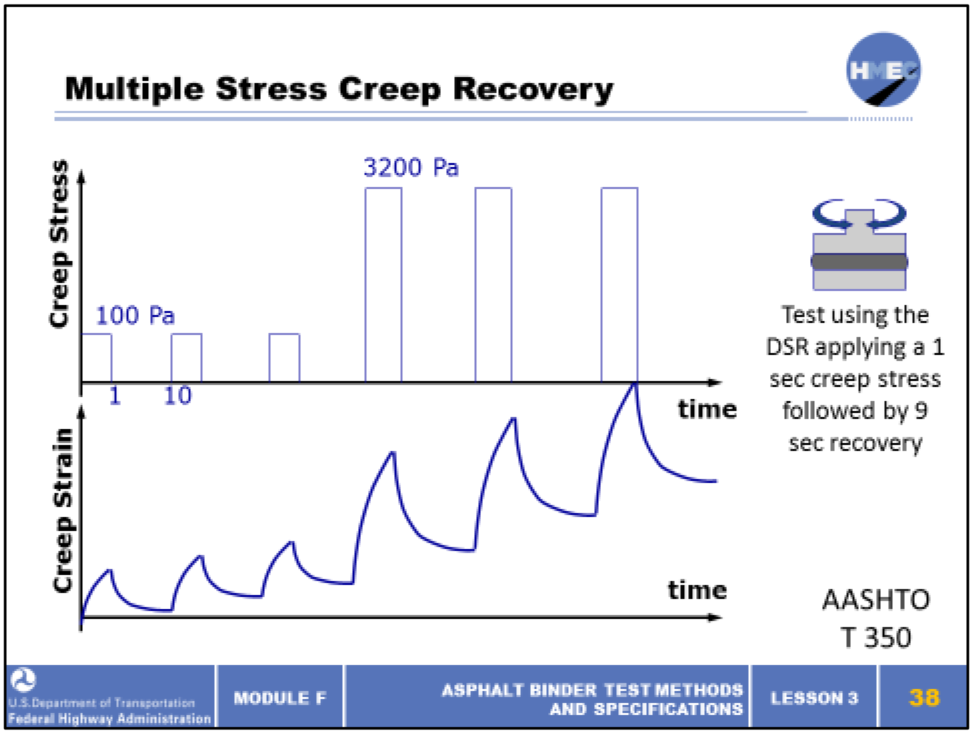
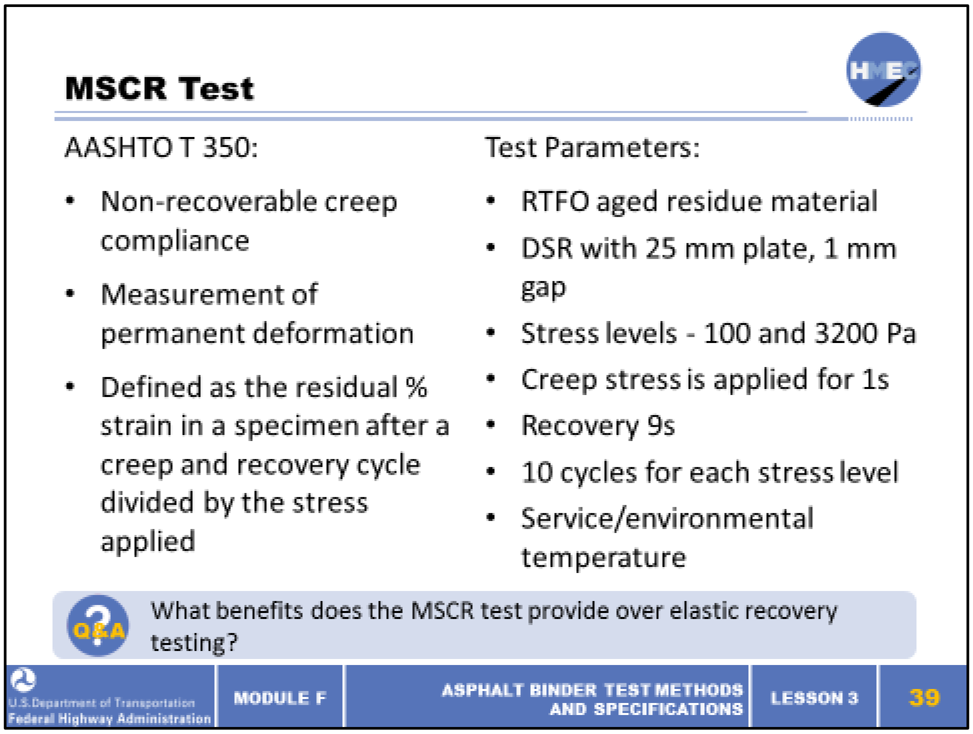
Slide 38



When rutting occurs (high stress and strain levels), asphalt does not necessarily behave as a linear material. Modified binders are much less likely to behave linearly. Testing at different stress levels can help us describe asphalt behavior in the non-linear range. The MSCR test as it is utilized today uses the DSR equipment that many laboratories already own. A load is applied to the asphalt, then the load is removed, and the response of the material is measured.

Slide 39

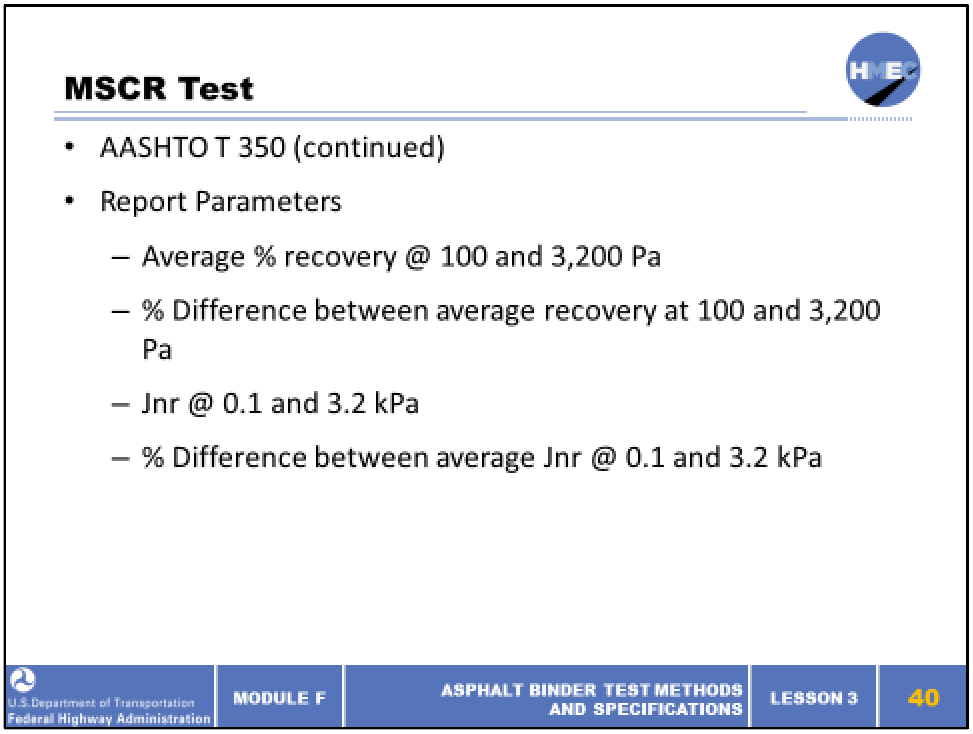


Creep is the tendency of a material to deform when a constant load is applied. Creep compliance is a measure of the deformation that occurs when a load is applied. Theoretically, creep compliance it is the rate at which strain increases when a constant stress is applied. J, creep compliance = strain/stress.

By releasing the load and allowing the material to recover, we can determine the recoverable creep compliance, Jr. We can then determine the non-recoverable creep compliance, Jnr as Jnr = J – Jr.

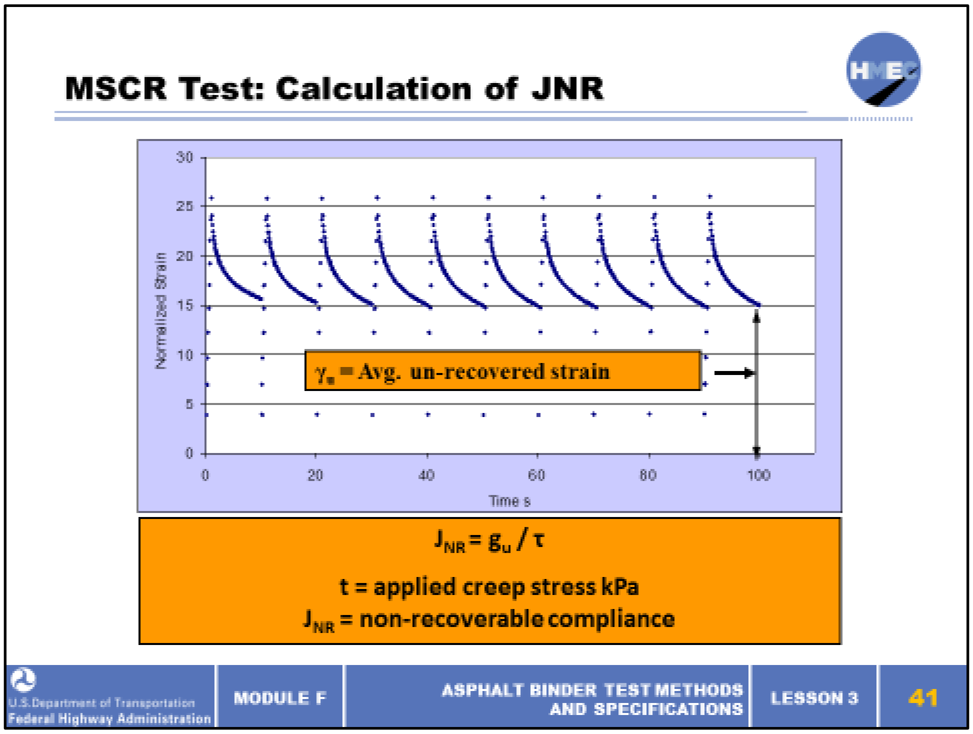
Cover each of the points on the slide in terms of what the test measures and the test parameters.

Slide 40



To truly evaluate binders, they need to be tested at the end of their linear range. Through testing, it has been determined that most neat binders reach the end of their linear range at 3.2 kPa. The material is tested at two stress levels: 0.1 kPa (in the linear region of behavior) and 3.2 kPa (at the end of the linear region of behavior). 10 cycles are run at each stress level to get a good test average. The goal is to evaluate the difference in performance of the material in the linear and non-linear region.

Slide 41



Information obtained from Dr. John D'Angelo's thesis summarizing the Jnr and % recovery and its use: The inadequacy of the Superpave high temperature specification parameter, G*/sin δ, to correctly grade the superior field performance of modified asphalt binders has been demonstrated by several researchers. A new parameter that is blind to modification type and is performance based is now needed. As a replacement for the existing high temperature binder parameter (G*/sin δ), this study has developed an easy to use multiple stress creep and recovery test (MSCR) that measures fundamental characteristics of asphalt binders.

In this study, several binder parameters proposed to replace the existing Superpave rutting parameter were validated using hot-mix testing. Several different binder tests were evaluated to determine which would provide a replacement for the Superpave high temperature binder criteria. The new test and criteria will have to be performance related and blind to modification. The results from these binder tests were compared against hot-mix rutting results from the asphalt pavement analyzer, the Hamburg wheel tracking test, and the FHWA ALF test sections and field roadway sites. The results from the hot-mix rut testing showed that different rut testers will provide completely different ranking of binders. This difference is related to the stress level applied by the different testers. This hot-mix testing indicates that the different binders, specifically the polymer modified binders, have different stress dependencies.

A separate sub-study was also conducted in this research to understand the effect of stress and strain on the microstructure of polymer modified binders. It was found that in MSCR data there is a clear relationship between %recovery in the recovery part and %strain in the creep portion of the test. In some cases, at least, this is the dominant relationship. Very high strain causes yield behavior in polymer modified asphalt binders (PMA). After high strain, PMAs still exhibit recovery but the rate of recovery is reduced. At high strain, binder morphology, tensile and shear properties change.

A test procedure was developed to run creep and recovery testing on one sample at multiple stress levels (MSCR). This test procedure makes it easy to evaluate how the binder response will change under different stress conditions. A property called nonrecoverable compliance Jnr was developed based on the non-recovered strain at the end of the recovery portion of the test divided by the initial stress applied during the creep portion of the test. The Jnr value normalizes the strain response of the binder to stress which clearly shows the differences between different polymer-modified binders.

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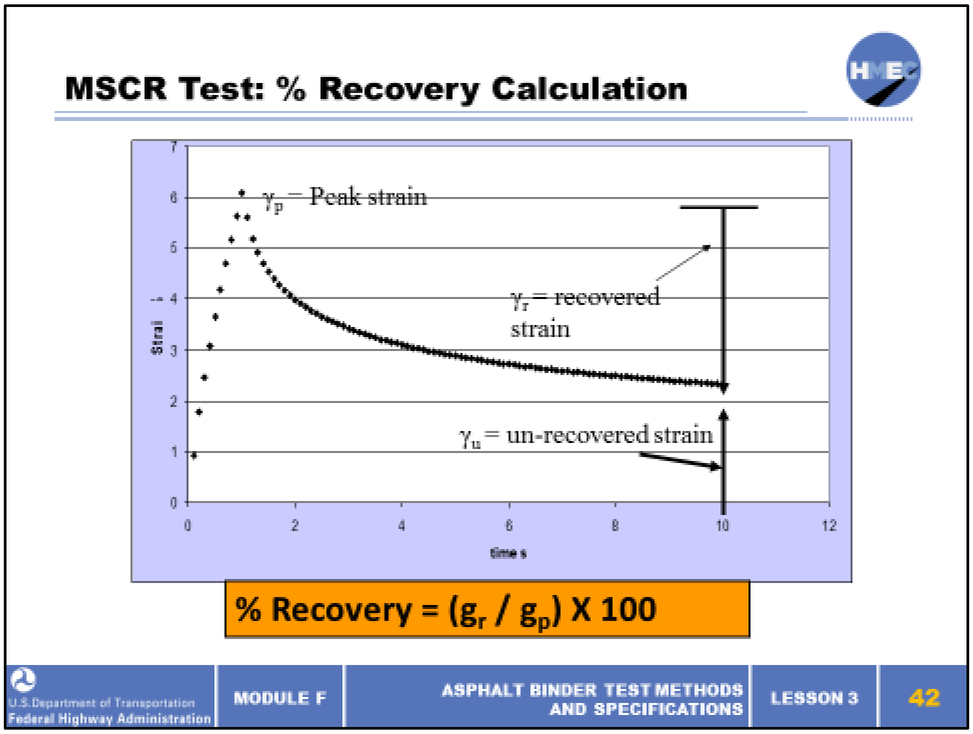
MSCR Test: % Recovery Calculation
HMEC
γ_p = Peak strain
γ_r = recovered strain
γ_u = un-recovered strain
Strai
time s
0
1
2
3
4
5
6
7
0
2
4
6
8
10
12
% Recovery = (g_r / g_p) X 100
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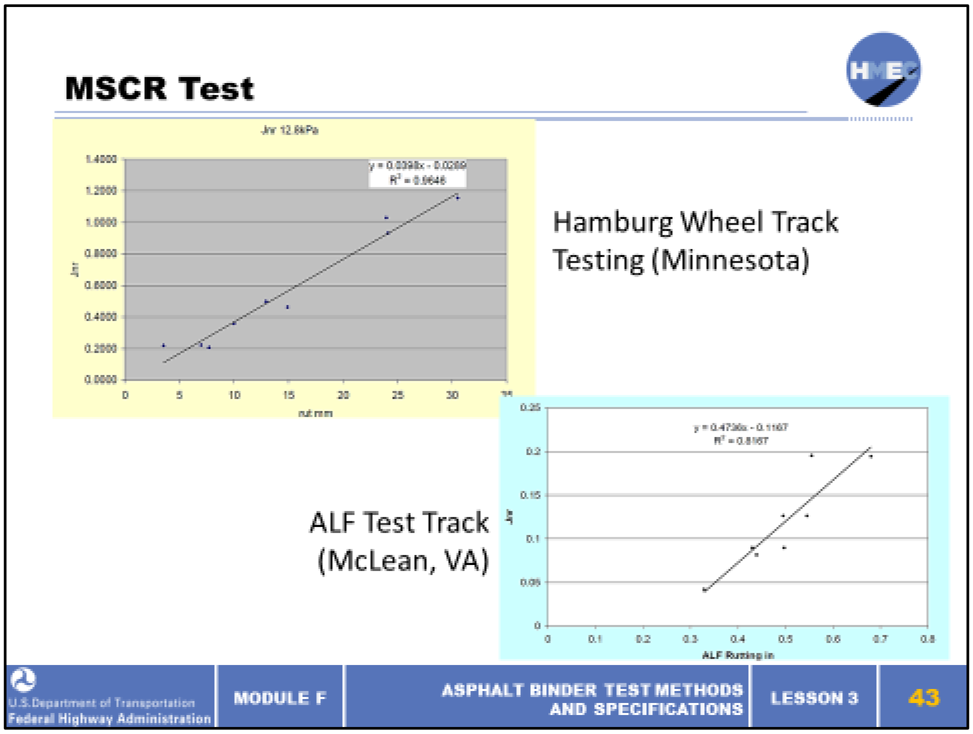
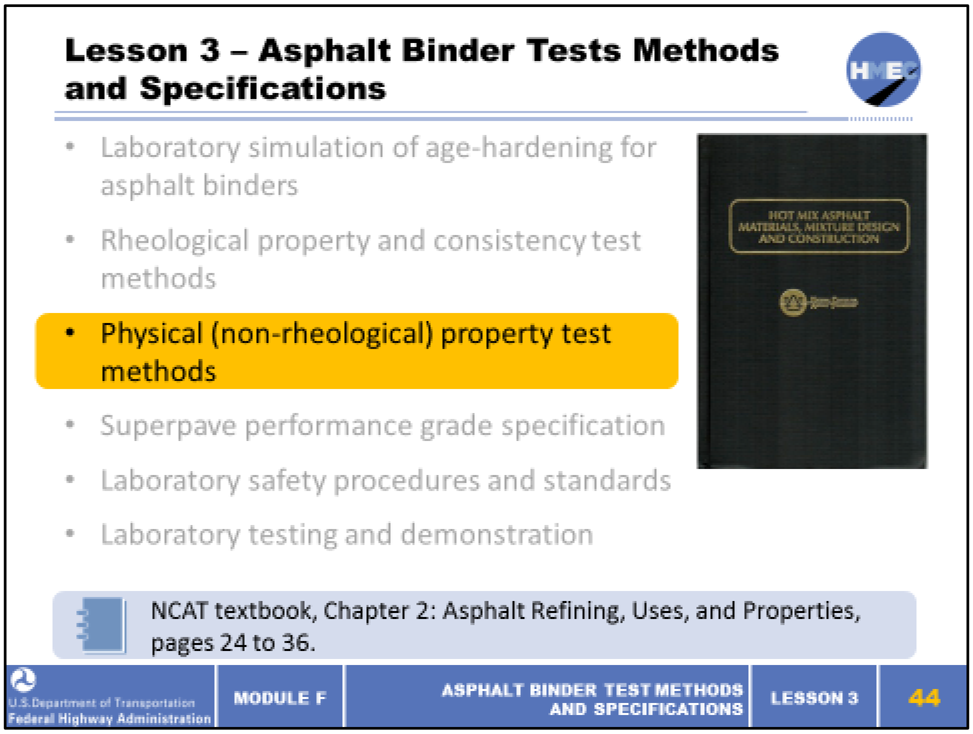


This data shows the correlation between Jnr and rutting on a Hamburg wheel track tester and on the ALF test track and FHWA Turner Fairbanks in McLean, VA.

Slide 44


Lesson 3 – Asphalt Binder Tests Methods and Specifications
HMEC
• Laboratory simulation of age-hardening for asphalt binders
• Rheological property and consistency test methods
• Physical (non-rheological) property test methods
• Superpave performance grade specification
• Laboratory safety procedures and standards
• Laboratory testing and demonstration
HOT MIX ASPHALT MATERIALS, MIXTURE DESIGN AND CONSTRUCTION
NCAT textbook, Chapter 2: Asphalt Refining, Uses, and Properties, pages 24 to 36.
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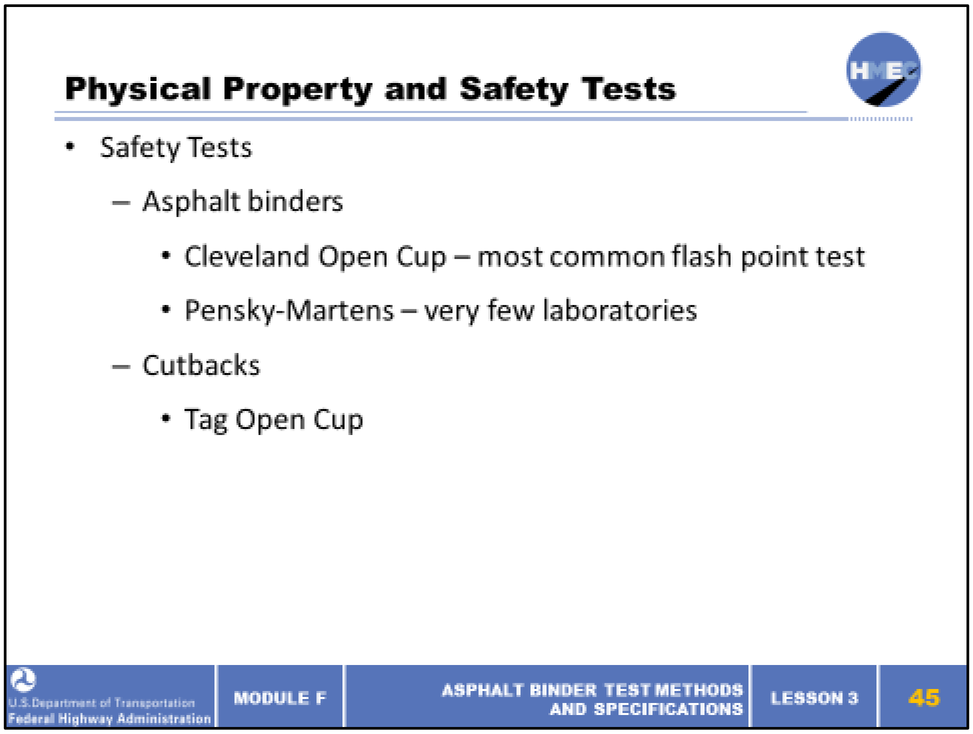
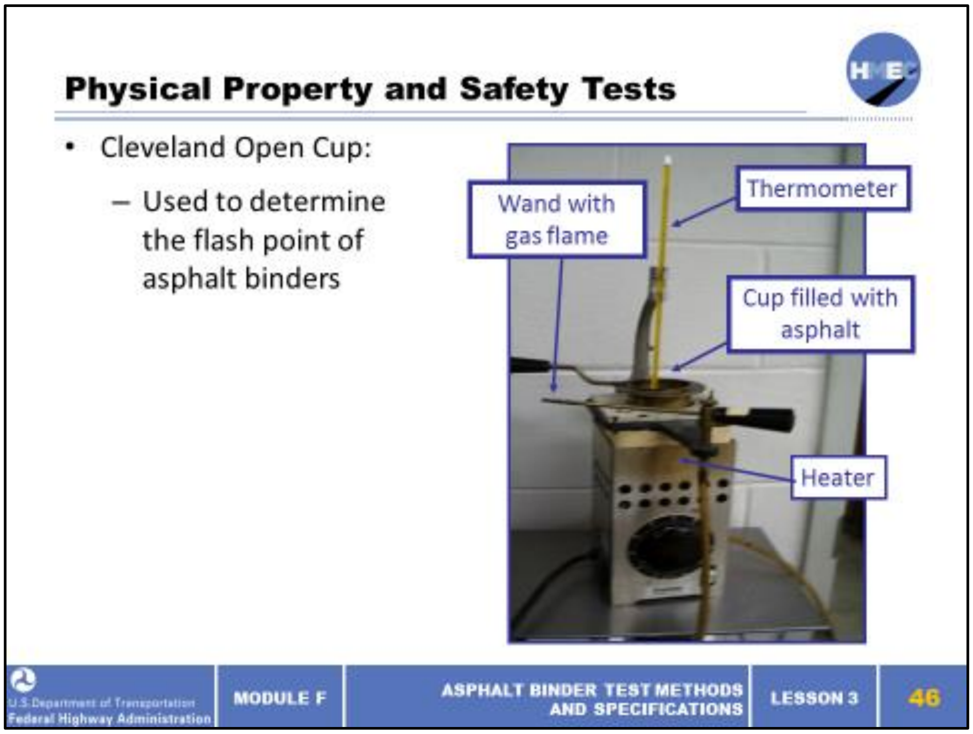


Flash and fire point tests are used to determine the temperature at which an asphalt binder will ignite and support a flame.

The Cleveland open cup and Pensky-Martens tests are normally used for asphalt binders, whereas the Tag open cup test is normally used for cutbacks. The Cleveland open cup test method is generally used to determine the flash point of asphalt binders. The Pensky-Martens is used by relatively few laboratories.

Flash point tests are not specified in emulsified asphalt binder specifications.

Slide 46



Significance and use:

• The flash point is one measure of the tendency of the test specimen to form a flammable mixture with air under controlled laboratory conditions. It is only one of a number of properties that should be considered in assessing the overall flammability hazard of a material.

• Flash point is used in shipping and safety regulations to define flammable and combustible materials. Consult the particular regulation involved for precise definitions of these classifications.

• Flash point can indicate the possible presence of highly volatile and flammable materials in a relatively nonvolatile or nonflammable material. For example, an abnormally low flash point on a test specimen of engine oil can indicate gasoline contamination.

• This test method shall be used to measure and describe the properties of materials, products, or assemblies in response to heat and a test flame under controlled laboratory conditions and shall not be used to describe or appraise the fire hazard or fire risk of materials, products, or assemblies under actual fire conditions. However, results of this test method may be used as elements of a fire risk assessment that takes into account all of the factors that are pertinent to an assessment of the fire hazard of a particular end use.

• The fire point is one measure of the tendency of the test specimen to support combustion.

• ASTM D92 - 05a Standard Test Method for Flash and Fire Points by Cleveland Open Cup Tester

• Test method may be found in ASTM D 92 - 05a Standard Test Method for Flash and Fire Points by Cleveland Open Cup Tester or AASHTO T 48-06

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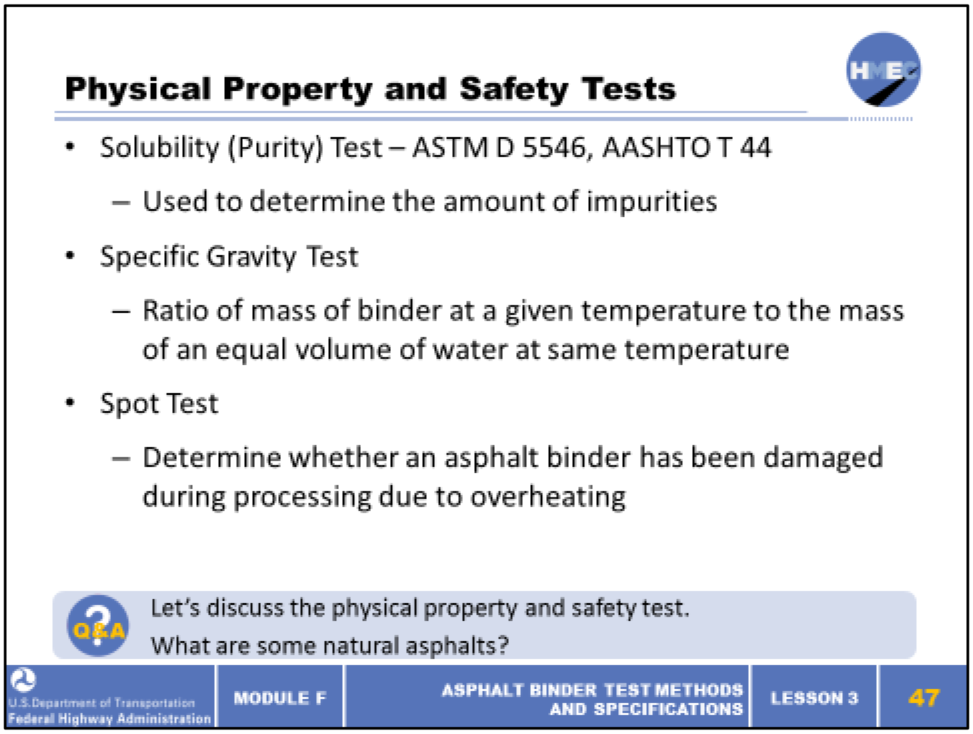

The solubility test is used to determine the amount of impurities in the asphalt binder. In general, most refined asphalt binders have little to no impurities. Higher impurities can exist in some of the natural asphalts that were discussed under Lesson 2. Most specifications for asphalt binders require a minimum of 99.0% solubility in trichloroethylene. ASTM D 5546 uses toluene, while AASHTO T 44 uses trichloroethylene.

The specific gravity of the asphalt binder is the ratio of the asphalt binder mass at a given temperature to the mass of an equal volume of water at the same temperature. The specific gravity is an important physical property because it is used in the volumetric analysis of asphalt mixtures. The specific gravity is dependent on temperature.

The spot test is used to determine whether an asphalt binder was damaged during processing due to overheating that will result in cracking. If an asphalt binder is overheated during processing, that binder is more susceptible to weathering and aging. However, molecular cracking is considered very usual in asphalt binders produced by refining practices, so it is usually not a requirement in most specifications for petroleum-based asphalt binders. Damaged asphalt binders generally have poor ductility and high oxidation or hardening rates.

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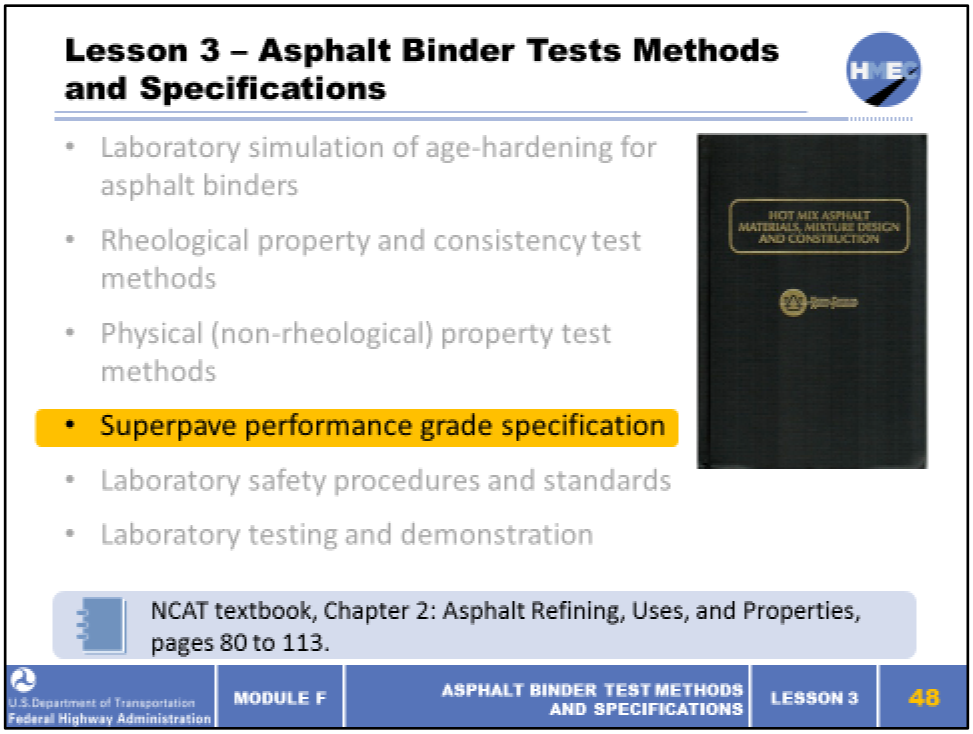
Slide 48


Lesson 3 – Asphalt Binder Tests Methods and Specifications
HMEC
• Laboratory simulation of age-hardening for asphalt binders
• Rheological property and consistency test methods
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• Superpave performance grade specification
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HOT MIX ASPHALT MATERIALS, MIXTURE DESIGN AND CONSTRUCTION
NCAT textbook, Chapter 2: Asphalt Refining, Uses, and Properties, pages 80 to 113.
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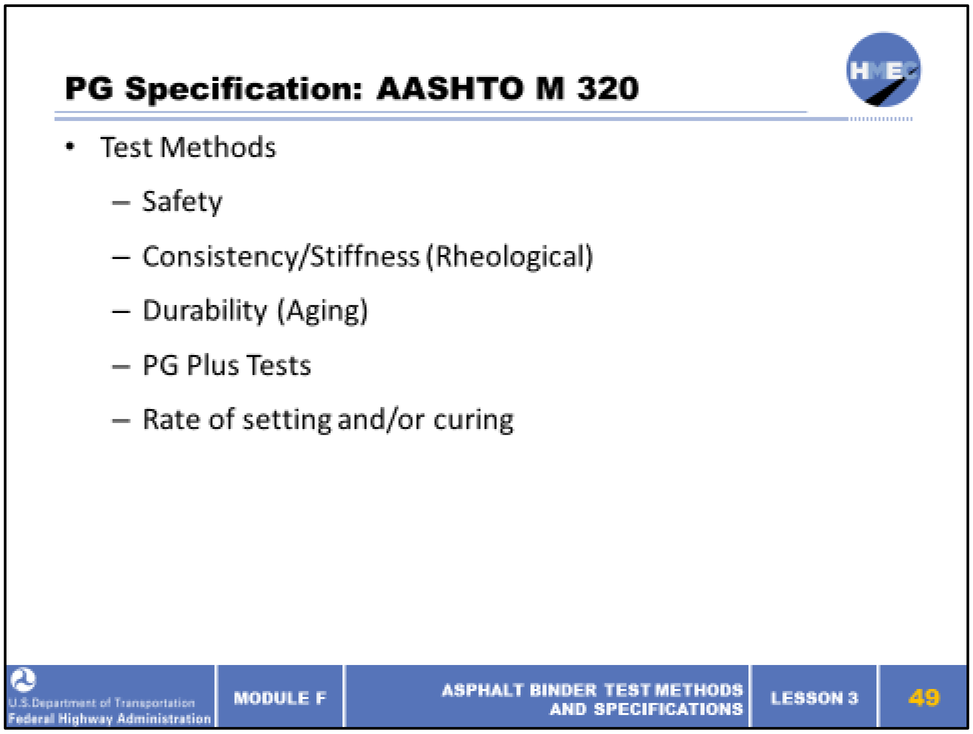

Historically, specifying agencies and researchers have used a large number of tests to define the physical and chemical properties of asphalt binders. The discussion presented is directed only toward those tests that are utilized in present specifications.

Specifications for asphalt binders contain test methods that describe:

- Information for worker's safety;
- Consistency of the binder;
- Durability or aging;
- PG plus tests; and
- Rate of setting and/or curing.

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Overview of PG System/Specification

Superpave Binder Test	Purpose of Test	
RTFO & PAV	Age binder for other tests & mass loss.	Short & Long Term Properties
Rotational Viscometer (RV)	Measure consistency	Pumpability at high temperature.
Dynamic Shear Rheometer (DSR)	Measure stiffness for rutting & fatigue	Evaluate at high & intermediate temp.
Bending Beam Rheometer (BBR)	Measure stiffness for thermal cracking	Evaluate at low temperature
Direct Tension Test (DDT)	Measure strength for thermal cracking	Evaluate at low temperature

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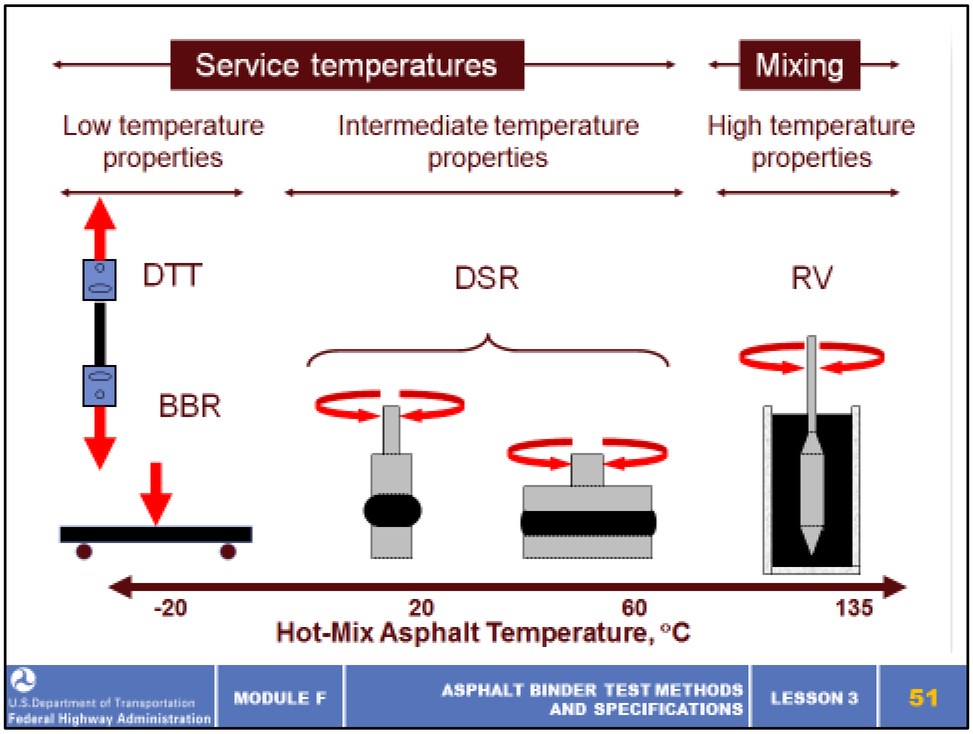

The diagram shown indicates how the tests are used to relate binder properties at various temperatures to HMA temperatures, both during the construction process and while in service.

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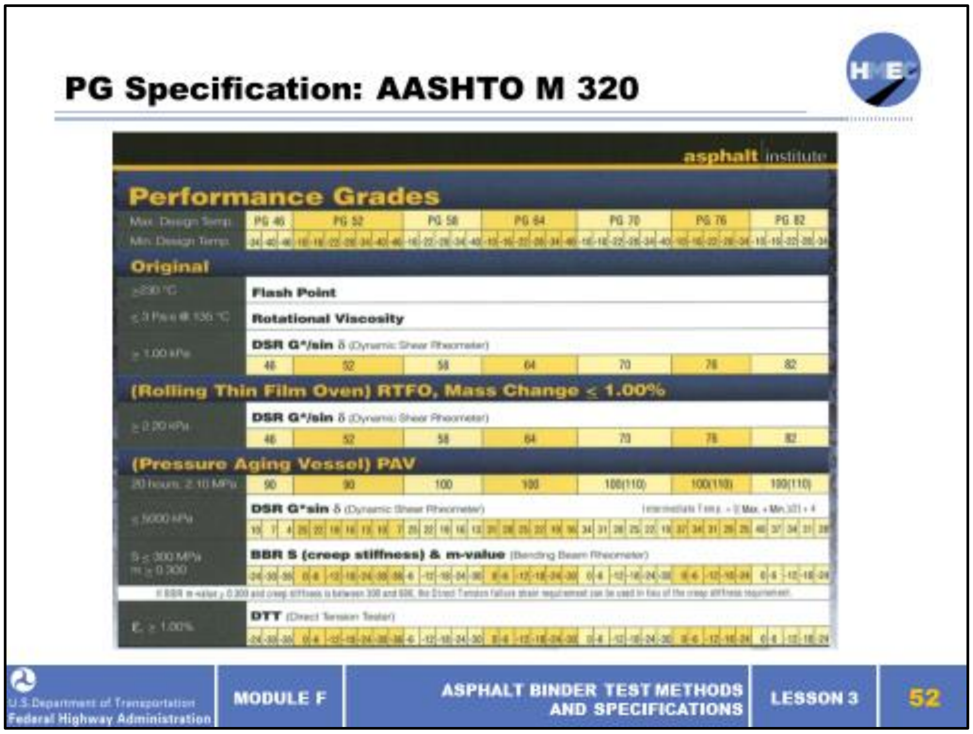

The mean 7-day high temperature is used to identify the appropriate major column heading from the first row. The single low pavement is used to select the specific column heading from the second row.

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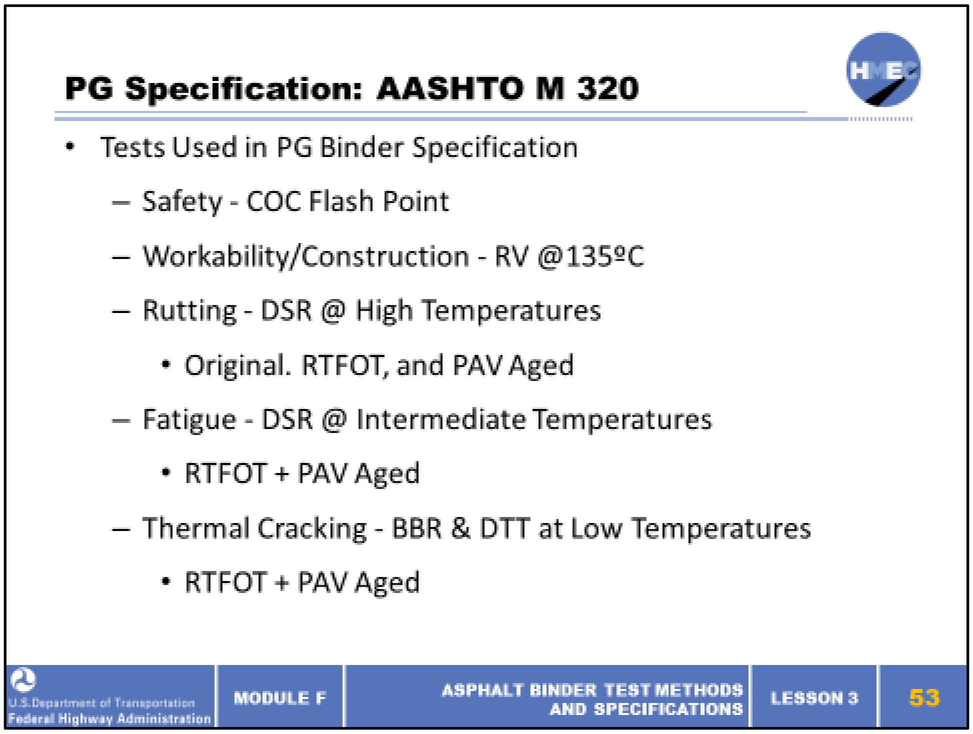

• For safety, the Cleveland open cup (COC) device is used to measure the flash point of the asphalt binder.
• The rotational viscometer (RV) is utilized for assessing aspects of the binder associated with workability and construction.
• The dynamic shear rheometer is used to assess a binder's contribution to rutting and fatigue cracking resistance.
• The bending beam rheometer (BBR) and direct tension tester (DTT) are used to assess the binder's contribution to thermal cracking resistance.

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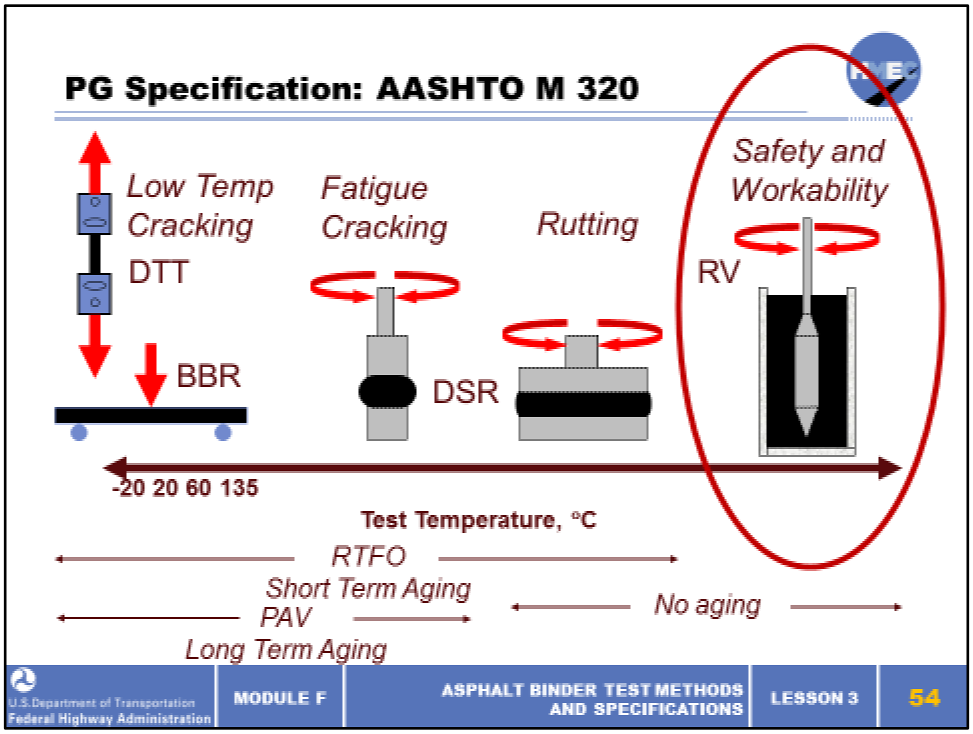

Again, the Cleveland open cup (COC) device is used to assess safety, whereas the rotational viscometer (RV) is used to assess workability (ability to pump the binder and construct an HMA pavement with the binder).

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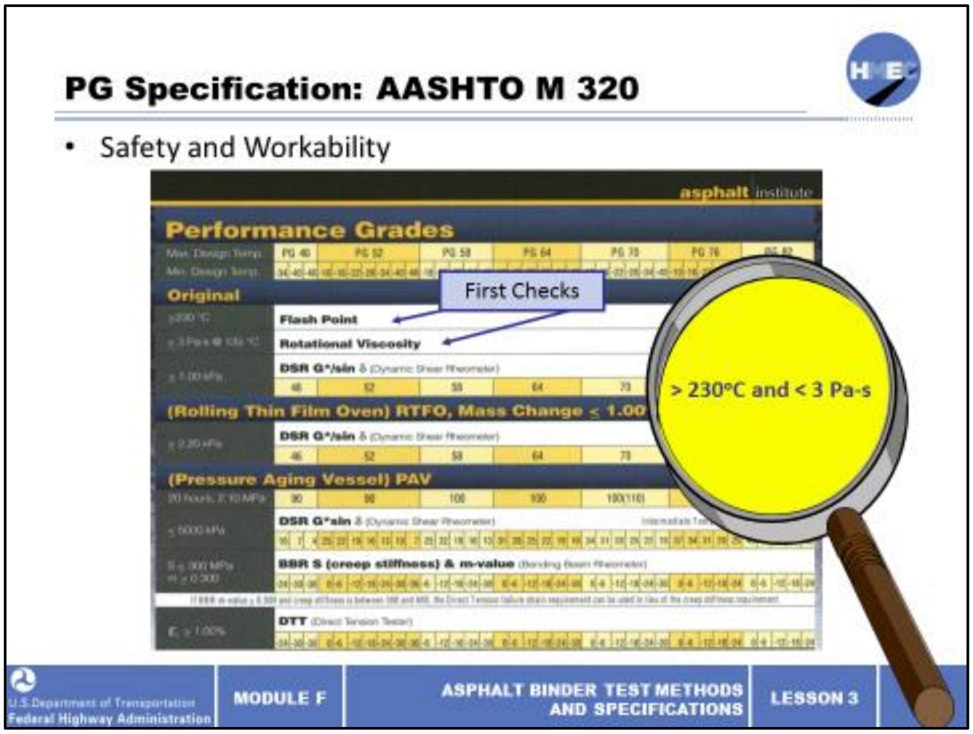
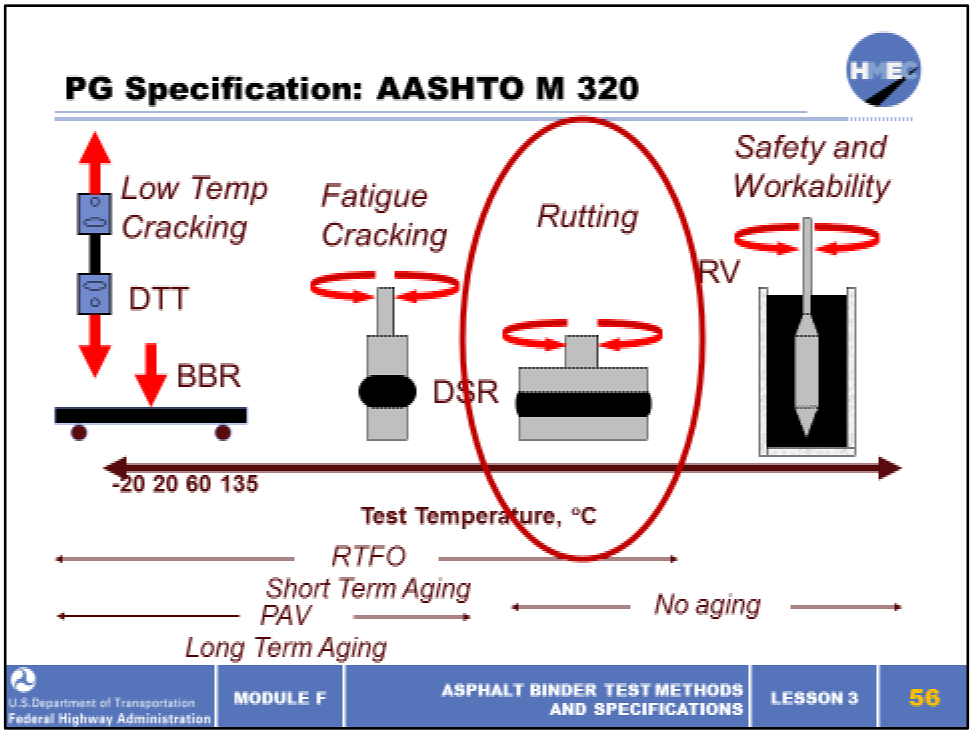


However, a contractor must be able to pump the binder to get it to the batch plant or drum mixer so that it can be mixed with aggregate. All grades in the PG system must have a flash point greater than 230 °C (446 °F) and a viscosity less than or equal to 3 Pa-s when the test is conducted at 135 °C (275 °F).

Slide 56



Tests are conducted at relatively high temperatures, so the 25 mm diameter spindle is used during these tests.

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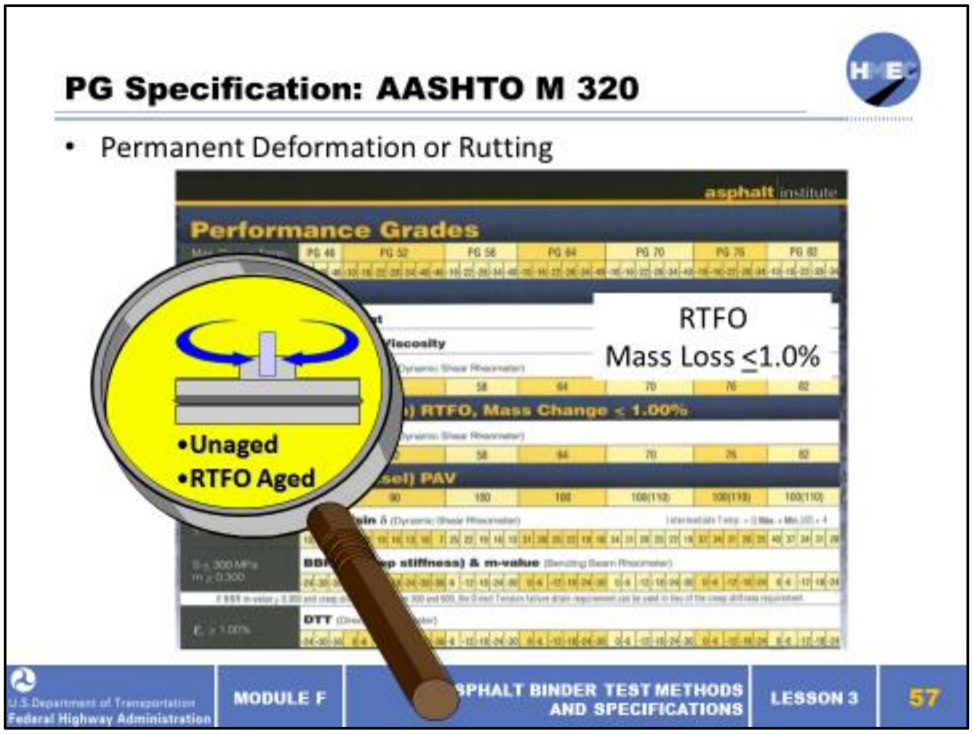

The unaged condition is checked in case the binder is not sufficiently aged in the plant. The RTFO-aged residue is checked to ensure that the material has sufficient stiffness after mixing it with the aggregate in the plant.

Mass loss is determined prior to testing the aged binder in the DSR, which is performed at the same temperature used for the original binder DSR testing. The equation for determining the percent change in mass during RTFO aging was included in an earlier portion of this lesson.

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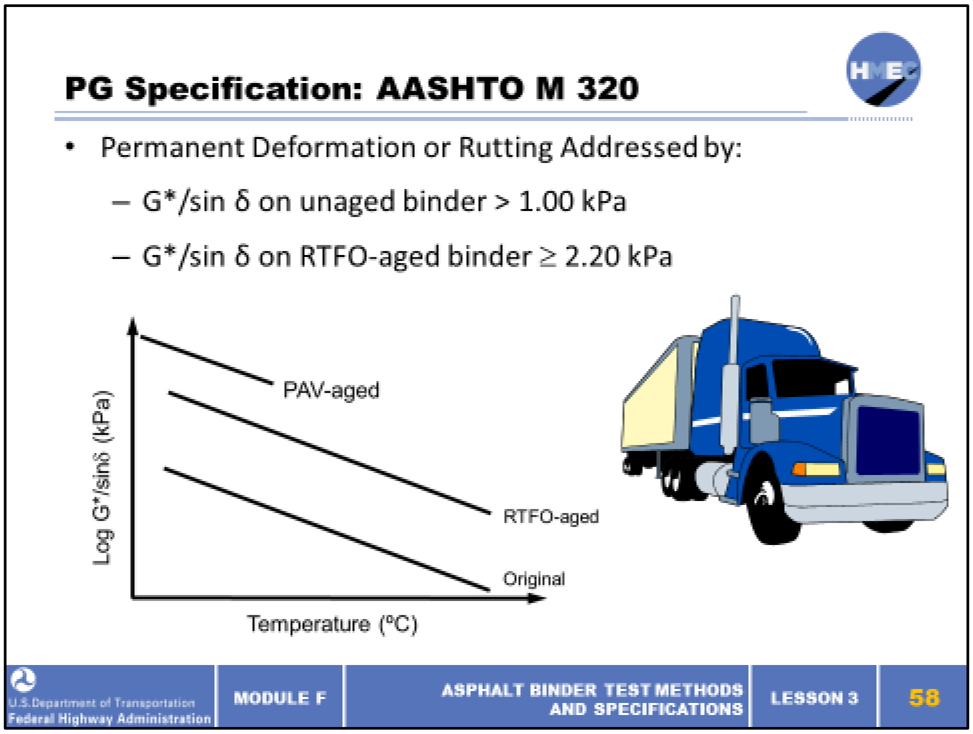
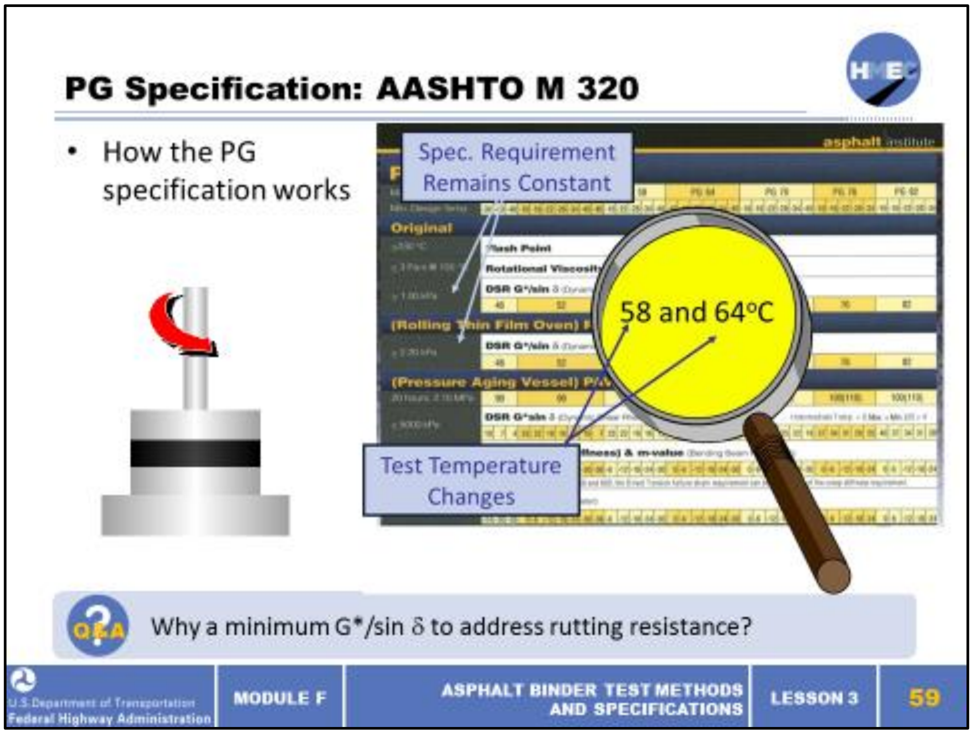


At high temperatures, the potential for permanent deformation (rutting) is the major pavement distress being evaluated. Results from DSR tests conducted on the PAV-aged binder are not used for assessing the binder's contribution to rutting resistance. This is because rutting generally occurs in the early life of an HMA pavement, which is represented better by the RTFO-aged binder state.

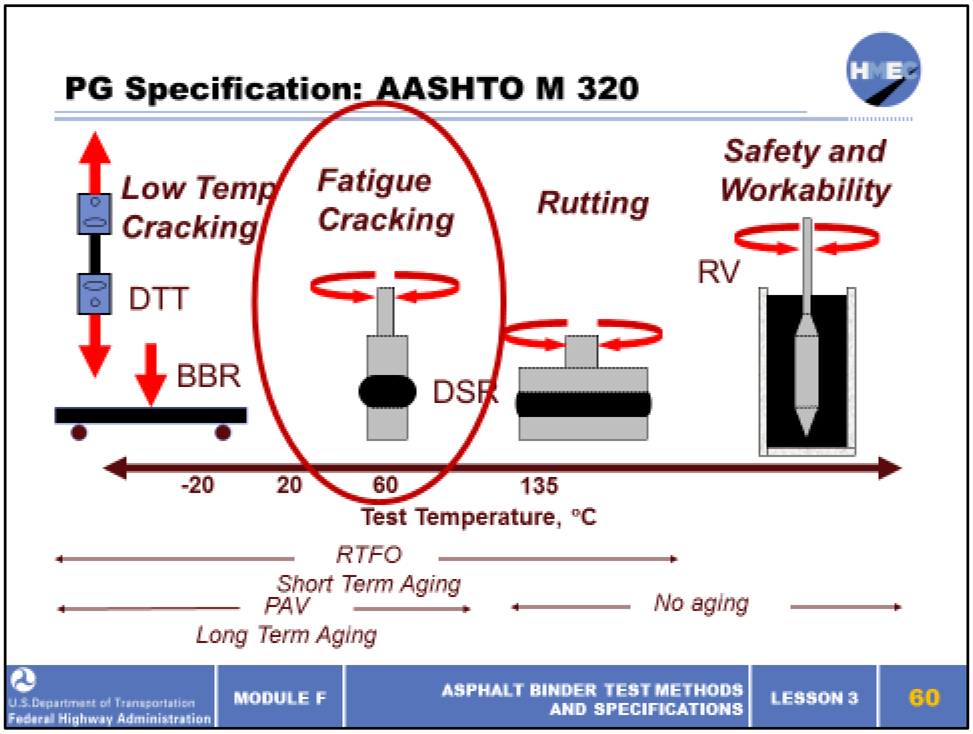
G* in Pascals at 10 rad/sec is numerically equal to viscosity in poises. Since increasing the viscosity of asphalt has been historically used as one means of helping to reduce rutting, using G* in this specification parameter makes sense. Including the sin d parameter allows a softer but more elastic (less permanent deformation per load cycle) binder to be used rather than simply increasing the viscosity.

Slide 59



The specification requirement or stiffness does not change; the temperature at which the value has to meet the criterion changes with grade. This was demonstrated and discussed within Lesson 2.

Slide 60



PG Specification: AASHTO M 320
Low Temp Cracking
DTT
BBR
Fatigue Cracking
DSR
Rutting
Safety and Workability
RV
-20
20
60
135
Test Temperature, °C
RTFO
Short Term Aging
PAV
Long Term Aging
No aging
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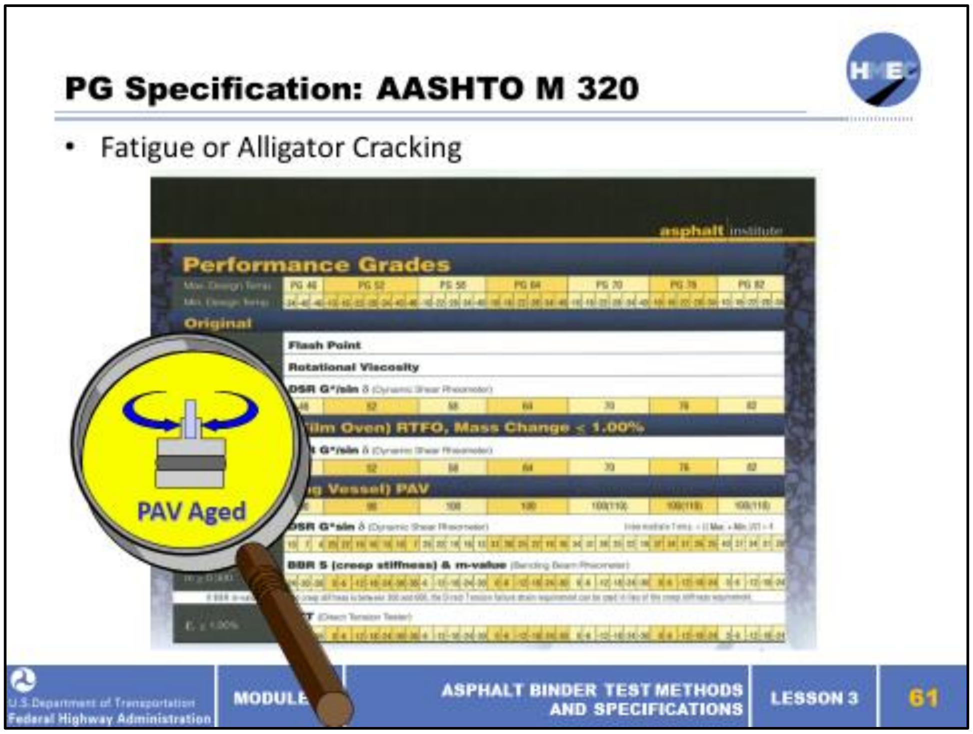

The DSR is used to measure binder stiffness on the RTFO- and PAV-aged residue at intermediate temperatures (recall that PAV aging is performed on the RTFO-aged binder).

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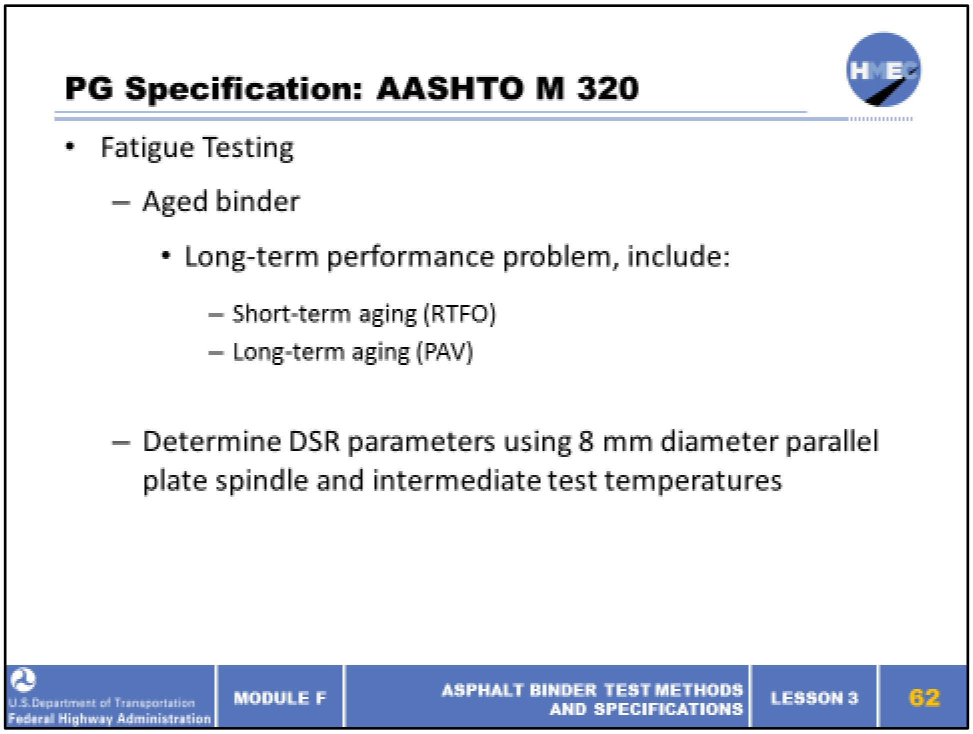

Because the test temperatures used to assess a binder's contribution to resistance to fatigue cracking are conducted at intermediate to low temperatures, an 8 mm diameter plate is used as opposed to the 25 mm diameter plate used for assessing a binder's contribution to rutting resistance.

In summary, fatigue cracking is a long-term performance problem and assessment of a binder's resistance to fatigue cracking should, therefore, include both short-term aging (that occurring in the construction process) and long-term aging (that occurring throughout the life of the HMA pavement).

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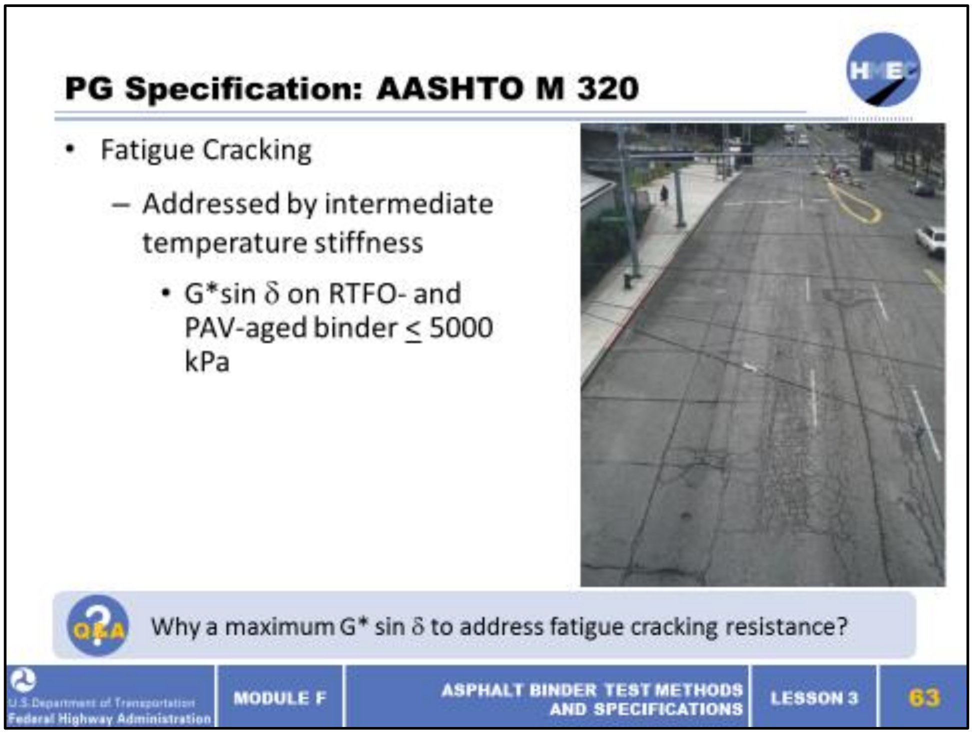

Increased stiffness tends to be associated with increased elasticity (and, consequently, increased brittleness) in asphalt binders.

However, resistance to fatigue cracking requires the binder to be sufficiently pliable at intermediate to low temperatures to resist fracturing.

Use of G* multiplied by the sine of the phase angle (δ) makes sense in this regard as it is a measure of the viscous (loss) modulus, the component of the complex modulus related to pliability (or ability to deform without fracturing).

Let's take a moment to think about what we've learned so far. Lead Q&A discussion.

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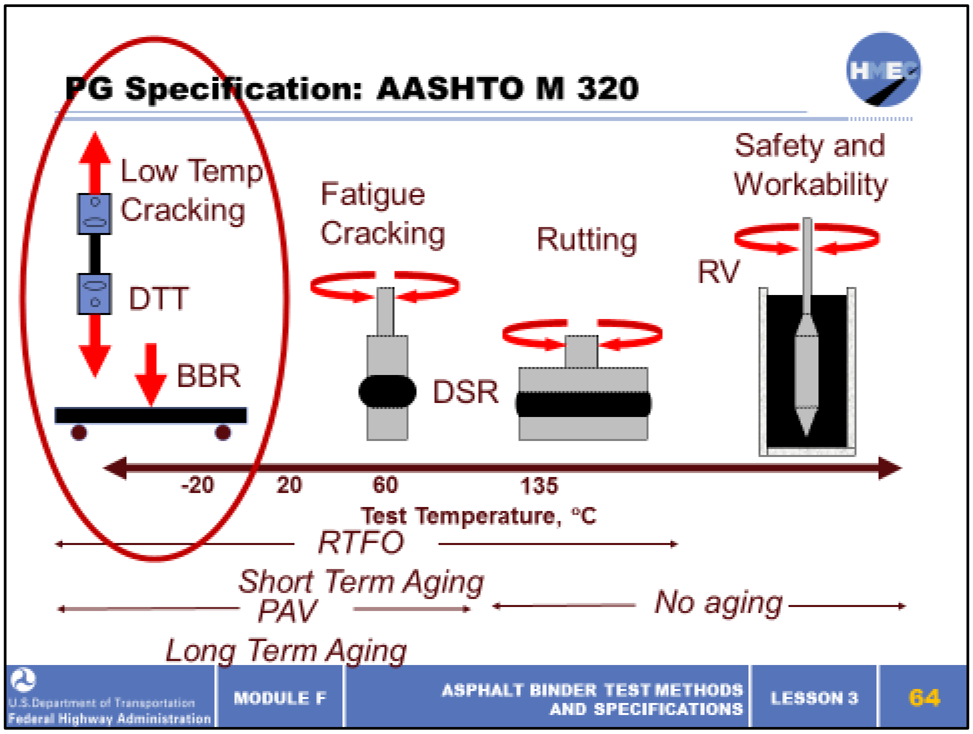
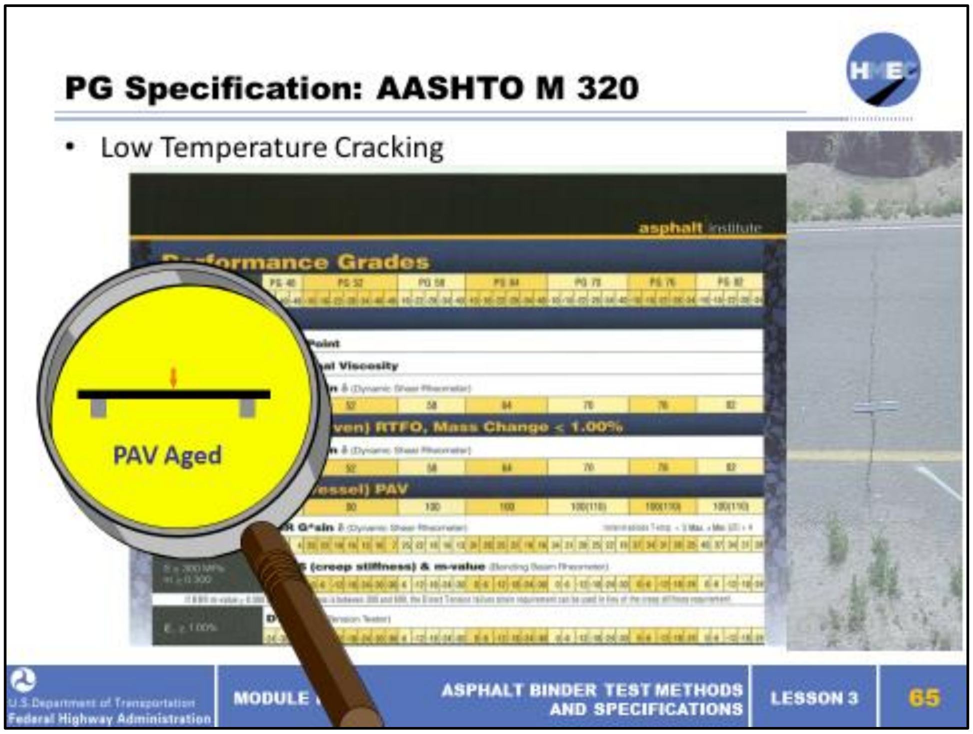


The PG binder specification requires the use of the bending beam rheometer and (possibly) the direct tension test to assess a binder's resistance to thermal cracking.

Slide 65


PG Specification: AASHTO M 320
Low Temperature Cracking
asphalt institute
PAV Aged
U.S. Department of Transportation
Federal Highway Administration
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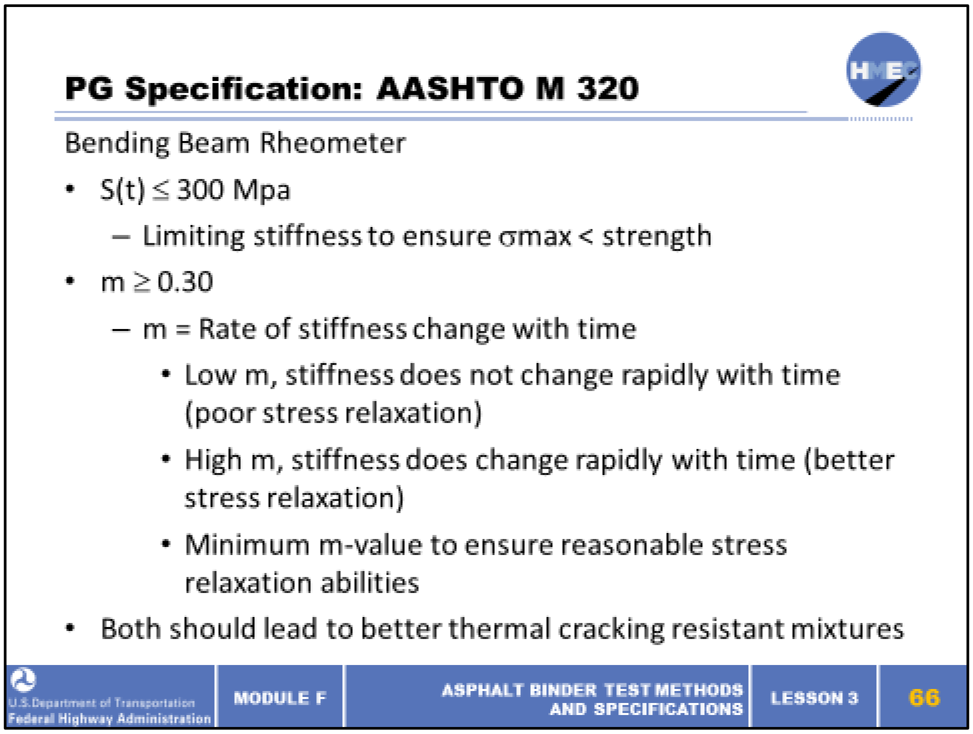
Slide 66

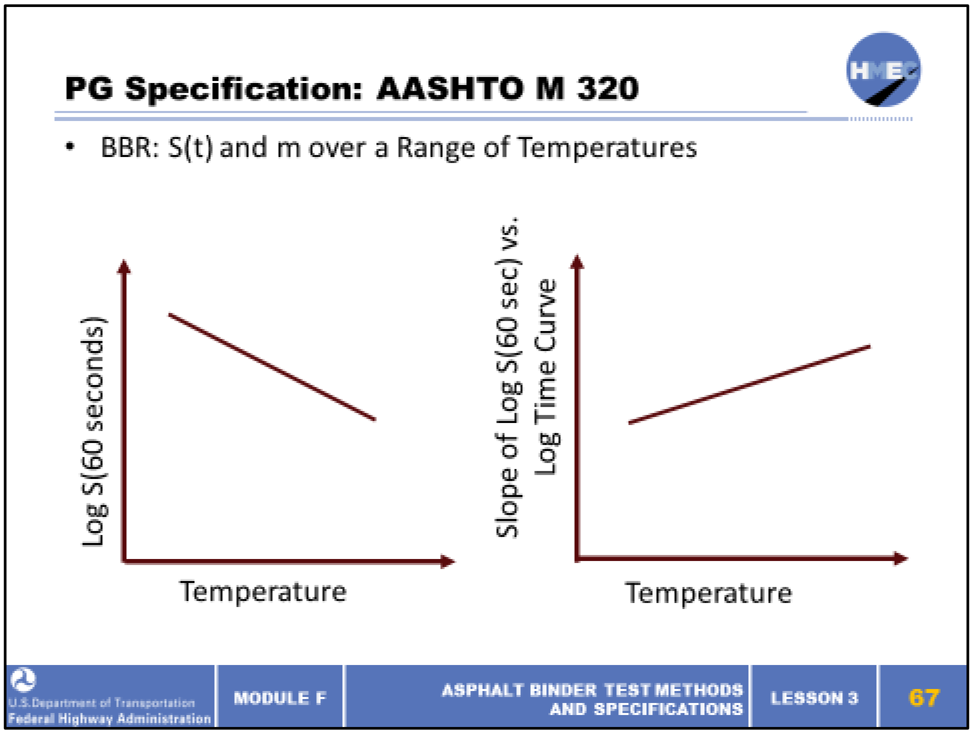


The stiffness requirement is intended to ensure that the maximum bending stress (resulting in maximum tensile strain on the underside of the deflected asphalt binder beam) is less than the tensile strength of the binder at a given temperature.

The m-value requirement is intended to ensure that the asphalt binder possesses reasonable stress relaxation properties.

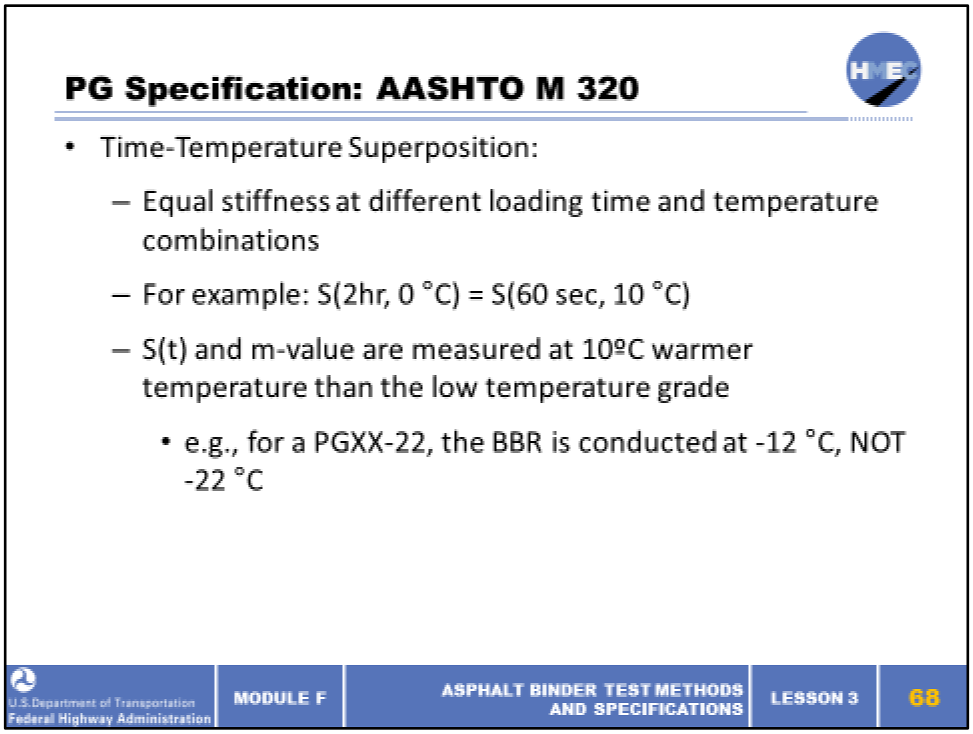
Both criteria should specify a binder that, when mixed with aggregate and compacted into an HMA pavement layer, is resistant to thermal cracking.

Slide 67



The PG binder specification stipulates that the stiffness must be less than or equal to 300 MPa and that the m-value must be greater than or equal to 0.30. Thus, a binder must not only possess sufficient pliability at low temperatures, it must also possess the ability to recover adequately after a load is removed.

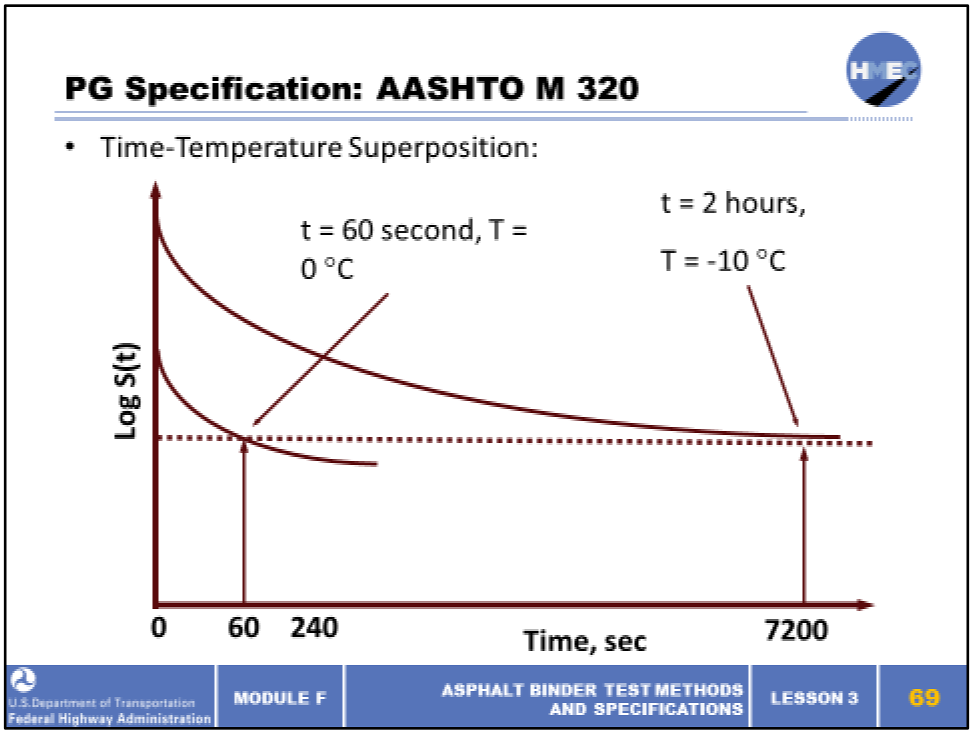
Slide 68



For example, a binder could have the same stiffness after two hours of static loading at 0 °C as it does after only 60 seconds of loading at 10 °C.

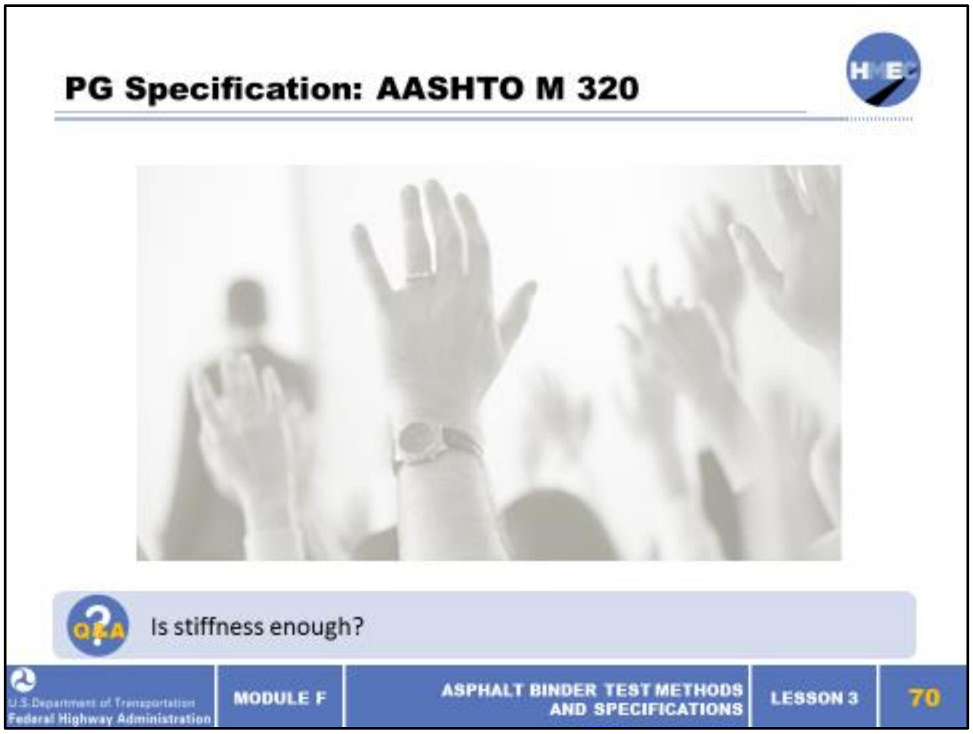
In the BBR, asphalt binders are tested at a temperature that is 10°C warmer than the low temperature grade that, by the principle of time-temperature superposition, equivalent to the stiffness of the binder at the low temperature grade after 2 hours of loading. Thus, for a PGXX-22, BBR tests are conducted at -12 °C, not -22 °C.

Slide 69



The binder exhibits the same stiffness after 2 hours of loading at -10 °C as it does after only 60 seconds of loading at 0 °C.

Slide 70


PG Specification: AASHTO M 320
HMEC
Q&A
Is stiffness enough?
U.S. Department of Transportation
Federal Highway Administration
MODULE F
ASPHALT BINDER TEST METHODS AND SPECIFICATIONS
LESSON 3
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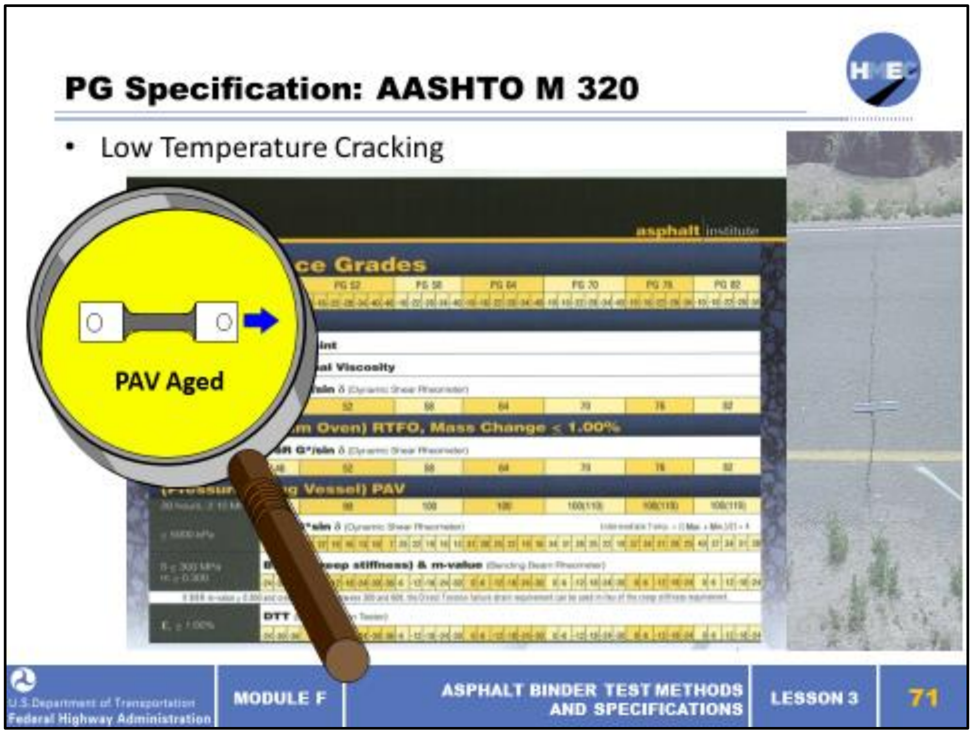
Slide 71


PG Specification: AASHTO M 320
• Low Temperature Cracking
PAV Aged
asphalt institute
MODULE F
ASPHALT BINDER TEST METHODS AND SPECIFICATIONS
LESSON 3
71
U.S. Department of Transportation
Federal Highway Administration

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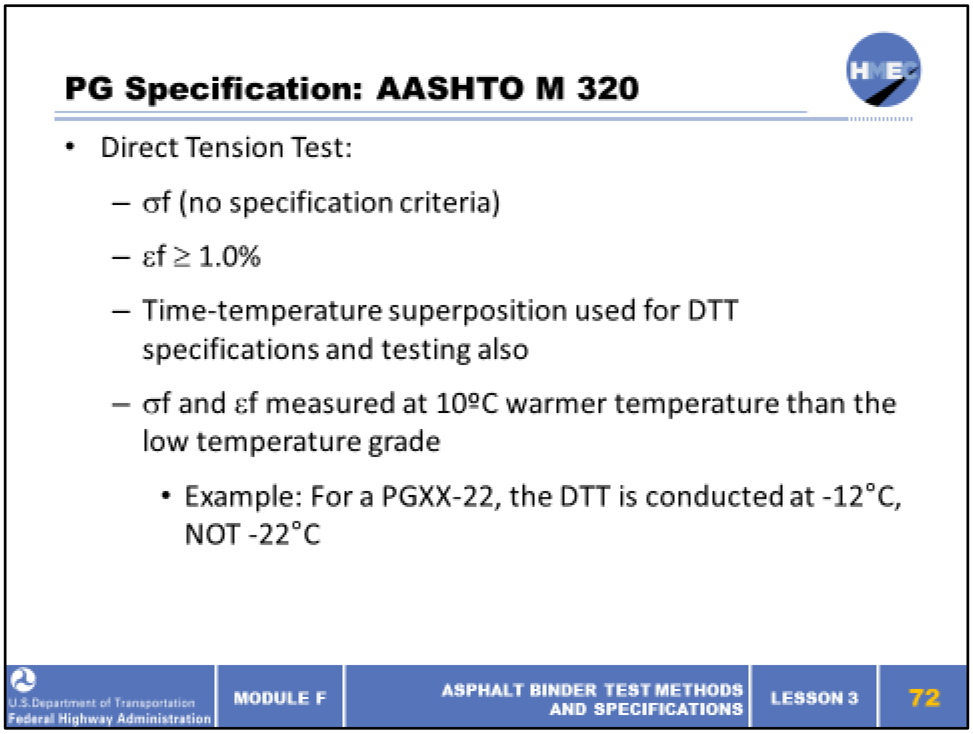
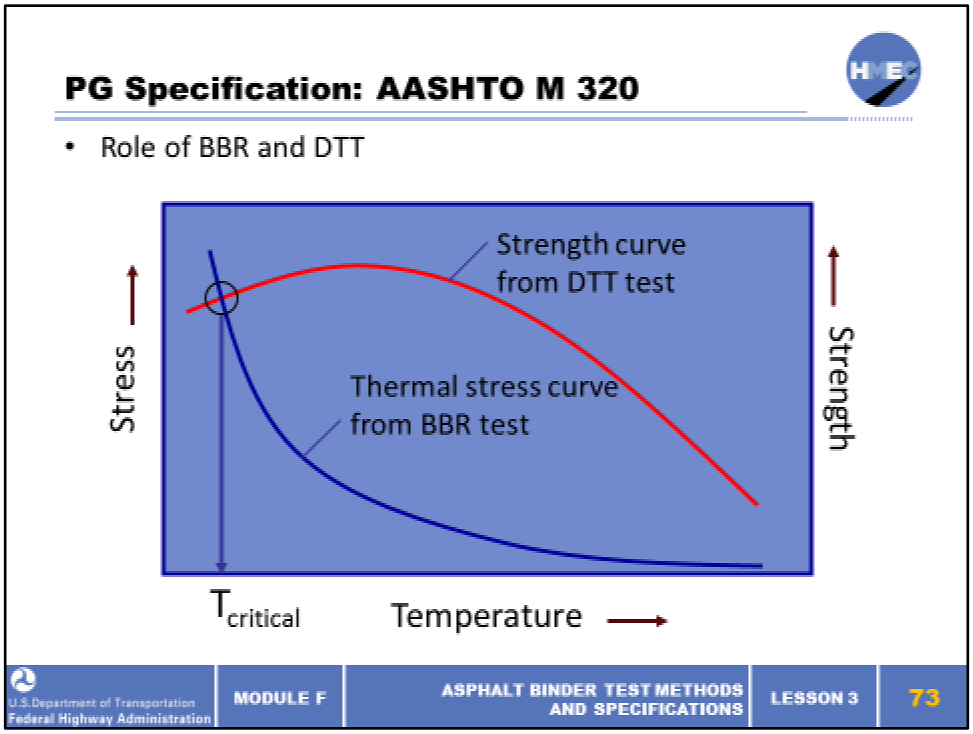
Slide 72



The specification stipulates that the strain at failure, ε_f, must be greater than or equal to 1.0%.

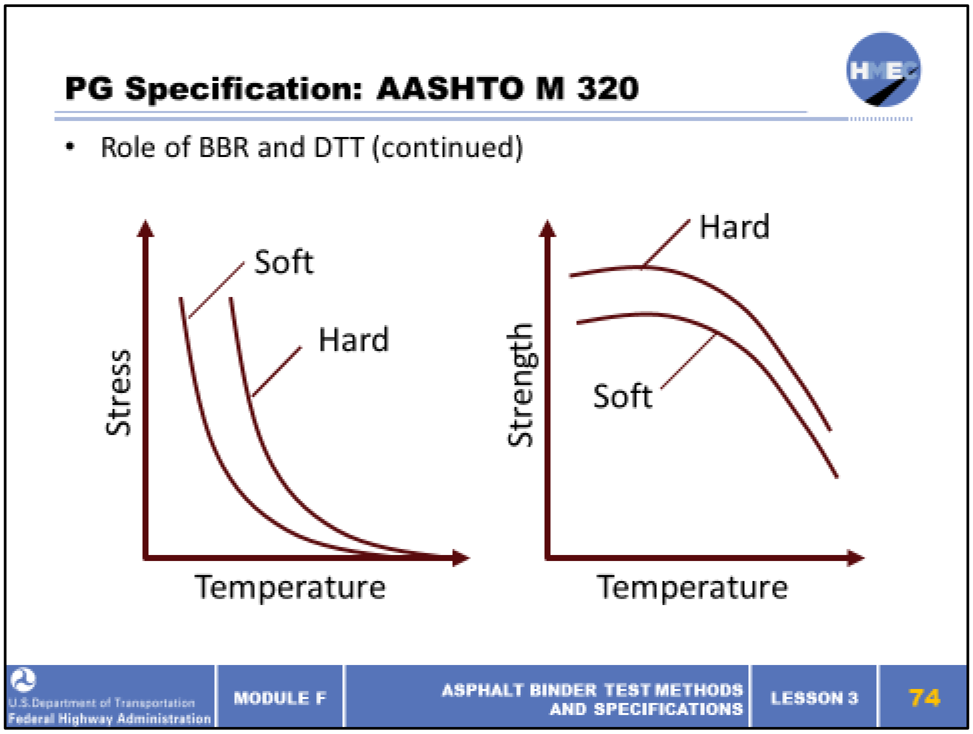
As with BBR testing, the direct tension tests are carried out at a temperature that is 10 °C warmer than the low temperature grade.

Slide 73



The criteria for the BBR and DTT in the PG specification have been established to ensure the stress developed in the binder is less than its fracture strength at a given temperature.

Slide 74



By selecting a soft binder for improved thermal cracking resistance, a sacrifice is made in its strength properties. This illustrates that a specification containing low temperature criteria must consider both properties to ensure adequate low temperature cracking resistance.

Slide 75

Exercise 1: PG Specification

Test	Property	Results	Criteria
PG Grade =			
Original Binder			
Flash Point	N/A		
Rotational Viscometer	135°C		
Rotational Viscometer	165°C		
Dynamic Shear Rheometer G*/sin(δ)	58°C		
	64°C		
	70°C		
	76°C		
RTFO Residue – Aged Binder			
Mass Loss	N/A		
Dynamic Shear Rheometer G*/sin(δ)	58°C		
	64°C		
	70°C		
	76°C		
RTFO + PAV Residue – Aged Binder			
Dynamic Shear Rheometer G*•sin(δ)	19°C		
	25°C		
	28°C		
Bending Beam Rheometer, Stiffness, S	-24°C		
	-18°C		
	-12°C		
Bending Beam Rheometer, m-value	-24°C		
	-18°C		
	-12°C		

Let's divide into groups for an exercise. Take about 10 minutes to complete your assigned PG Grade Interpretation Table.

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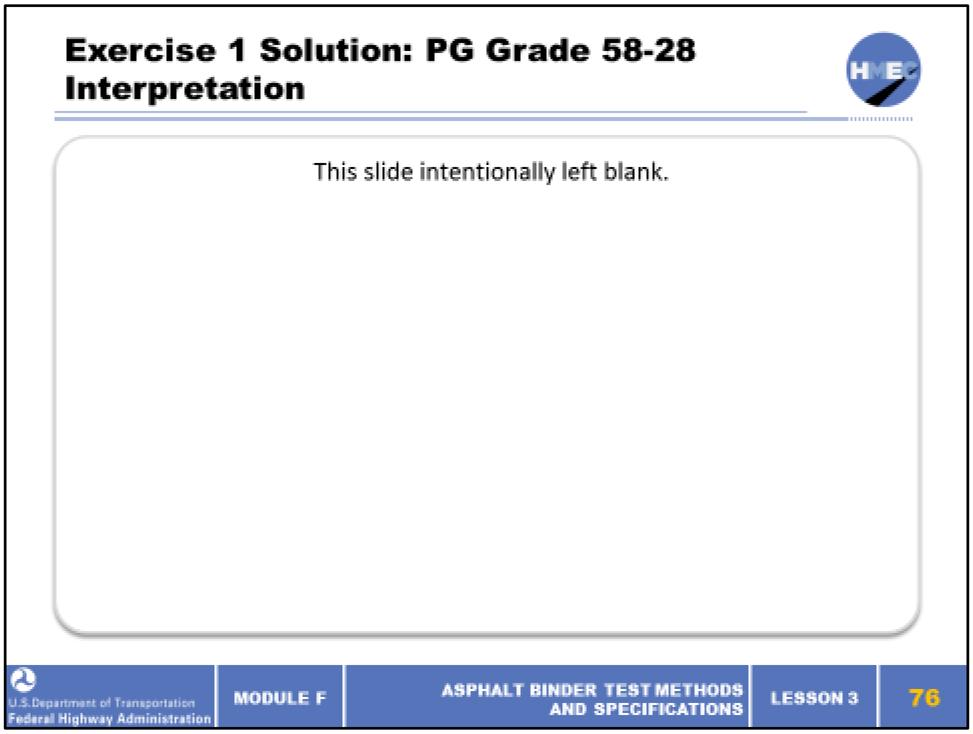
Exercise 1 Solution: PG Grade 58-28 Interpretation
HMEC
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U.S. Department of Transportation
Federal Highway Administration
MODULE F
ASPHALT BINDER TEST METHODS AND SPECIFICATIONS
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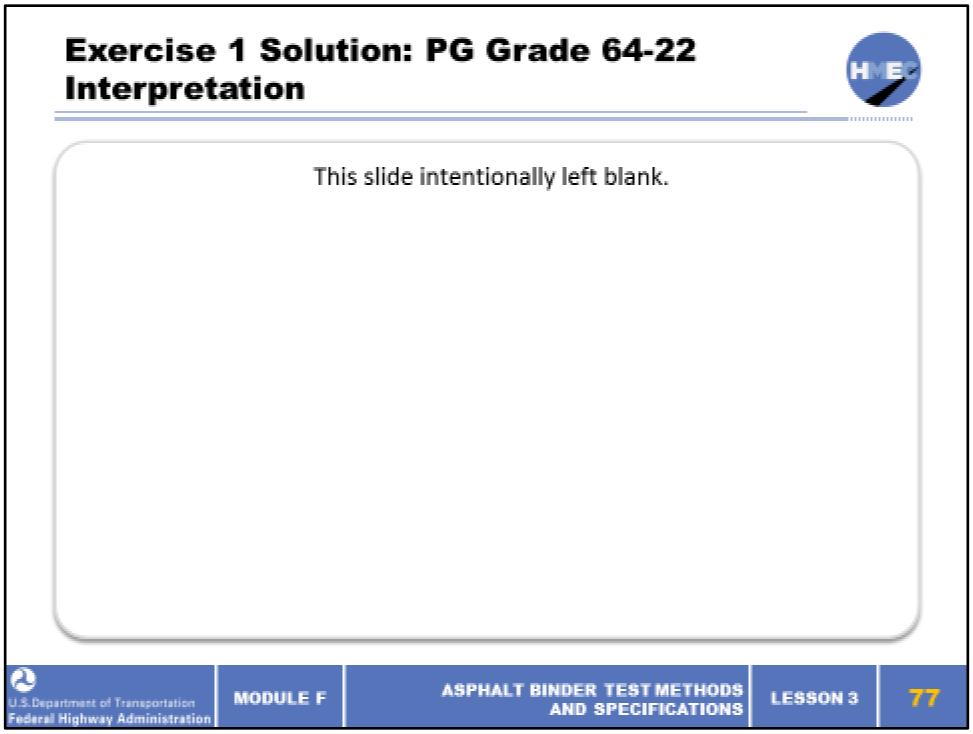
Exercise 1 Solution: PG Grade 64-22 Interpretation
HMEC
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U.S. Department of Transportation
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MODULE F
ASPHALT BINDER TEST METHODS AND SPECIFICATIONS
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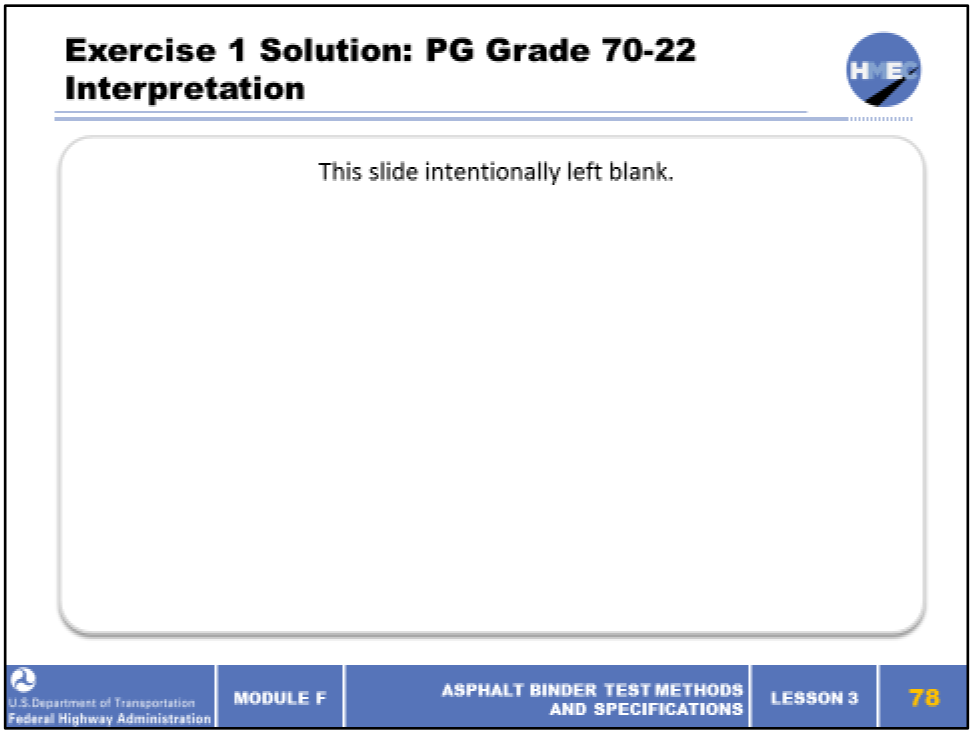
Slide 78


Exercise 1 Solution: PG Grade 70-22 Interpretation
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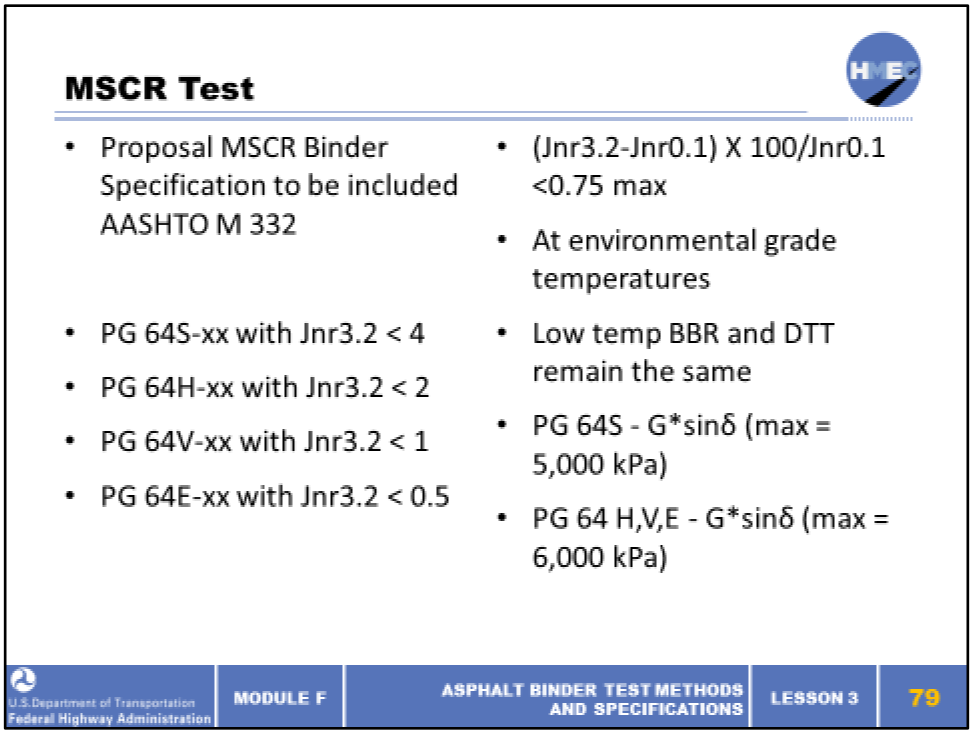
Slide 79


MSCR Test
HMEC
Proposal MSCR Binder Specification to be included AASHTO M 332
PG 64S-xx with Jnr3.2 < 4
PG 64H-xx with Jnr3.2 < 2
PG 64V-xx with Jnr3.2 < 1
PG 64E-xx with Jnr3.2 < 0.5
(Jnr3.2-Jnr0.1) X 100/Jnr0.1 <0.75 max
At environmental grade temperatures
Low temp BBR and DTT remain the same
PG 64S - G*sinδ (max = 5,000 kPa)
PG 64 H,V,E - G*sinδ (max = 6,000 kPa)
U.S. Department of Transportation Federal Highway Administration
MODULE F
ASPHALT BINDER TEST METHODS AND SPECIFICATIONS
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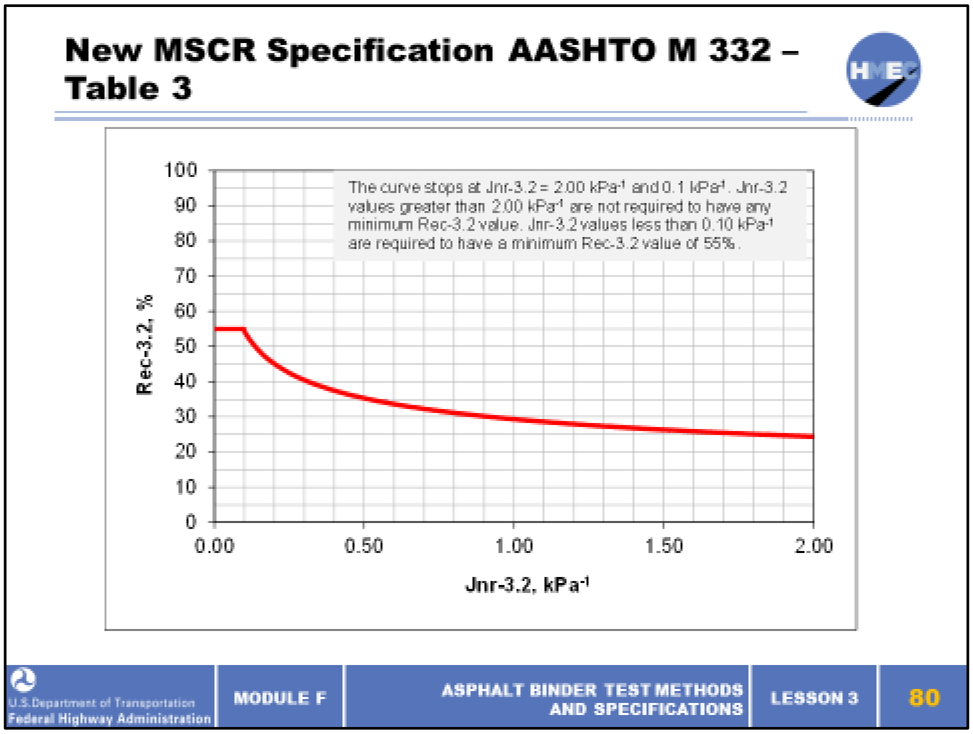
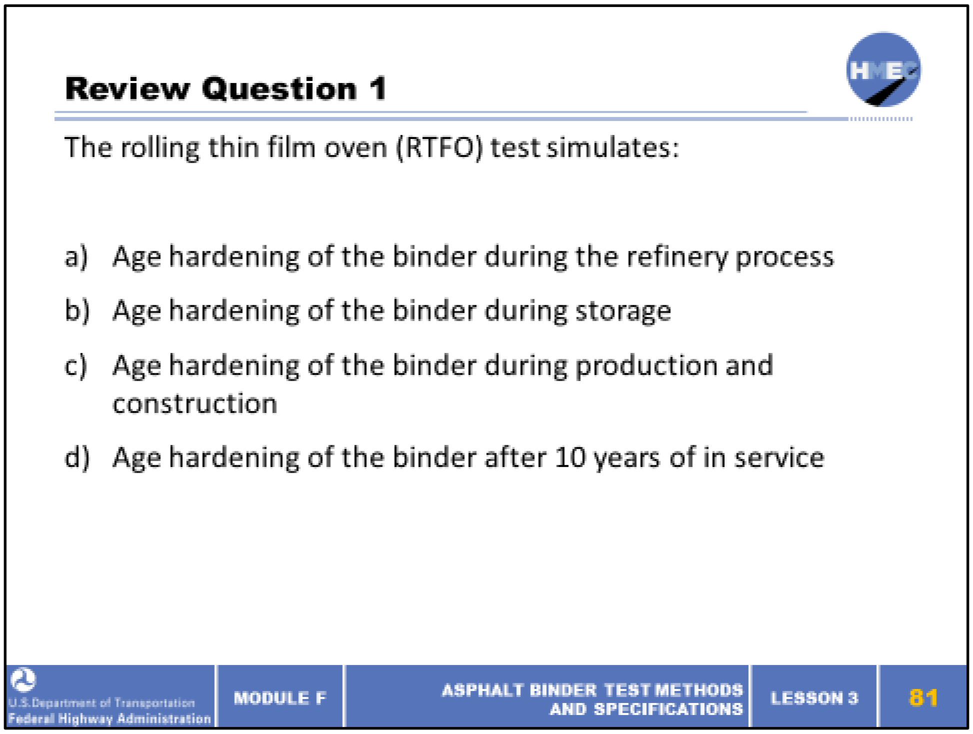


The data on the red line and above it indicates adequate Jnr and % recovery according to AASHTO M 332 Table 3

Slide 81


Review Question 1
The rolling thin film oven (RTFO) test simulates:
a) Age hardening of the binder during the refinery process
b) Age hardening of the binder during storage
c) Age hardening of the binder during production and construction
d) Age hardening of the binder after 10 years of in service
U.S. Department of Transportation
Federal Highway Administration
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ASPHALT BINDER TEST METHODS AND SPECIFICATIONS
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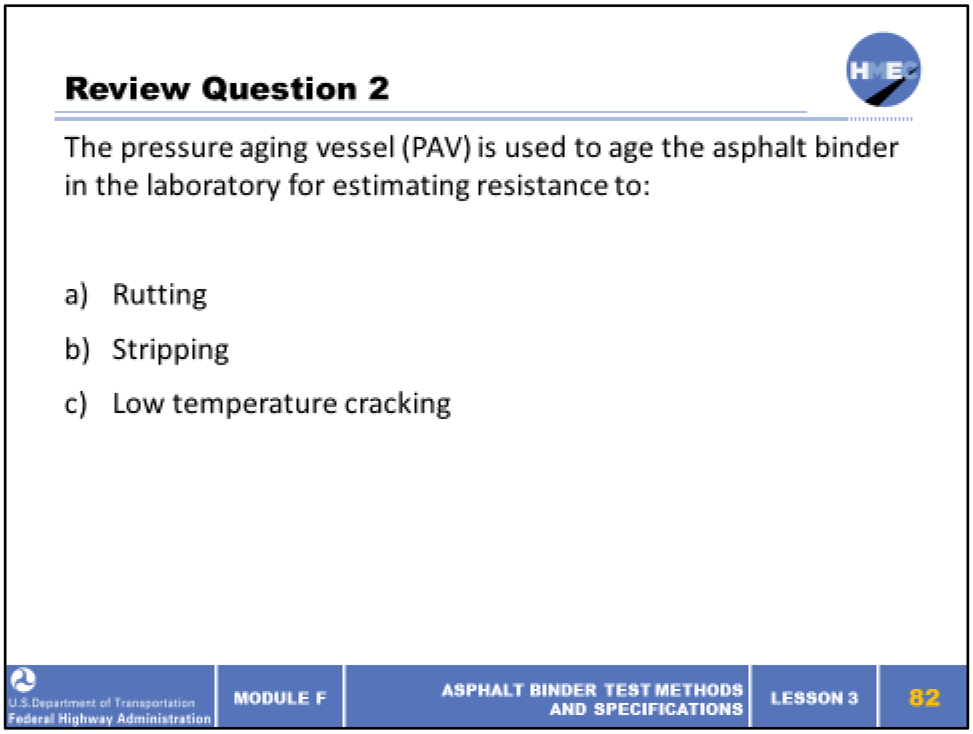
Review Question 2
HMEC
The pressure aging vessel (PAV) is used to age the asphalt binder in the laboratory for estimating resistance to:
a) Rutting
b) Stripping
c) Low temperature cracking
U.S. Department of Transportation
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ASPHALT BINDER TEST METHODS AND SPECIFICATIONS
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Review Question 3

The rotational viscometer (RV) estimates which following properties of the asphalt binder?

a) Pumpability

b) Ductility

c) Strength

d) Specific gravity

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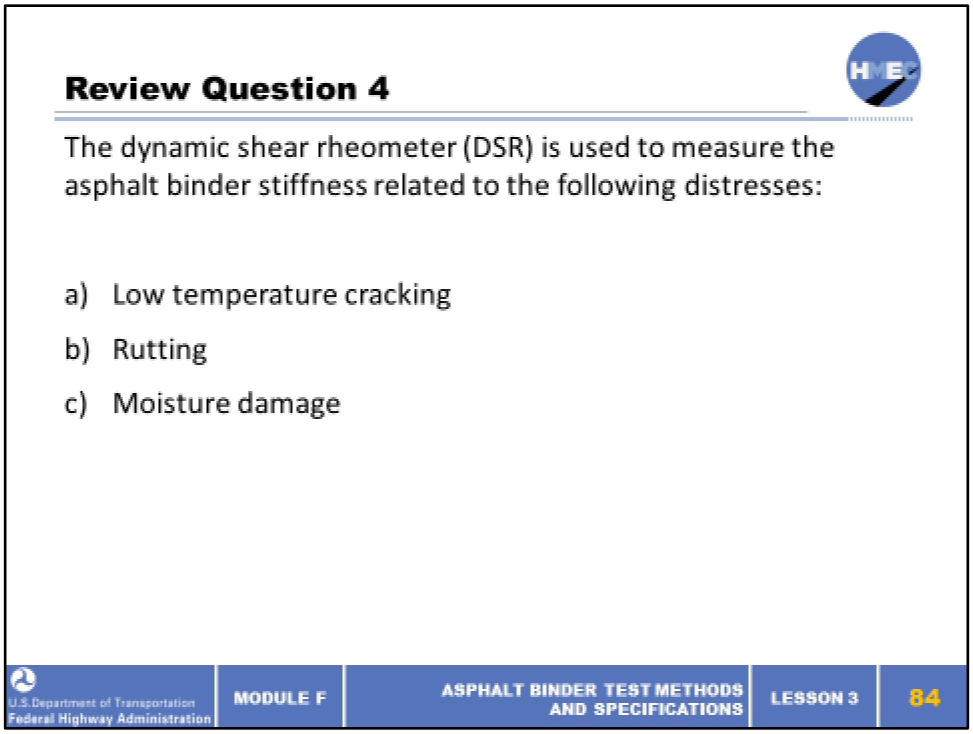
Review Question 4
HMEC
The dynamic shear rheometer (DSR) is used to measure the asphalt binder stiffness related to the following distresses:
a) Low temperature cracking
b) Rutting
c) Moisture damage
U.S. Department of Transportation
Federal Highway Administration
MODULE F
ASPHALT BINDER TEST METHODS AND SPECIFICATIONS
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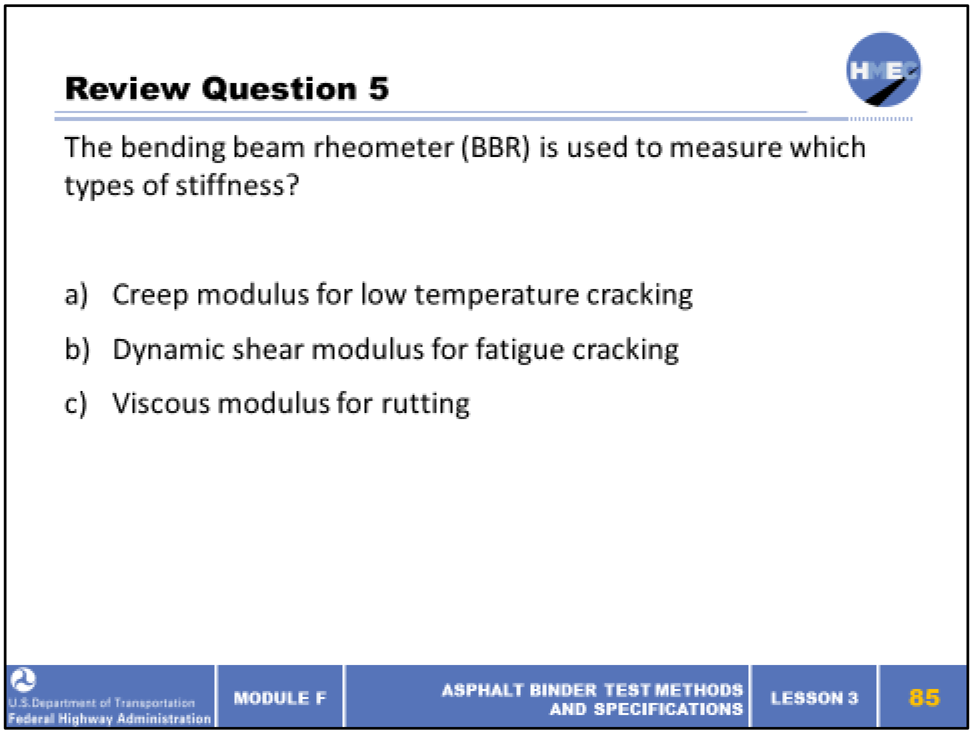
Review Question 5
HMEC
The bending beam rheometer (BBR) is used to measure which types of stiffness?
a) Creep modulus for low temperature cracking
b) Dynamic shear modulus for fatigue cracking
c) Viscous modulus for rutting
U.S. Department of Transportation
Federal Highway Administration
MODULE F
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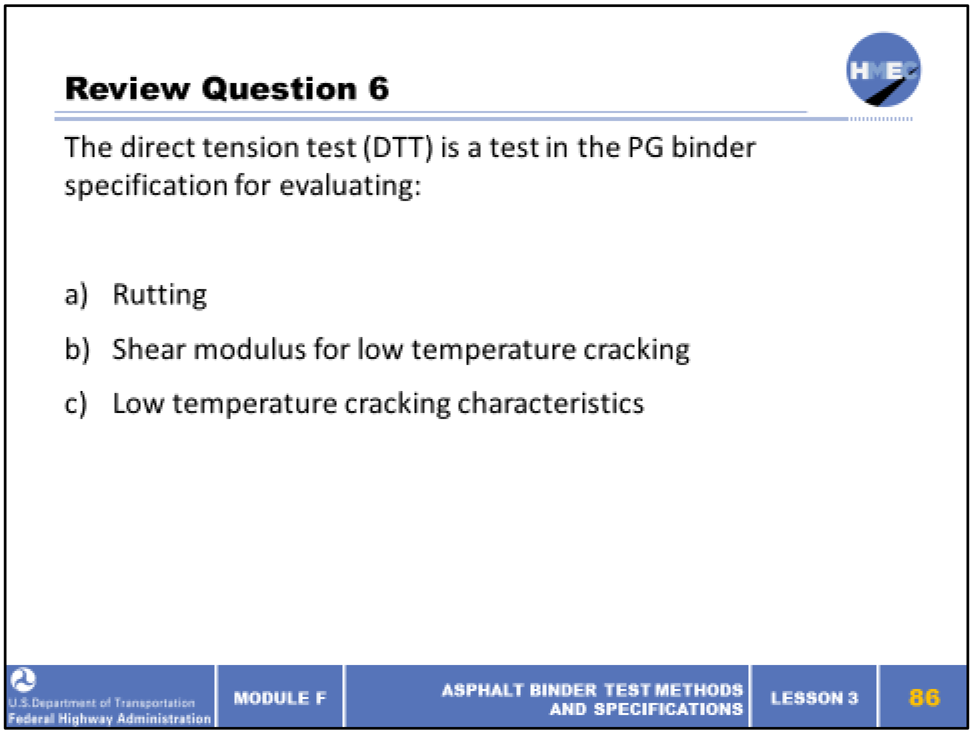
Slide 86


Review Question 6
The direct tension test (DTT) is a test in the PG binder specification for evaluating:
a) Rutting
b) Shear modulus for low temperature cracking
c) Low temperature cracking characteristics
U.S. Department of Transportation
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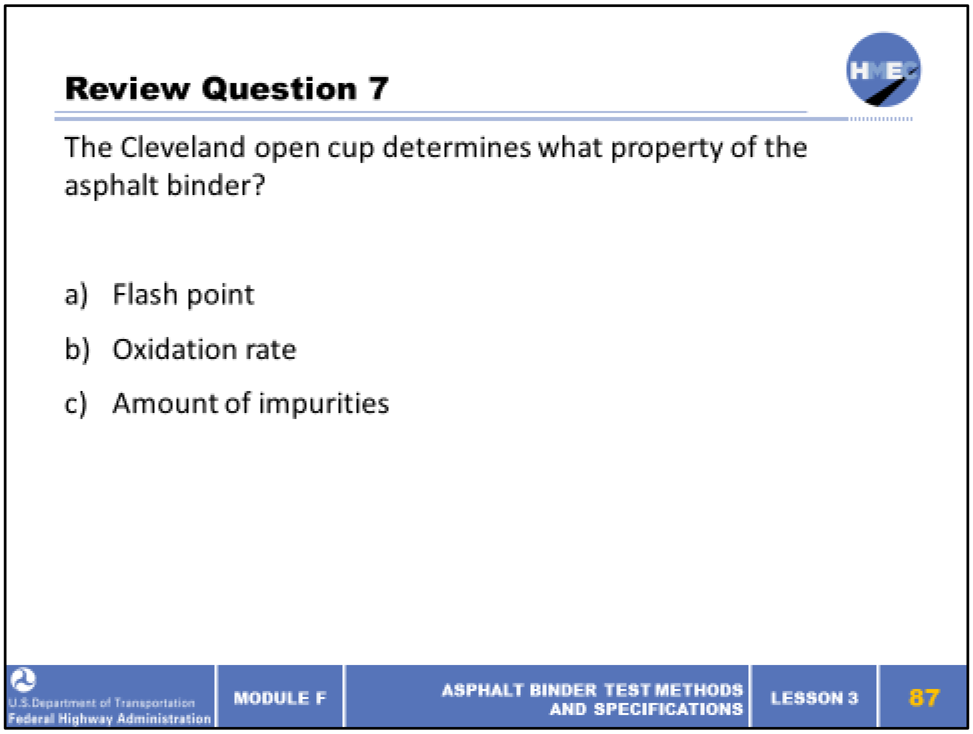
Review Question 7
The Cleveland open cup determines what property of the asphalt binder?
a) Flash point
b) Oxidation rate
c) Amount of impurities
U.S. Department of Transportation
Federal Highway Administration
MODULE F
ASPHALT BINDER TEST METHODS AND SPECIFICATIONS
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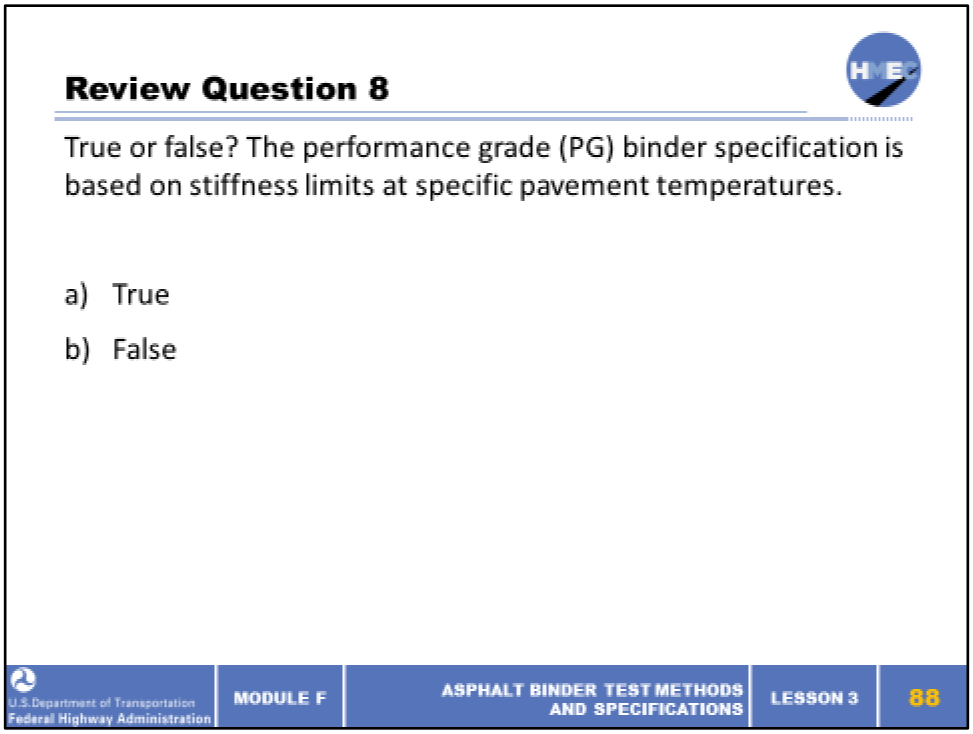
Review Question 8
HMEC
True or false? The performance grade (PG) binder specification is based on stiffness limits at specific pavement temperatures.
a) True
b) False
U.S.Department of Transportation
Federal Highway Administration
MODULE F
ASPHALT BINDER TEST METHODS AND SPECIFICATIONS
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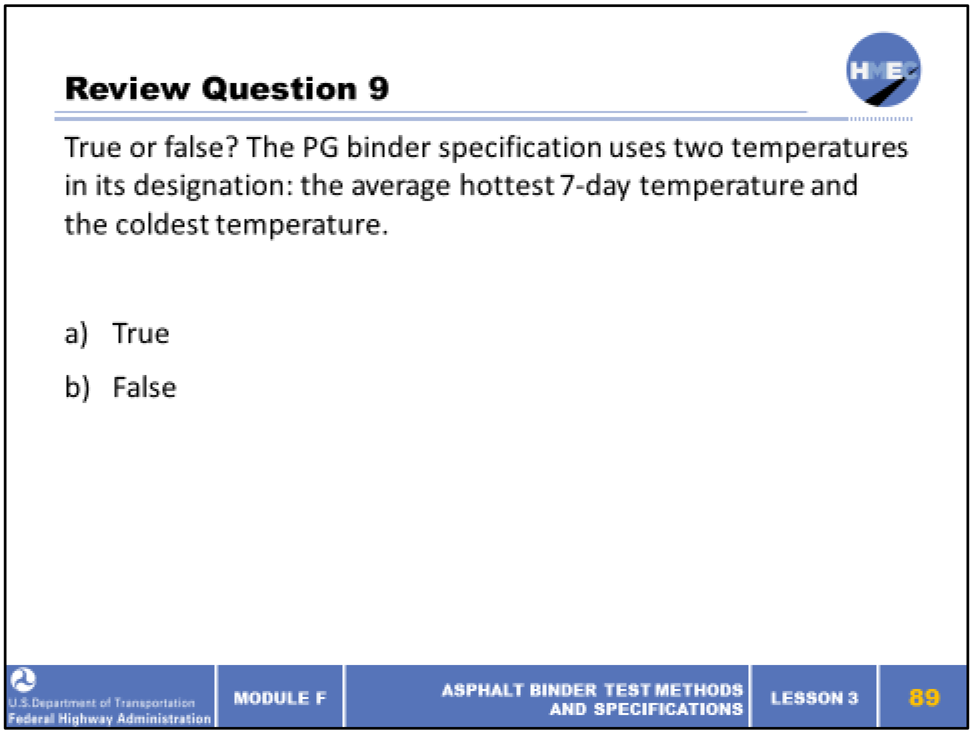
Review Question 9
True or false? The PG binder specification uses two temperatures in its designation: the average hottest 7-day temperature and the coldest temperature.
a) True
b) False
U.S. Department of Transportation
Federal Highway Administration
MODULE F
ASPHALT BINDER TEST METHODS AND SPECIFICATIONS
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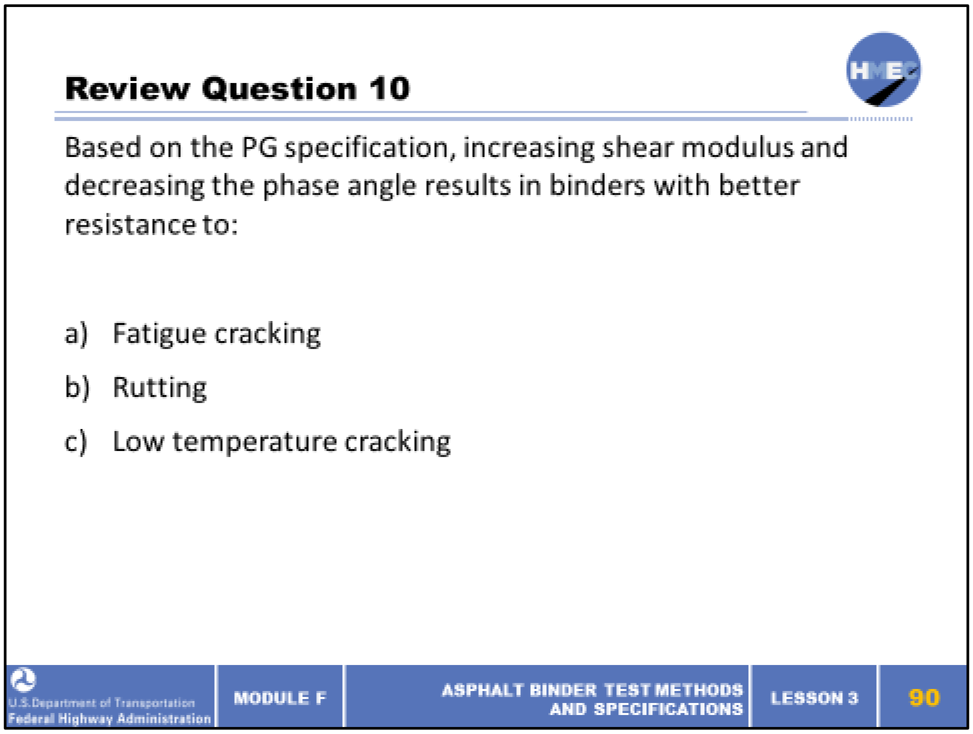
Review Question 10
Based on the PG specification, increasing shear modulus and decreasing the phase angle results in binders with better resistance to:
a) Fatigue cracking
b) Rutting
c) Low temperature cracking
U.S. Department of Transportation
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ASPHALT BINDER TEST METHODS AND SPECIFICATIONS
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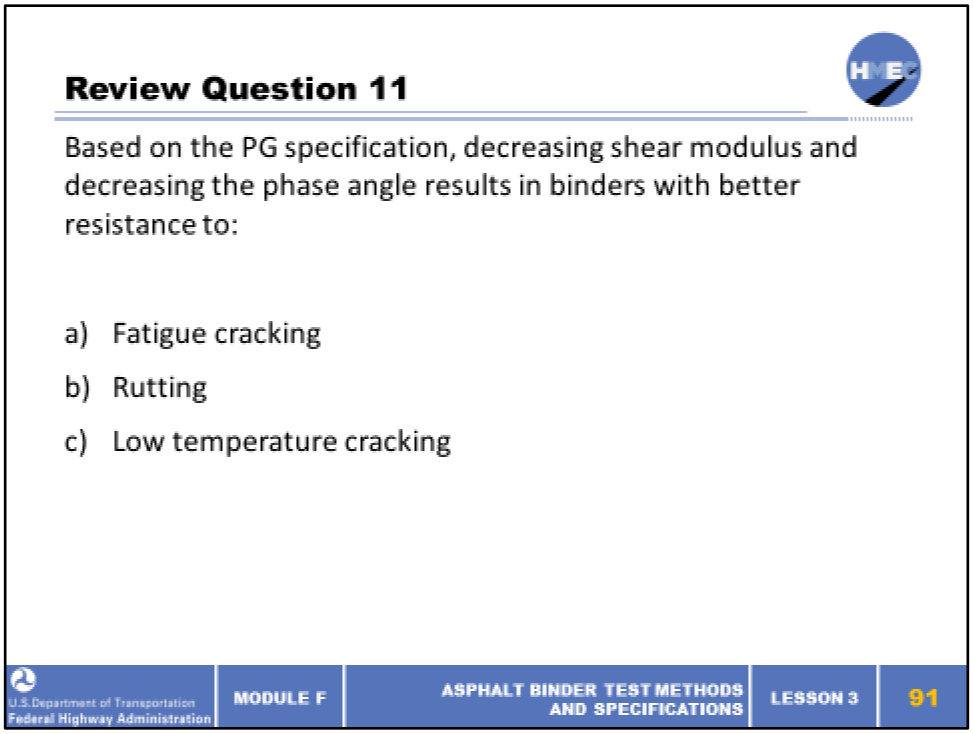
Review Question 11
Based on the PG specification, decreasing shear modulus and decreasing the phase angle results in binders with better resistance to:
a) Fatigue cracking
b) Rutting
c) Low temperature cracking
U.S. Department of Transportation
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MODULE F
ASPHALT BINDER TEST METHODS AND SPECIFICATIONS
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Review Question 12

True or false? Based on the PG specification, the bending beam rheometer (BBR) estimates the strength of the asphalt binder for resistance to low temperature cracking.

a) True

b) False

U.S. Department of Transportation Federal Highway Administration | MODULE F | ASPHALT BINDER TEST METHODS AND SPECIFICATIONS | LESSON 3 | 92

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Lab Experience
Preparing for Lab Experience: Safety
U.S. Department of Transportation
Federal Highway Administration
MODULE F
ASPHALT BINDER TEST METHODS AND SPECIFICATIONS
LESSON 3
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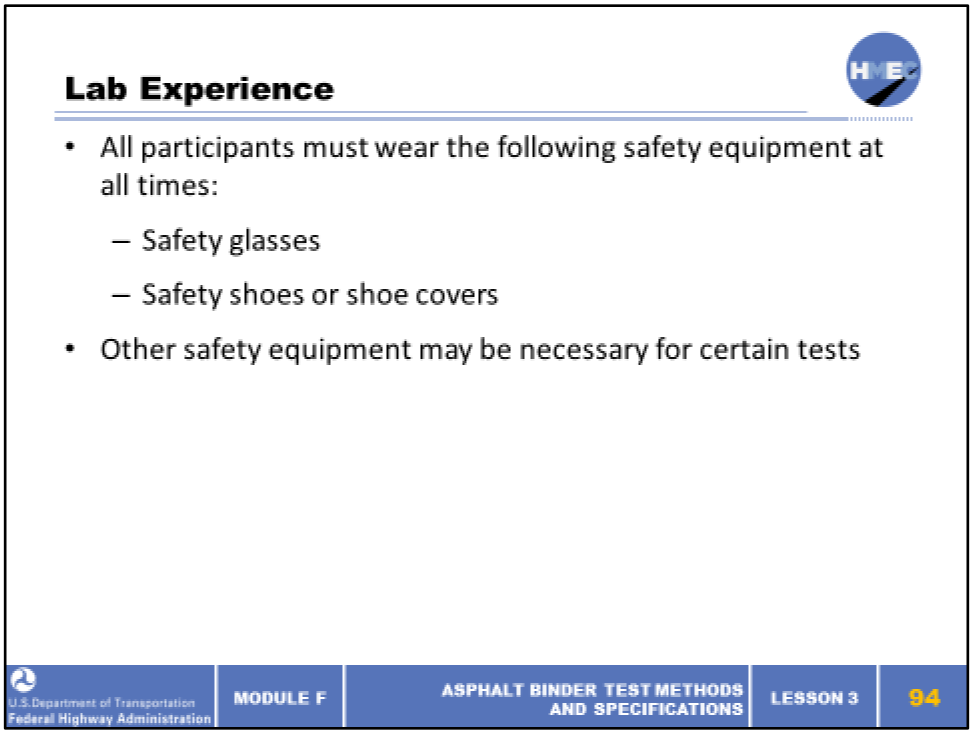
Slide 94


Lab Experience
- All participants must wear the following safety equipment at all times:
  - Safety glasses
  - Safety shoes or shoe covers
- Other safety equipment may be necessary for certain tests
U.S. Department of Transportation
Federal Highway Administration
MODULE F
ASPHALT BINDER TEST METHODS AND SPECIFICATIONS
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Lab Experience

- Hazard exposures in the laboratory:
  - Chemical: See safety data sheet (SDS) handouts for more information
    - Excel Clean HD (a citrus-based cleaner)
    - Glycerin-Talc Mixture
    - Ethanol
    - Asphalt

U.S. Department of Transportation Federal Highway Administration | MODULE F | ASPHALT BINDER TEST METHODS AND SPECIFICATIONS | LESSON 3 | 95

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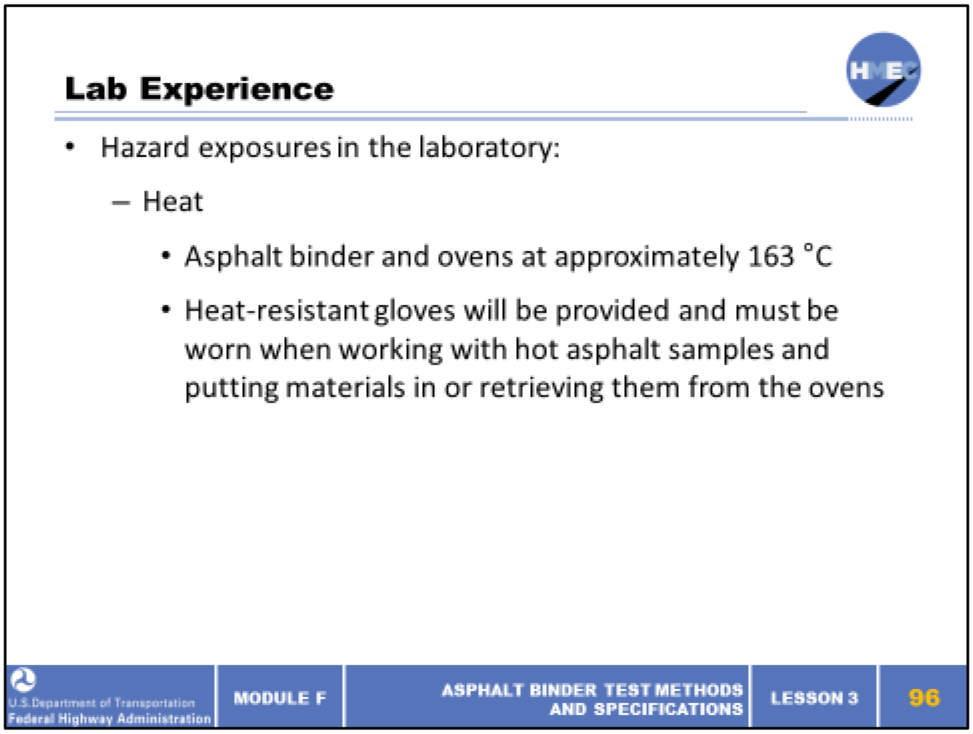
Lab Experience
HMEC
• Hazard exposures in the laboratory:
– Heat
• Asphalt binder and ovens at approximately 163 °C
• Heat-resistant gloves will be provided and must be worn when working with hot asphalt samples and putting materials in or retrieving them from the ovens
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ASPHALT BINDER TEST METHODS AND SPECIFICATIONS
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Lab Experience

- Ensuring your safety in the laboratory:
  - For your safety, please follow all instructions provided by the laboratory experience instructors
  - Do not touch or handle equipment unless you have been given permission to do so
  - There may be times when you will be required to use a laboratory ventilation hood
    - None of the chemicals we will be working with today require the use of a hood
    - Hoods are always on in the AMRL laboratory
    - If proper flow is not achieved, an alarm will sound

U.S. Department of Transportation Federal Highway Administration | MODULE F | ASPHALT BINDER TEST METHODS AND SPECIFICATIONS | LESSON 3 | 97

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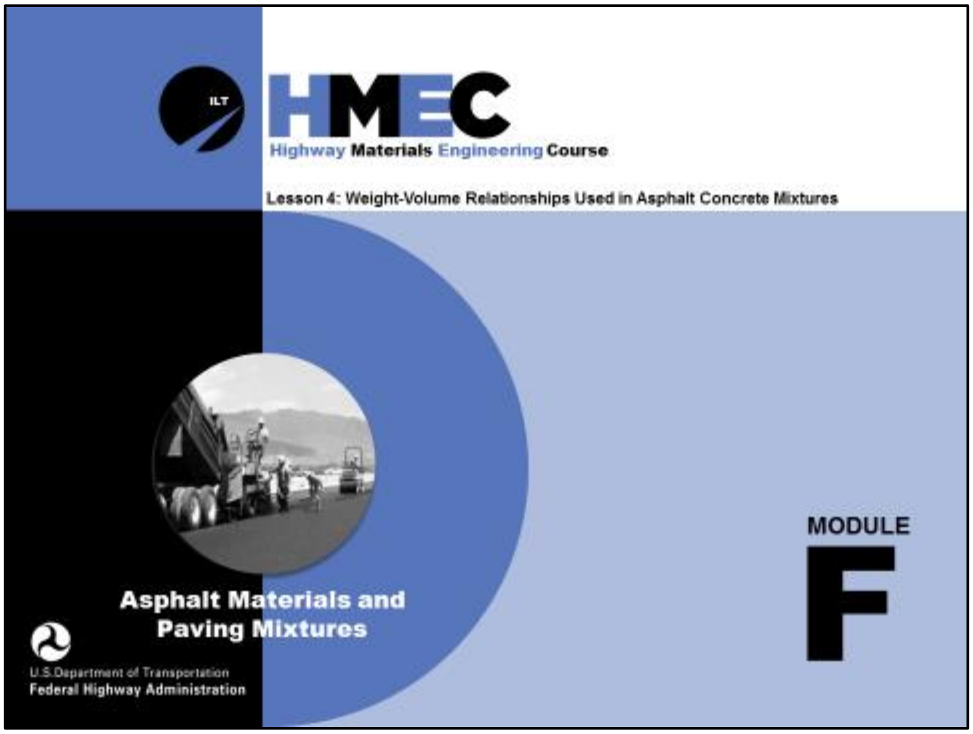
Learning Outcomes Review

You are now able to:

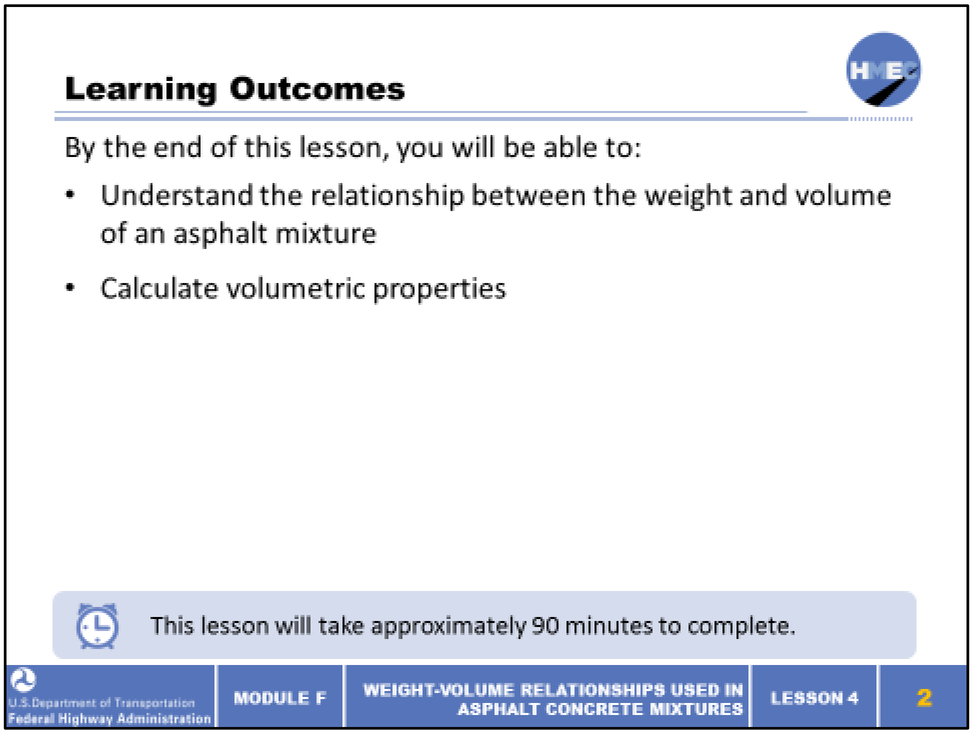
- Explain how laboratory aging should correlate to performance
- Describe the rheological properties of asphalt binders
- Describe non-rheological asphalt binder tests and their importance
- Use laboratory data to grade an asphalt binder according to the Superpave system
- Understand the chemical and physical hazards associated with asphalt binder laboratory testing
- Recognize the equipment and materials required to run each test and learn how to perform each test in a laboratory environment

U.S. Department of Transportation Federal Highway Administration | MODULE F | ASPHALT BINDER TEST METHODS AND SPECIFICATIONS | LESSON 3 | 98

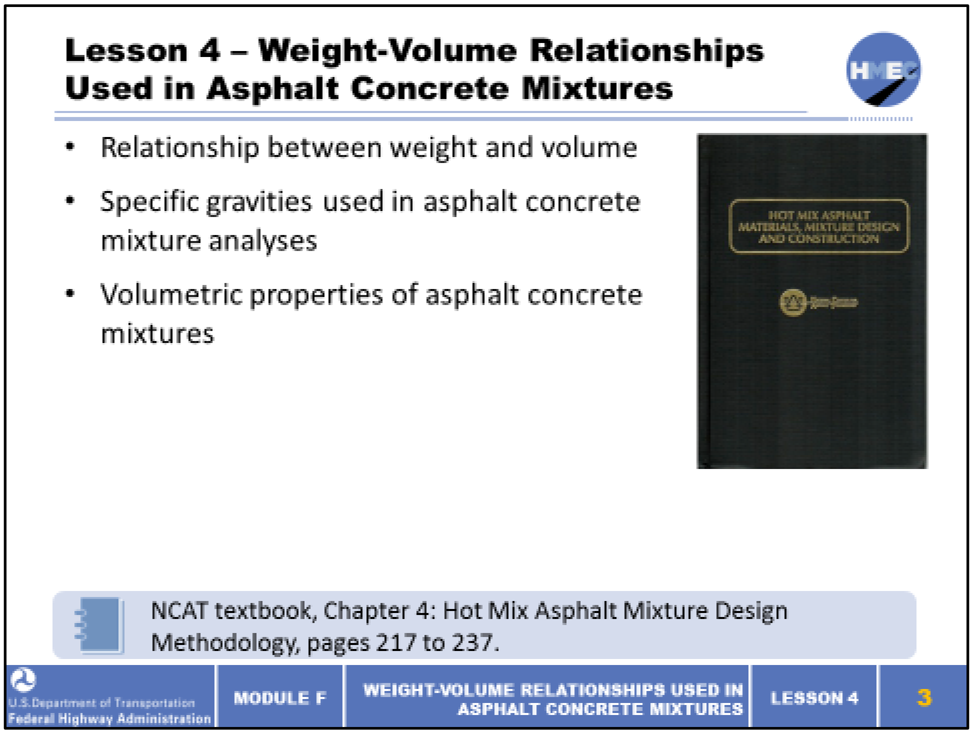
Slide 1


HMEC
Highway Materials Engineering Course
Lesson 4: Weight-Volume Relationships Used in Asphalt Concrete Mixtures
Asphalt Materials and
Paving Mixtures
U.S.Department of Transportation
Federal Highway Administration
MODULE
F

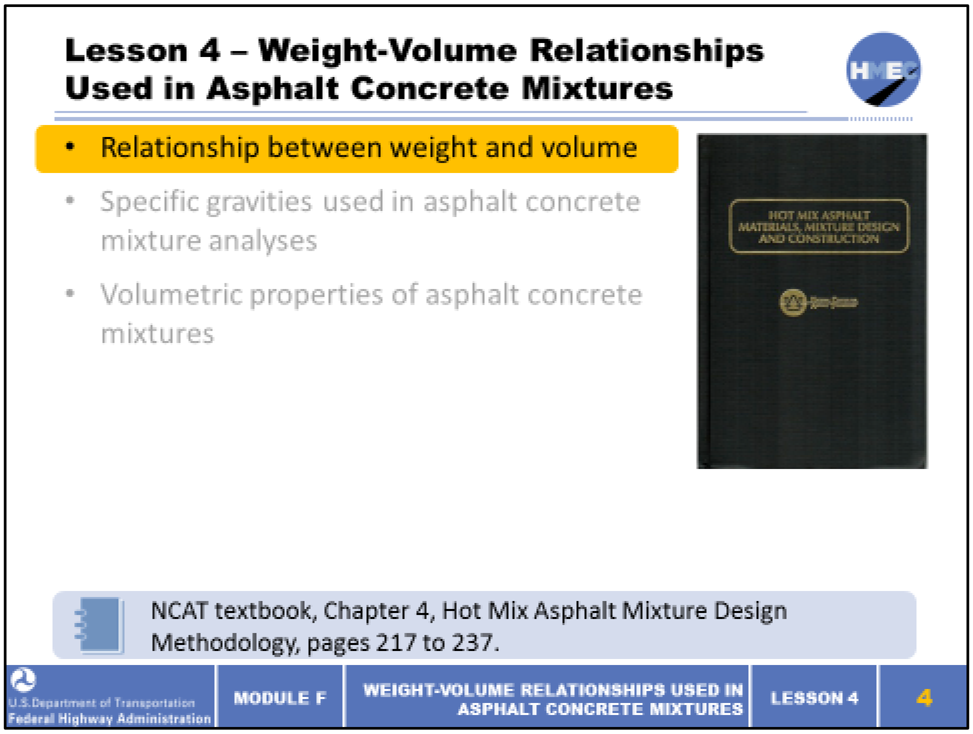
Slide 2


Learning Outcomes
By the end of this lesson, you will be able to:
• Understand the relationship between the weight and volume of an asphalt mixture
• Calculate volumetric properties
This lesson will take approximately 90 minutes to complete.
U.S. Department of Transportation
Federal Highway Administration
MODULE F
WEIGHT-VOLUME RELATIONSHIPS USED IN ASPHALT CONCRETE MIXTURES
LESSON 4
2

Slide 3


Lesson 4 – Weight-Volume Relationships Used in Asphalt Concrete Mixtures
HMEC
• Relationship between weight and volume
• Specific gravities used in asphalt concrete mixture analyses
• Volumetric properties of asphalt concrete mixtures
HOT MIX ASPHALT MATERIALS, MIXTURE DESIGN AND CONSTRUCTION
NCAT textbook, Chapter 4: Hot Mix Asphalt Mixture Design Methodology, pages 217 to 237.
U.S. Department of Transportation
Federal Highway Administration
MODULE F
WEIGHT-VOLUME RELATIONSHIPS USED IN ASPHALT CONCRETE MIXTURES
LESSON 4
3

Slide 4


Lesson 4 – Weight-Volume Relationships Used in Asphalt Concrete Mixtures
HMEC
• Relationship between weight and volume
• Specific gravities used in asphalt concrete mixture analyses
• Volumetric properties of asphalt concrete mixtures
HOT MIX ASPHALT MATERIALS, MIXTURE DESIGN AND CONSTRUCTION
NCAT textbook, Chapter 4, Hot Mix Asphalt Mixture Design Methodology, pages 217 to 237.
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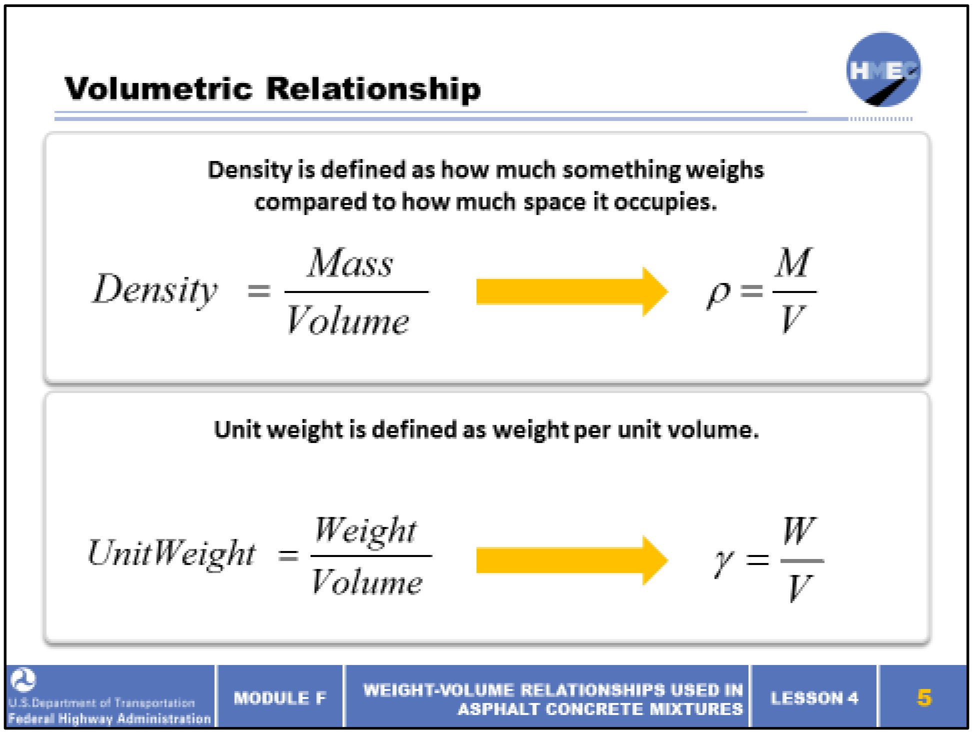
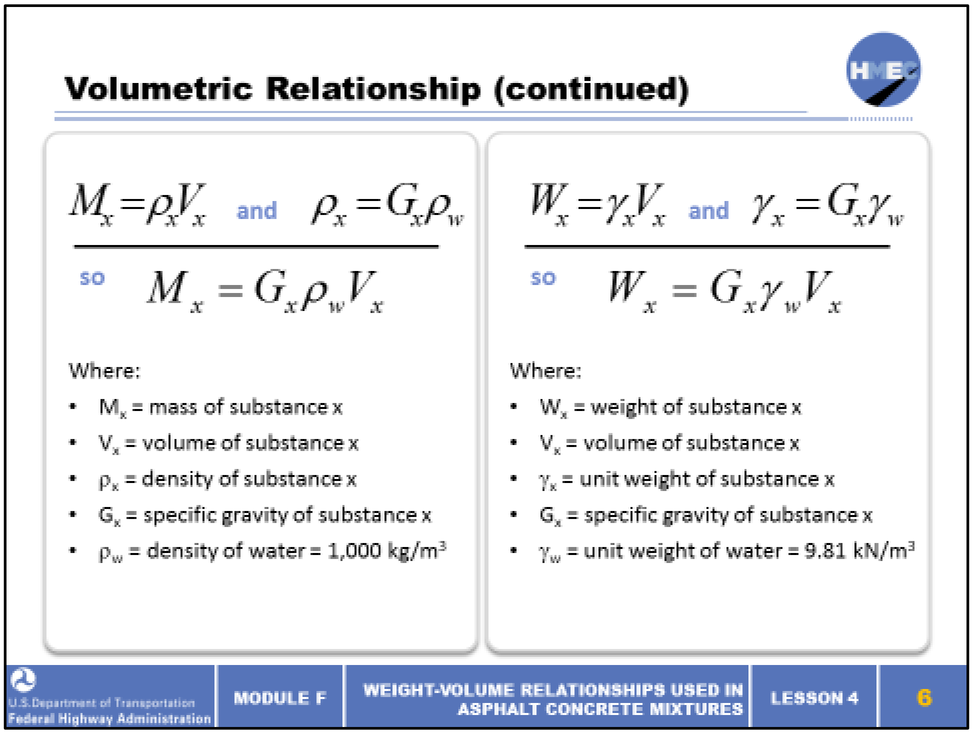
WEIGHT-VOLUME RELATIONSHIPS USED IN ASPHALT CONCRETE MIXTURES
LESSON 4
4

Slide 5



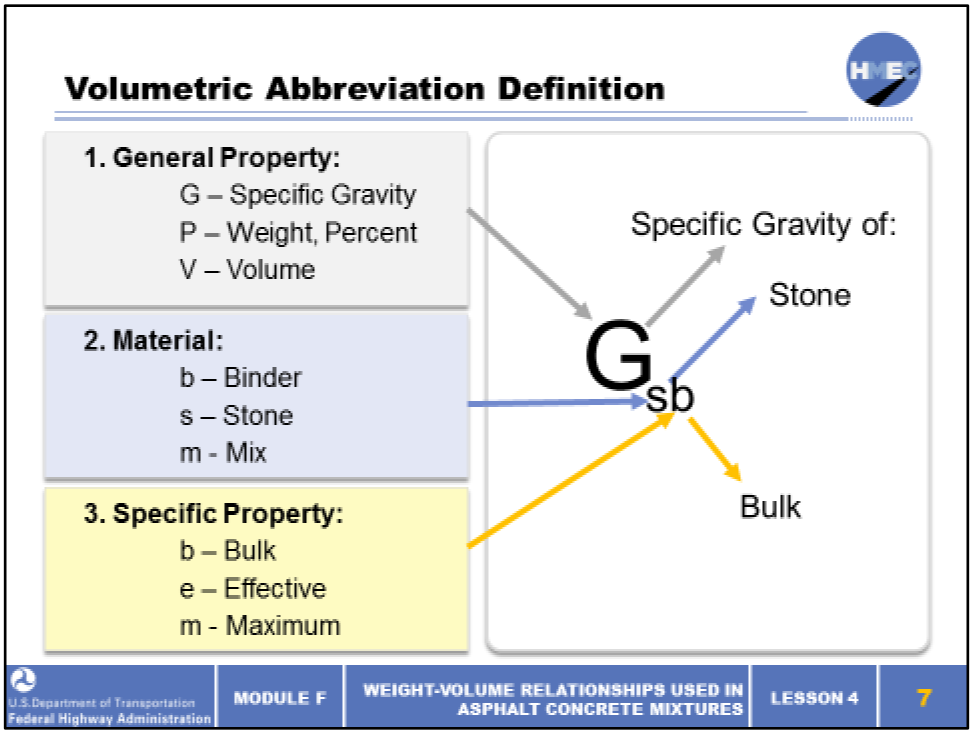
One of the fundamental engineering relationships used in weight-volume relationships is that of density. Density is defined as the mass per unit volume. Another fundamental engineering relationship used in weight-volume relationships is that of unit weight. Unit weight is defined as the weight (mass $\times$ gravity) per unit volume.

Slide 6



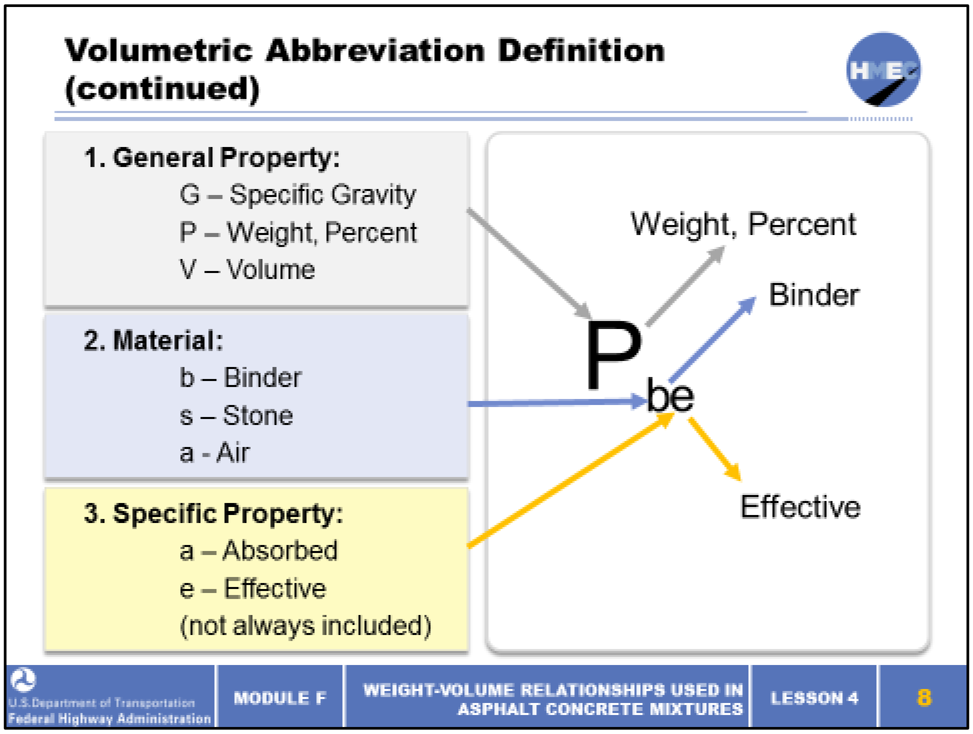
Similarly, the volume of the substance can be determined if its mass and specific gravity are known.

Slide 7



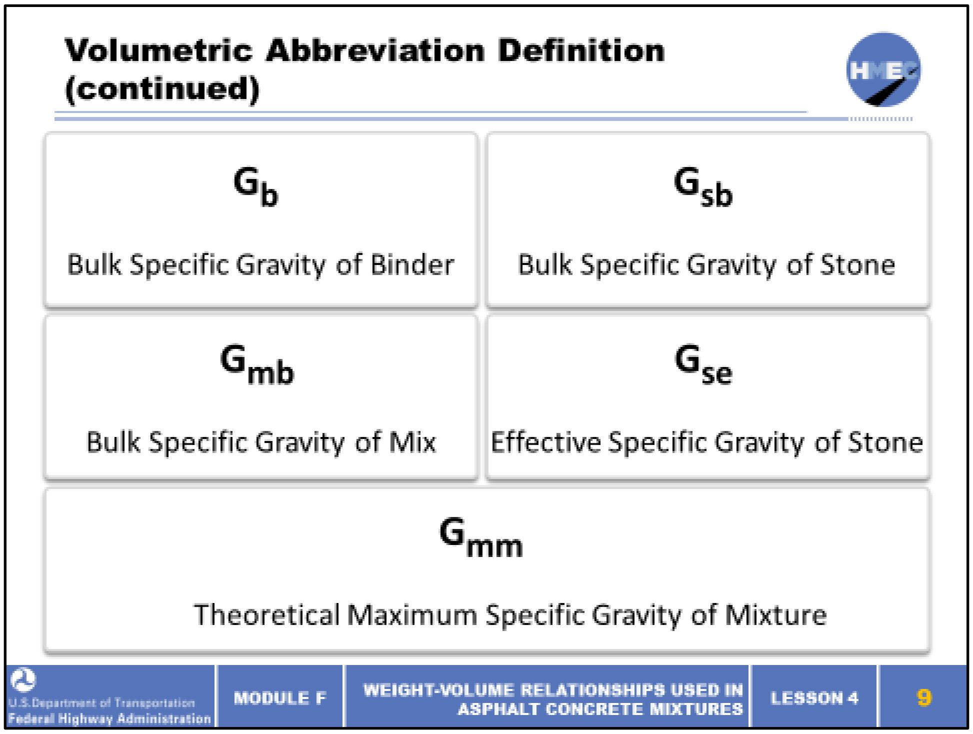
Just about all weight-volumetric terms are defined using three parameters, as shown here. The general property is the capital letter and is used to designate the property. The first lowercase letter defines or designates the material type, while the second lowercase letter defines the specific type of property. The example is for G, specific gravity of the stone (or aggregate), s, and its bulk specific gravity, or b.

Slide 8



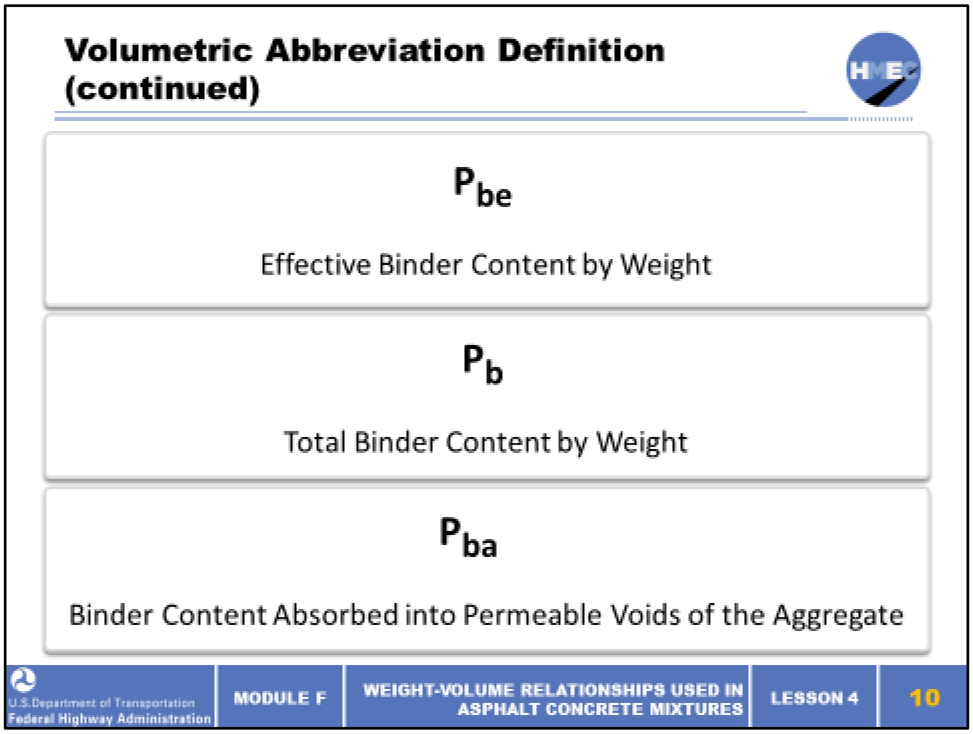
The example is for the general property of weight, P, in percent. The first lowercase letter is the material type and in this case it is for the binder, or b. The second lowercase letter is for the specific property, and in this case, it is the absorbed binder, a.

Slide 9



These abbreviations are used and referred to in most of the literature and reports.

Slide 10


Volumetric Abbreviation Definition (continued)
HMEC
P_{be}
Effective Binder Content by Weight
P_b
Total Binder Content by Weight
P_{ba}
Binder Content Absorbed into Permeable Voids of the Aggregate
U.S. Department of Transportation
Federal Highway Administration
MODULE F
WEIGHT-VOLUME RELATIONSHIPS USED IN ASPHALT CONCRETE MIXTURES
LESSON 4
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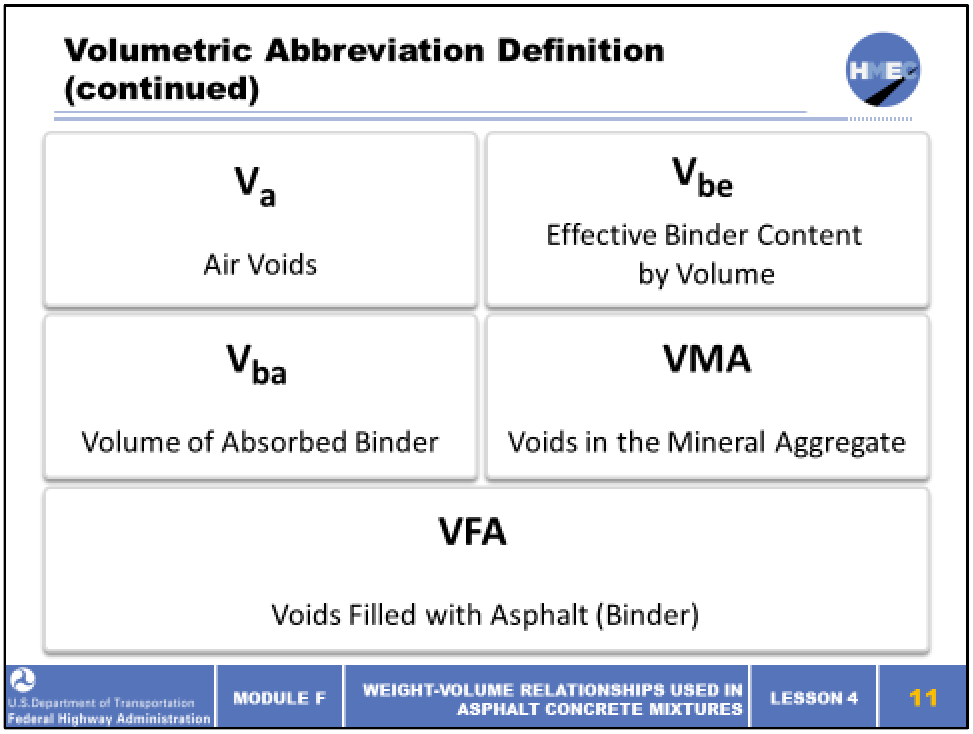
Slide 11



The voids in the mineral aggregate (VMA) and voids filled with asphalt (VFA) are two additional terms typically used in all textbooks and mixture design manuals. These will be defined and explained in the next screen and represent basic volumetric parameters that are related to how the mixture will perform.

Slide 12

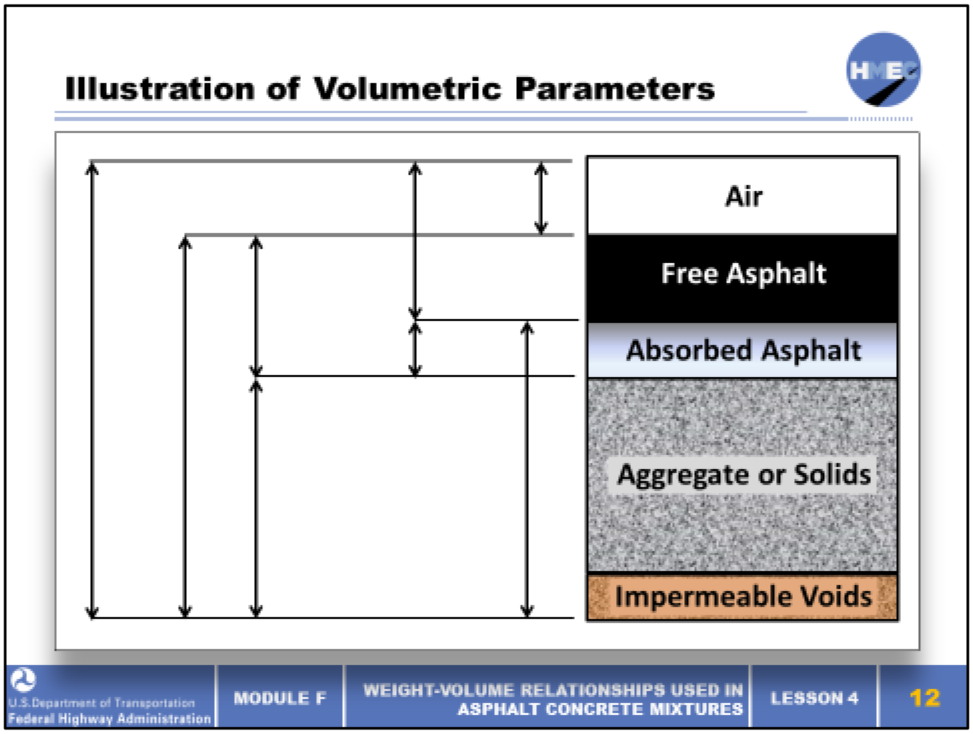


Illustration of Volumetric Parameters
HMEC
Air
Free Asphalt
Absorbed Asphalt
Aggregate or Solids
Impermeable Voids
U.S. Department of Transportation
Federal Highway Administration
MODULE F
WEIGHT-VOLUME RELATIONSHIPS USED IN ASPHALT CONCRETE MIXTURES
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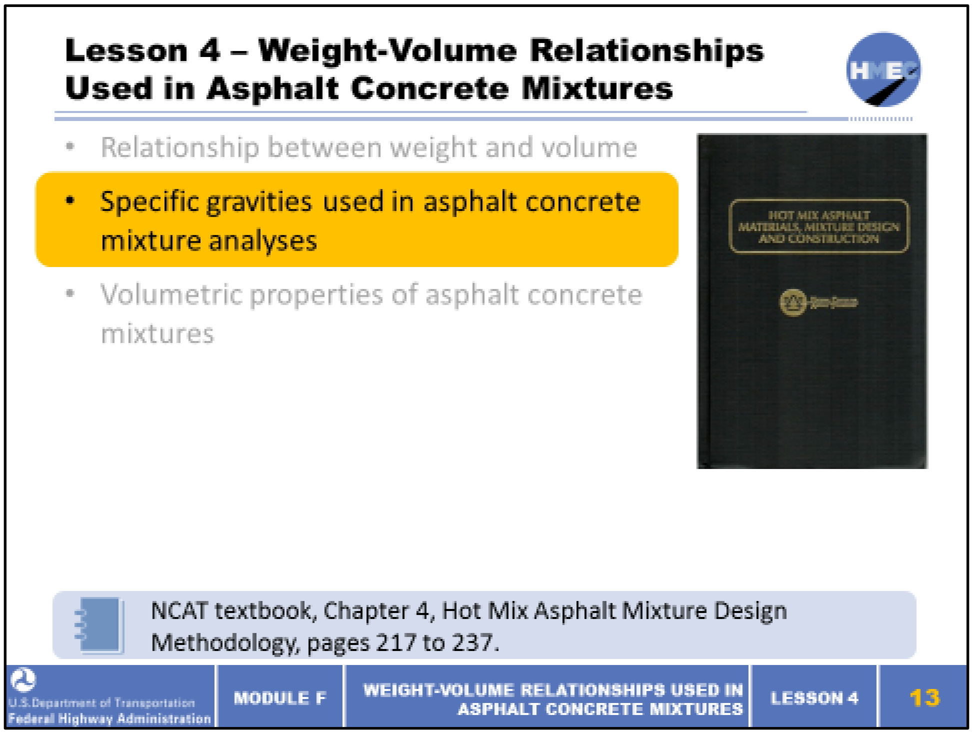
Slide 13


Lesson 4 – Weight-Volume Relationships Used in Asphalt Concrete Mixtures
HMEC
• Relationship between weight and volume
• Specific gravities used in asphalt concrete mixture analyses
• Volumetric properties of asphalt concrete mixtures
HOT MIX ASPHALT MATERIALS, MIXTURE DESIGN AND CONSTRUCTION
NCAT textbook, Chapter 4, Hot Mix Asphalt Mixture Design Methodology, pages 217 to 237.
U.S. Department of Transportation Federal Highway Administration
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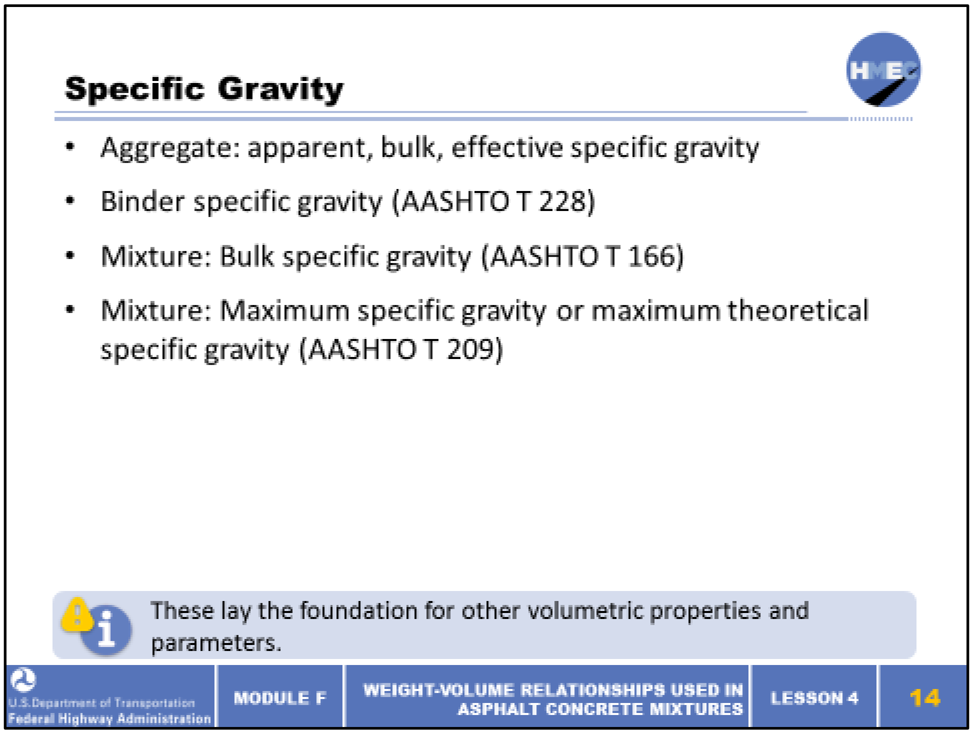
Slide 14


Specific Gravity
Aggregate: apparent, bulk, effective specific gravity
Binder specific gravity (AASHTO T 228)
Mixture: Bulk specific gravity (AASHTO T 166)
Mixture: Maximum specific gravity or maximum theoretical specific gravity (AASHTO T 209)
These lay the foundation for other volumetric properties and parameters.
U.S. Department of Transportation
Federal Highway Administration
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WEIGHT-VOLUME RELATIONSHIPS USED IN ASPHALT CONCRETE MIXTURES
LESSON 4
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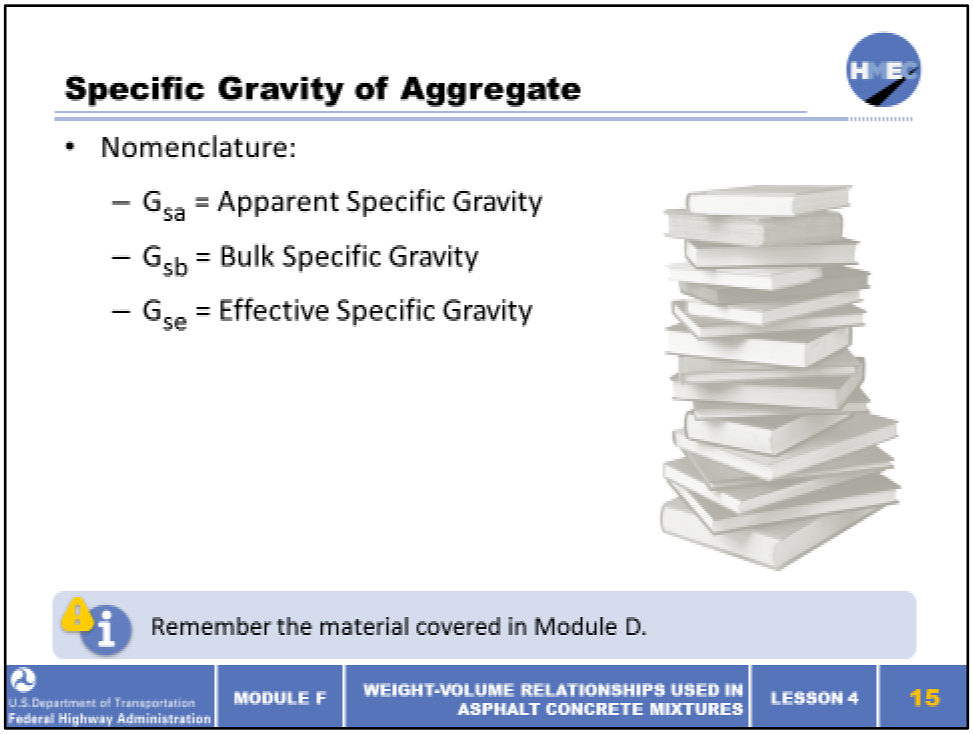
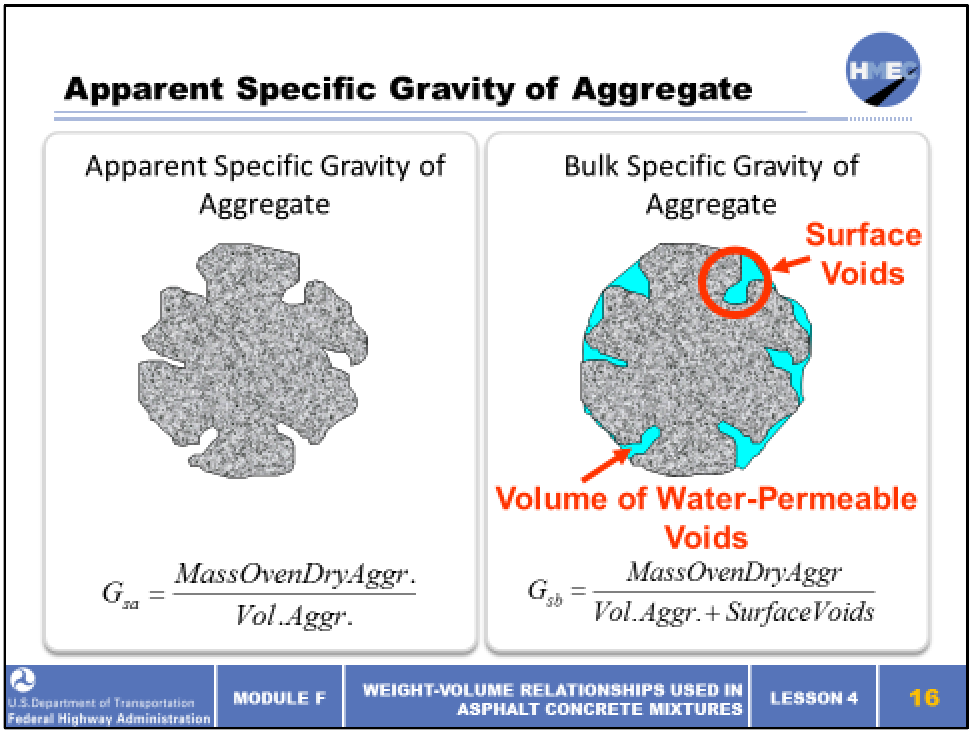
Slide 15



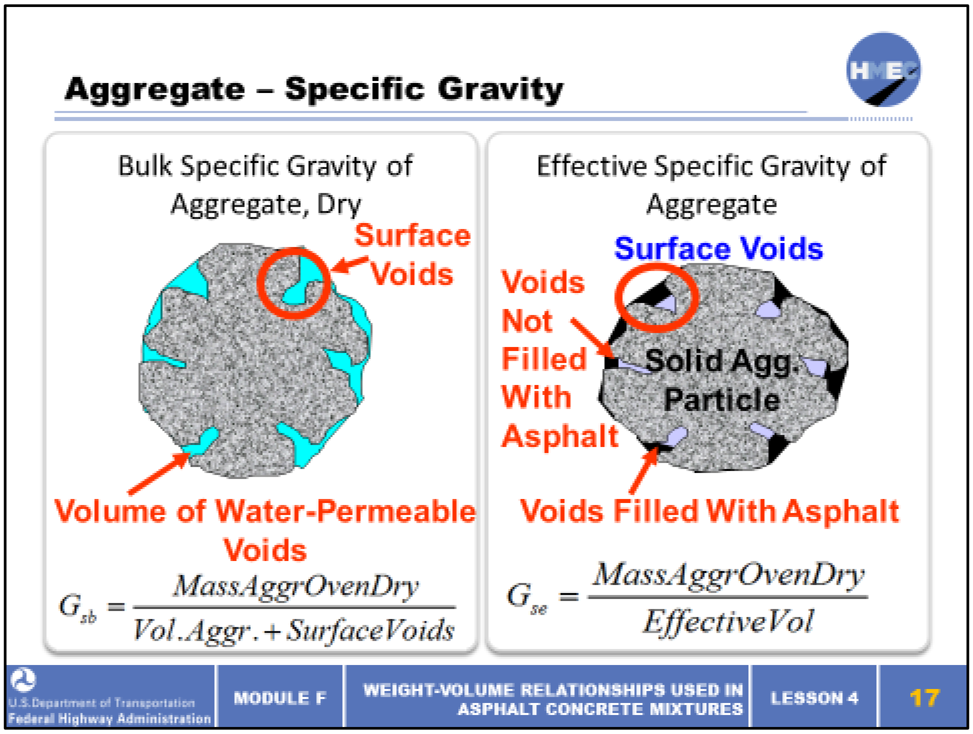
Module D covered the specific gravity of the aggregate, so this slide is a reminder and knowledge check on the material that relates to asphalt concrete mixtures for volumetric analysis.

Slide 16



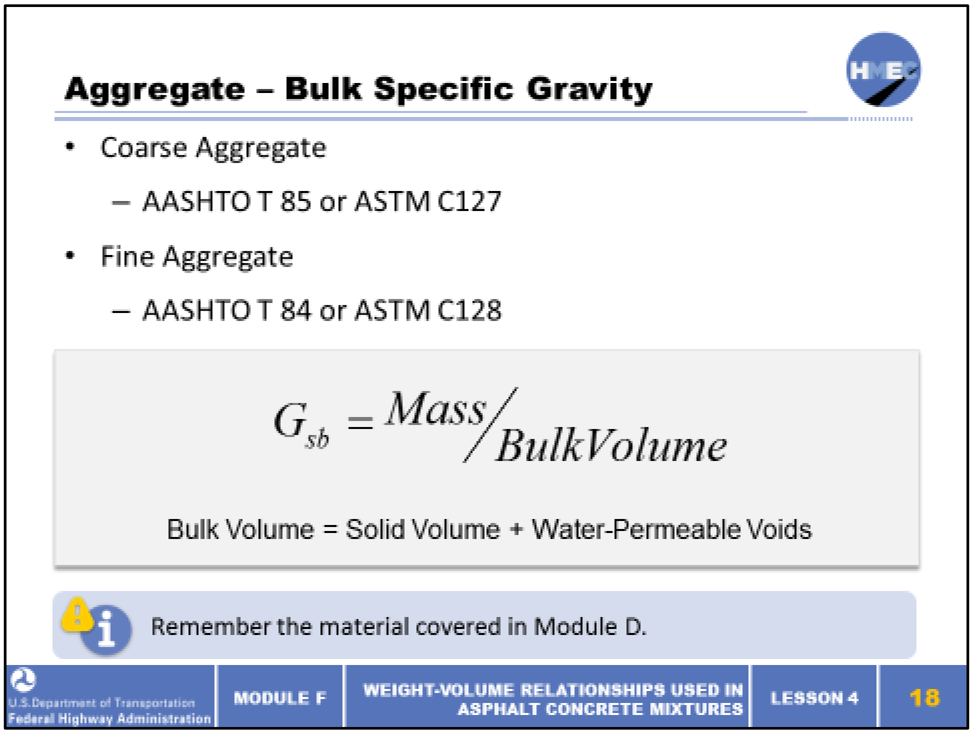
The illustration shows the mass and volume used to determine the apparent specific gravity in comparison to the bulk specific gravity of the aggregate.

Slide 17



The illustration shows the mass and volume used to determine the effective specific gravity in comparison to the bulk specific gravity of the aggregate.

Slide 18



Both of these test procedures were explained and described in Module D. This is just a reminder for use in the asphalt concrete weight-volume relationships.

Slide 19

Aggregate – Bulk Specific Gravity

- Combining Aggregates with Different Specific Gravities

$$G_{sb} = \frac{P_1 + P_2 + \ldots + P_n}{\frac{P_1}{G_1} + \frac{P_2}{G_2} + \ldots + \frac{P_n}{G_n}}$$

  - G_{sb} = bulk specific gravity for the total aggregate
  - P_i = percentages by weight of aggregates 1, 2, n
  - G_i = bulk specific gravities of aggregates 1, 2, n

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This equation can be used to calculate the bulk specific gravity of different combinations of multiple aggregates resulting in different aggregate blends.

Slide 20

Combining Aggregates with Different Specific Gravities

Aggregate Type	Bulk Specific Gravity	Percentages in Aggregate Blend
Crushed Trap Rock	2.560	23
Crushed Limestone	2.434	62
Crushed Fines	2.727	15
Bulk Specific Gravity of Combined Blend	?	100

$$G_{sb} = \frac{P_1 + P_2 + \ldots + P_n}{\dfrac{P_1}{G_1} + \dfrac{P_2}{G_2} + \ldots + \dfrac{P_n}{G_n}}$$

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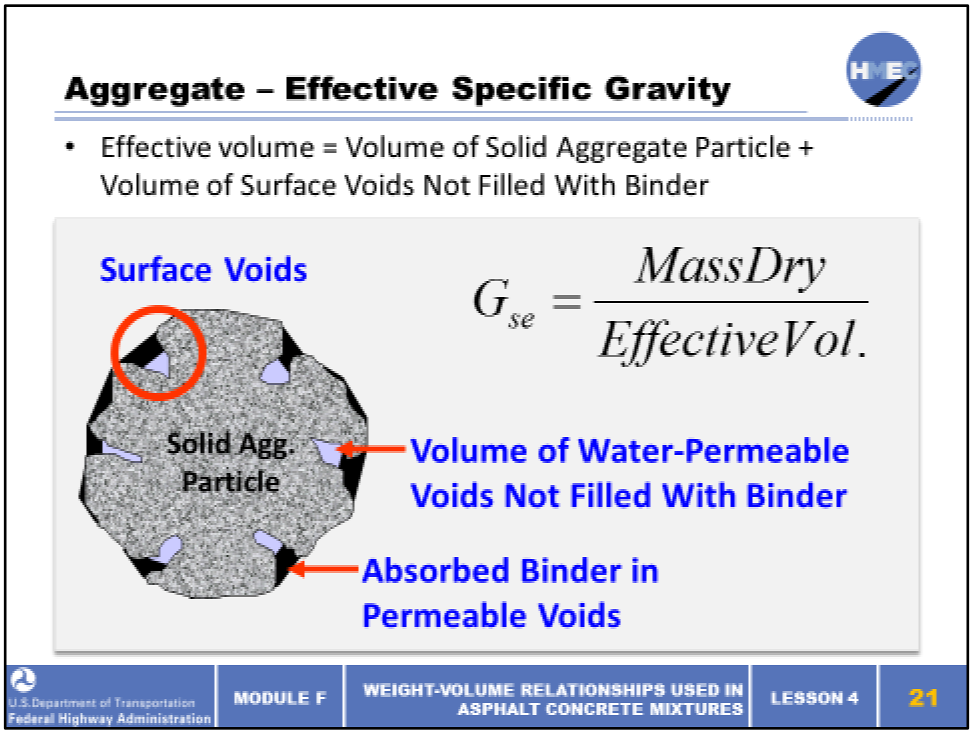
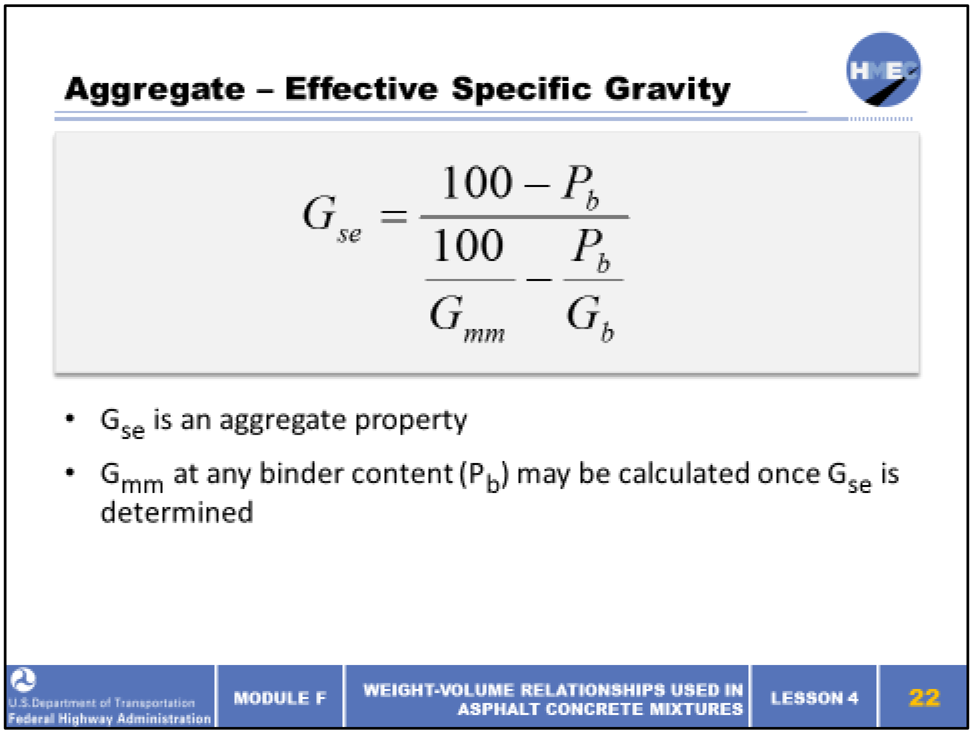
This is used in preparing preliminary mixture designs for different combinations of aggregates to produce an economical mixture that will satisfy the mixture design requirements and material specifications.

Slide 21



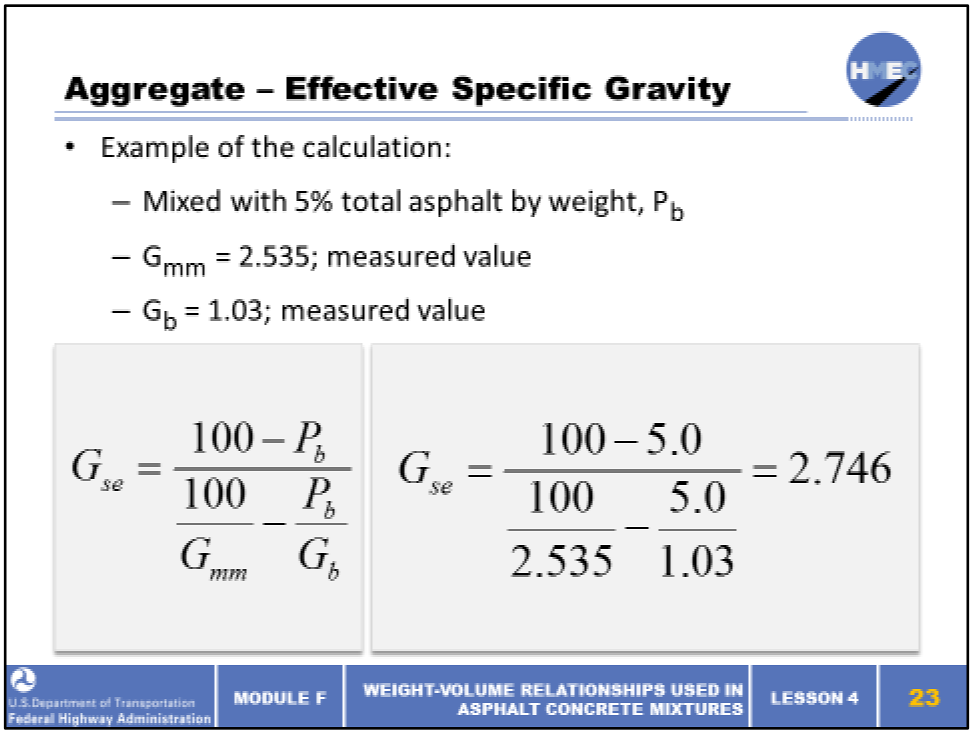
The effective specific gravity of the aggregate was defined earlier in this lesson. It is determined after the aggregate is coated with asphalt or binder. The mass is the dry mass of the aggregate without binder, but the volume is the volume of the dry particle plus only the surface aggregate voids not filled with binder.

Slide 22



The effective specific gravity of the aggregate is calculated using the maximum specific gravity of the mixture, which is measured, the percent of binder (by mass) in the sample used to determine the maximum specific gravity, and the specific gravity of the binder, which is also measured.

Slide 23


Aggregate – Effective Specific Gravity
HMEC
• Example of the calculation:
– Mixed with 5% total asphalt by weight, P_b
– G_{mm} = 2.535; measured value
– G_b = 1.03; measured value
$$G_{se} = \frac{100 - P_b}{\frac{100}{G_{mm}} - \frac{P_b}{G_b}}$$
$$G_{se} = \frac{100 - 5.0}{\frac{100}{2.535} - \frac{5.0}{1.03}} = 2.746$$
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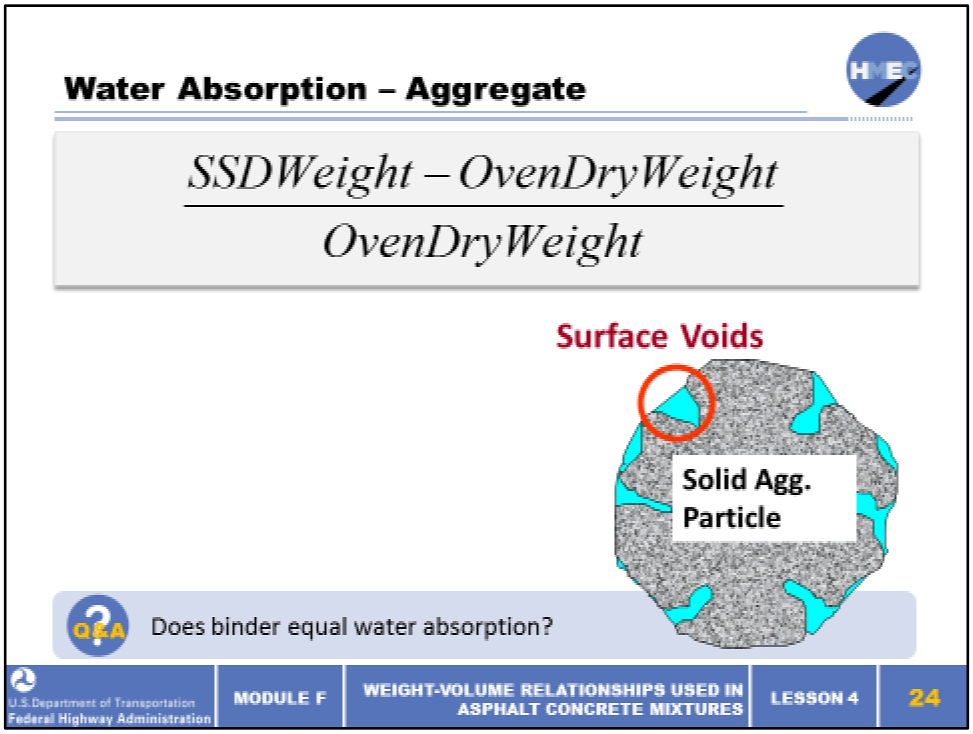
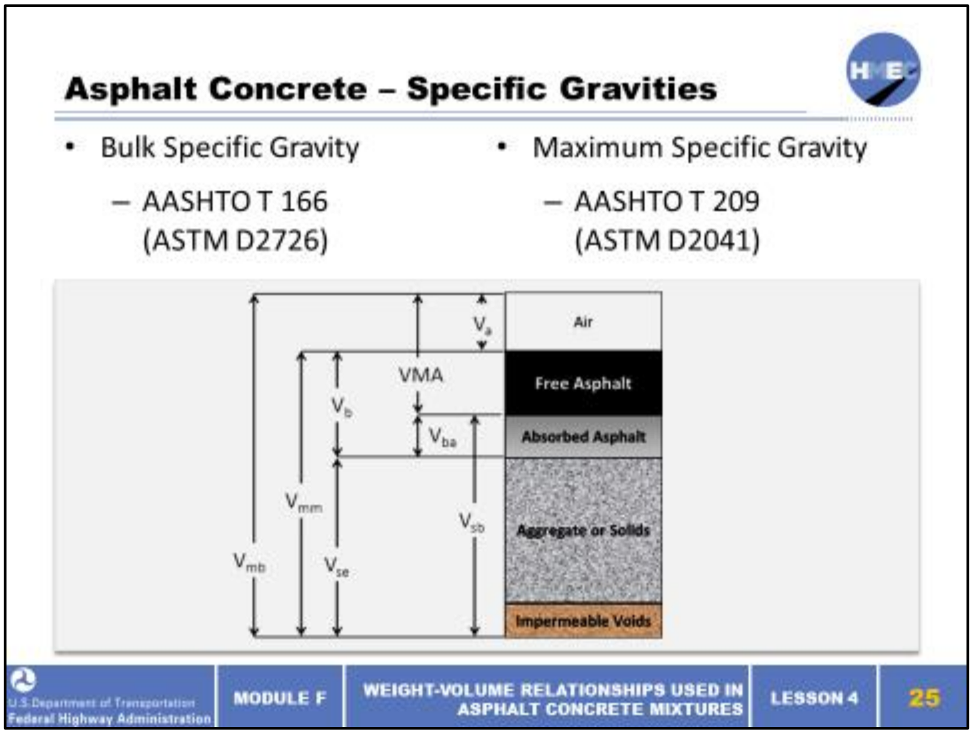
Slide 24



The aggregate is considered "dry" when it has been maintained at a temperature of 110 $\pm$ 5 °C for a sufficient enough time to remove all water in the permeable voids. Generally this is needed when doing concrete mix design. But, it can be used to estimate the amount of binder absorption for a mix. The ratio of asphalt to moisture absorption will vary with different aggregates and different asphalt or binders. But, it will provide a mix designer with a general estimate of the amount of binder absorption.

Slide 25

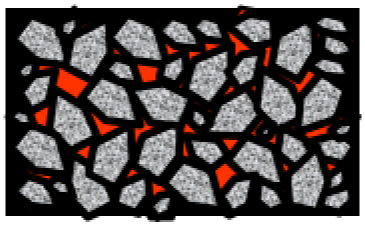


Both are illustrated and defined on the next couple of screens.

Slide 26

Asphalt Concrete – Bulk Specific Gravity

$$G_{mb} = \frac{Mass\ of\ Aggr + AC}{Volume\ of\ Aggr + AC + Voids}$$



Binder mixed with aggregate and compacted into sample

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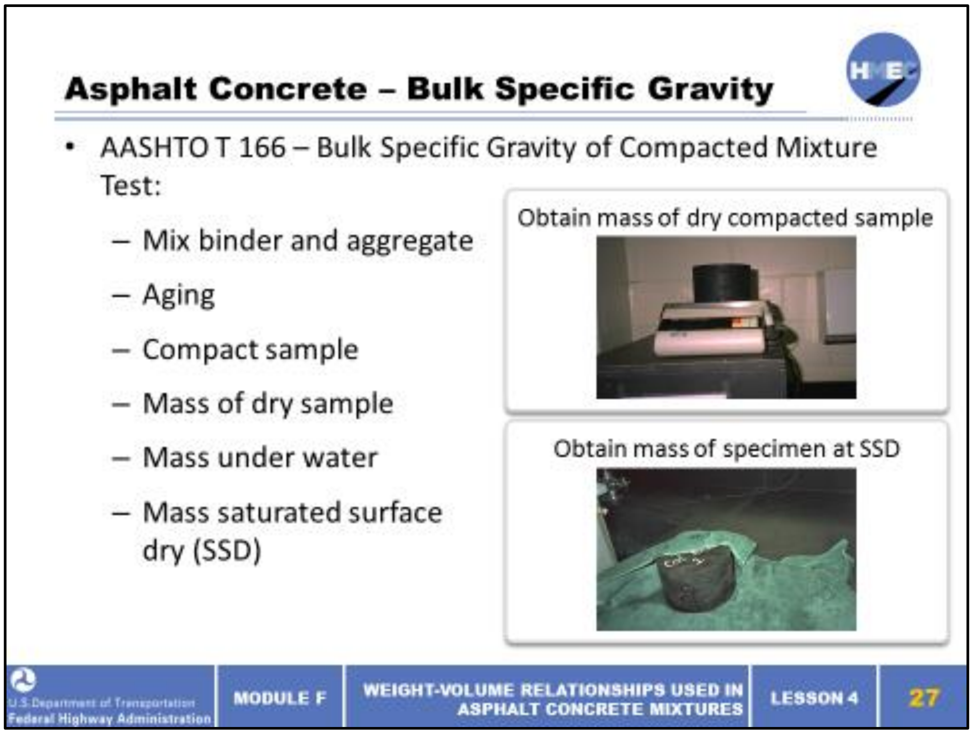
Bulk specific gravity is the mass of aggregate and binder divided by the bulk volume (i.e., volume of aggregate, binder, and air). In other words, it is the ratio of the weight in air of a unit volume of a compacted specimen of asphalt concrete (including the permeable voids) at a state temperature to the weight of an equal volume of gas-free distilled water at a stated temperature. The equation for calculating the bulk specific gravity of a compacted asphalt concrete specimen is shown on the screen.

Slide 27


Asphalt Concrete – Bulk Specific Gravity
• AASHTO T 166 – Bulk Specific Gravity of Compacted Mixture Test:
– Mix binder and aggregate
– Aging
– Compact sample
– Mass of dry sample
– Mass under water
– Mass saturated surface dry (SSD)
Obtain mass of dry compacted sample
Obtain mass of specimen at SSD
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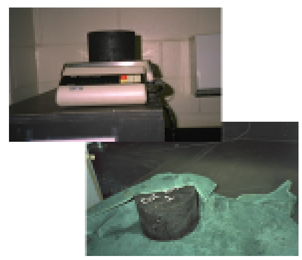
Slide 28

Asphalt Concrete – Bulk Specific Gravity

$$G_{mb} = \frac{A}{B - C}$$

- Where:
  - A = mass of dry sample
  - B = mass of SSD sample
  - C = mass of sample under water



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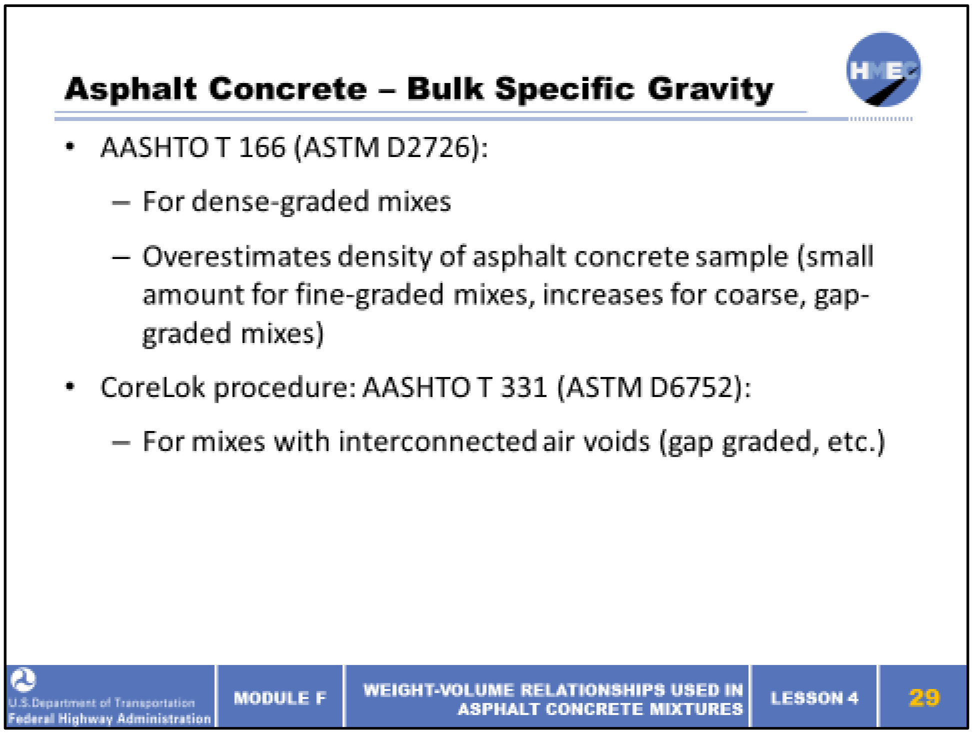
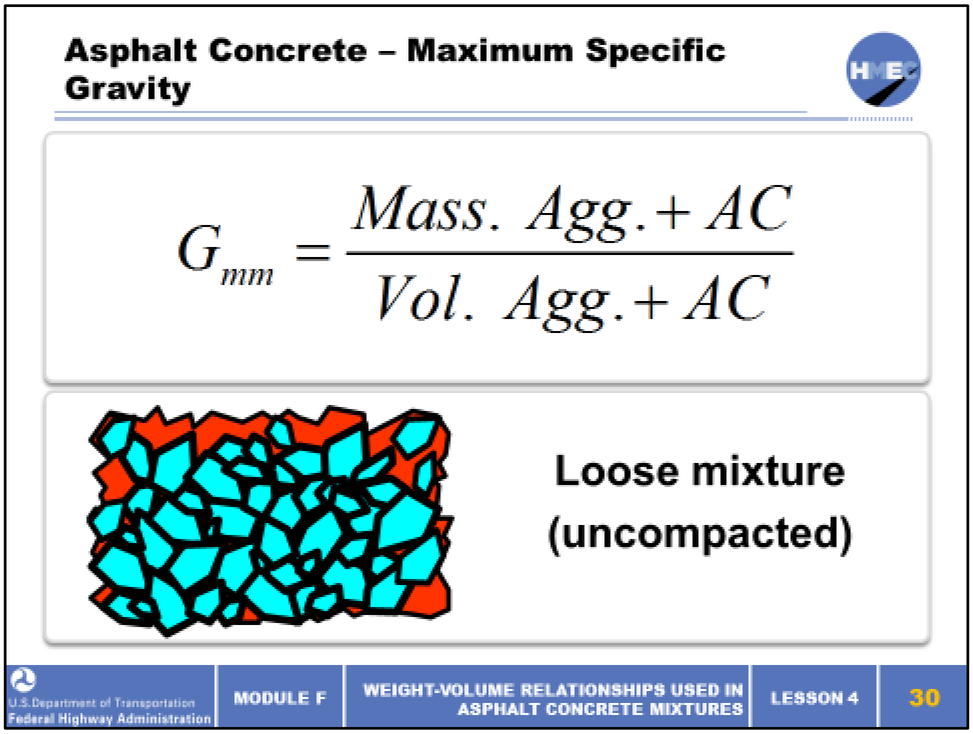


The density or specific gravity is over-estimated for mixtures with higher air voids or coarse-graded and gap-graded aggregate blends. Increase in the use of coarse- and open-graded mixes created a need for a more reliable and accurate method of bulk specific gravity measurement of laboratory and field specimens. Open-graded mixes readily absorb water and drain quickly when removed from the water tank. The lack of control over the penetration and drainage of water in and out of asphalt samples creates a fundamental problem with the water displacement measurement using the current principles for determination of specific gravity. For these conditions, the CoreLok procedure is used to measure the bulk specific gravity, which is AASHTO T 331.

The CoreLok system is a vacuum chamber that is used with specially designed polymer bags to completely seal field and laboratory asphalt samples from water during the bulk specific gravity measurements. The CoreLok system can be used for determination of the bulk specific gravity of compacted asphalt samples (ASTM D6752 and AASHTO T 331).

Prior to the CoreLok system, the procedure was to seal the samples prior to testing in water. The sealing methods included paraffin and parafilm.

Slide 30



The test is conducted on a loose (uncompacted) mixture. The maximum specific gravity of the mixture is defined as the ratio of the weight in air of a unit volume of an uncompacted bituminous paving mixture at a stated temperature to the weight of an equal volume of gas-free distilled water at a stated temperature.

It is also called the Rice specific gravity, which is the name of the individual (James Rice) that developed the test or theoretical maximum specific gravity.

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Asphalt Concrete – Maximum Specific Gravity
HMEC
• AASHTO T 209 – Maximum Theoretical Specific Gravity, G_{mm}
• Mix binder and aggregate
• Aging
• Mass in air
• Mass of pycnometer and water
• Mass of pycnometer, water, and loose mix (after vacuum)
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Maximum Specific Gravity Testing (continued)
Loose Mix at Room Temperature
U.S. Department of Transportation
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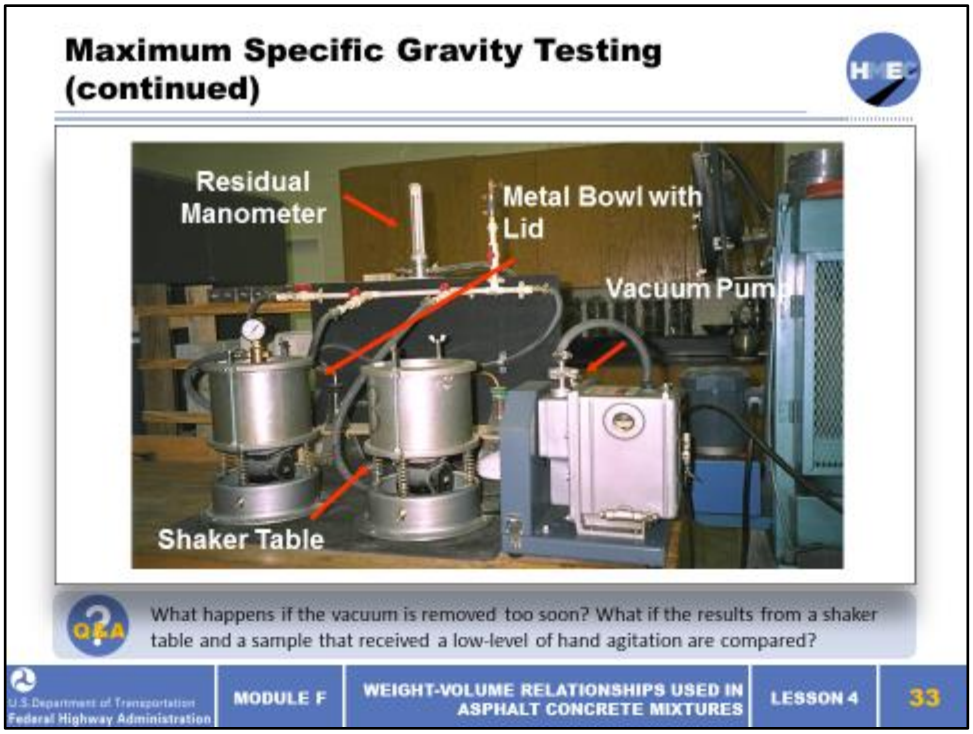
Slide 33



A predetermined mass of the dry loose mix is placed in a metal bowl and covered with water.

A vacuum lid is fitted and secured to the bowl and placed on a vibratory shaker table.

A vacuum pump is started and the manometer reading is used to determine the proper vacuum adjustment.

Once the proper partial vacuum is obtained, the shaker table is started. This provides gentle agitation to help in the removal of any air between particles.

This is continued for 5 to 15 minutes. This effort is to ensure that the air in the mixture is as close as possible to zero.

However, AASHTO T 209 has a lot of options. The shaker table is the preferred option, but the procedure also allows for hand agitation. Most States specify the exact method the test is to be run: mechanical agitation versus hand agitation, pycnometer versus flask, etc. Specifying the options to be used, States have been able to reduce the variability in the test outcome, as compared to the precision statement in T 209. There is an insignificant difference in the precision between hand versus mechanical agitation, but some have reported hand agitation is not as repeatable as the shaker table.

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HMA – Maximum Specific Gravity, G_{mm}

$$G_{mm} = \frac{A}{A-C}$$

- Where:
  - A = mass of dry sample
  - C = mass of sample under water

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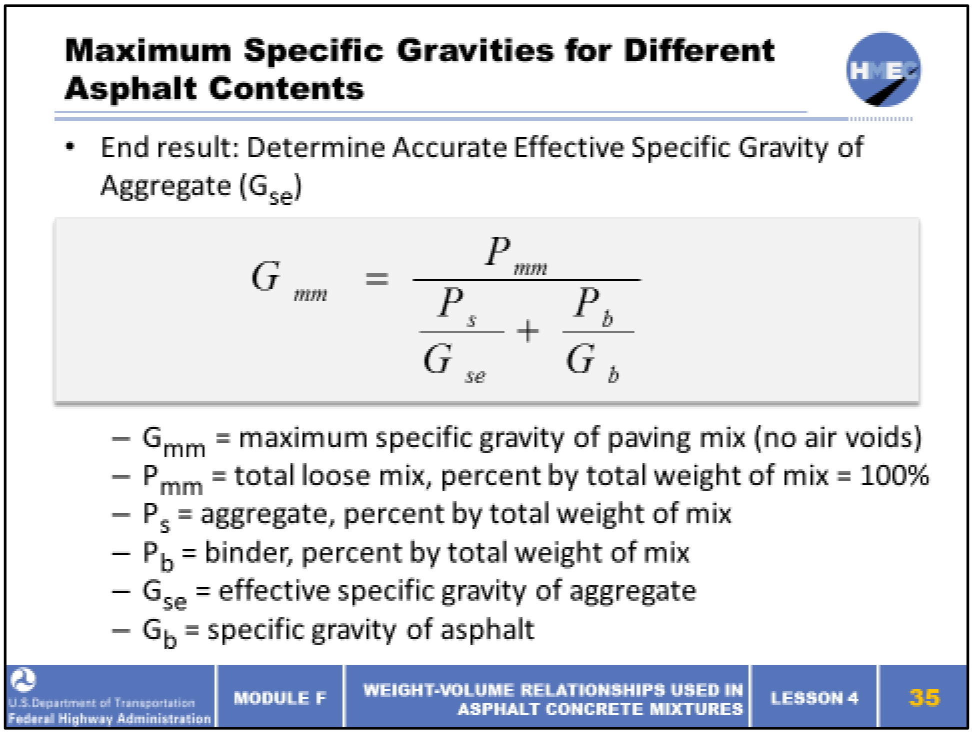
It is the mass of the coated aggregate divided by the volume of coated aggregate.

Slide 35



As noted earlier in this lesson, the effective specific gravity is calculated from the maximum specific gravity of the asphalt concrete paving mixture, the specific gravity of the binder, and the mass of the binder by weight by total weight of the mixture. Once the effective specific gravity of the aggregate is determined, the maximum specific gravity of the mixture can be calculated at slightly different binder contents, as shown on the screen. This equation is typically used during mixture design to calculate the maximum specific gravity over a range of binder contents.

Slide 36

Maximum Specific Gravities for Different Asphalt Contents

- Example of the calculation:
  - Mixed with 6% total asphalt by weight, P_b, so P_s is 94%
  - G_{se} = 2.746, calculated from 5% asphalt and the measured Gmm of 2.535
  - G_b = 1.03, measured value

$$G_{mm} = \frac{P_{mm}}{\frac{P_s}{G_{se}} + \frac{P_b}{G_b}}$$

$$G_{mm} = \frac{100}{\frac{94}{2.746} + \frac{6}{1.030}} = 2.496$$

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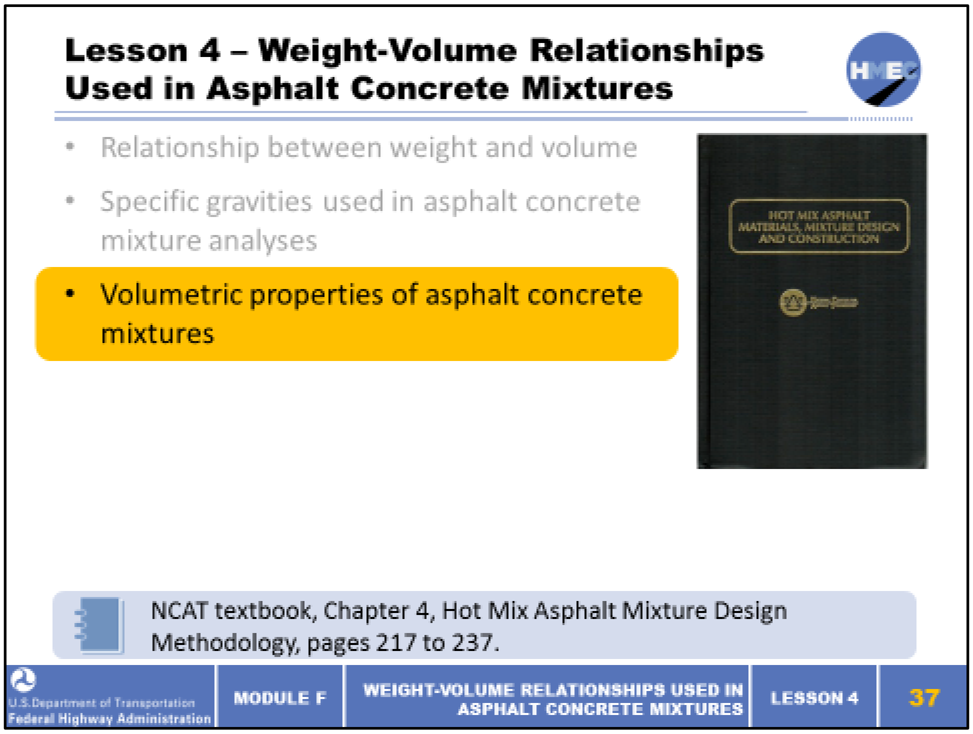
Lesson 4 – Weight-Volume Relationships Used in Asphalt Concrete Mixtures
HMEC
• Relationship between weight and volume
• Specific gravities used in asphalt concrete mixture analyses
• Volumetric properties of asphalt concrete mixtures
HOT MIX ASPHALT MATERIALS, MIXTURE DESIGN AND CONSTRUCTION
NCAT textbook, Chapter 4, Hot Mix Asphalt Mixture Design Methodology, pages 217 to 237.
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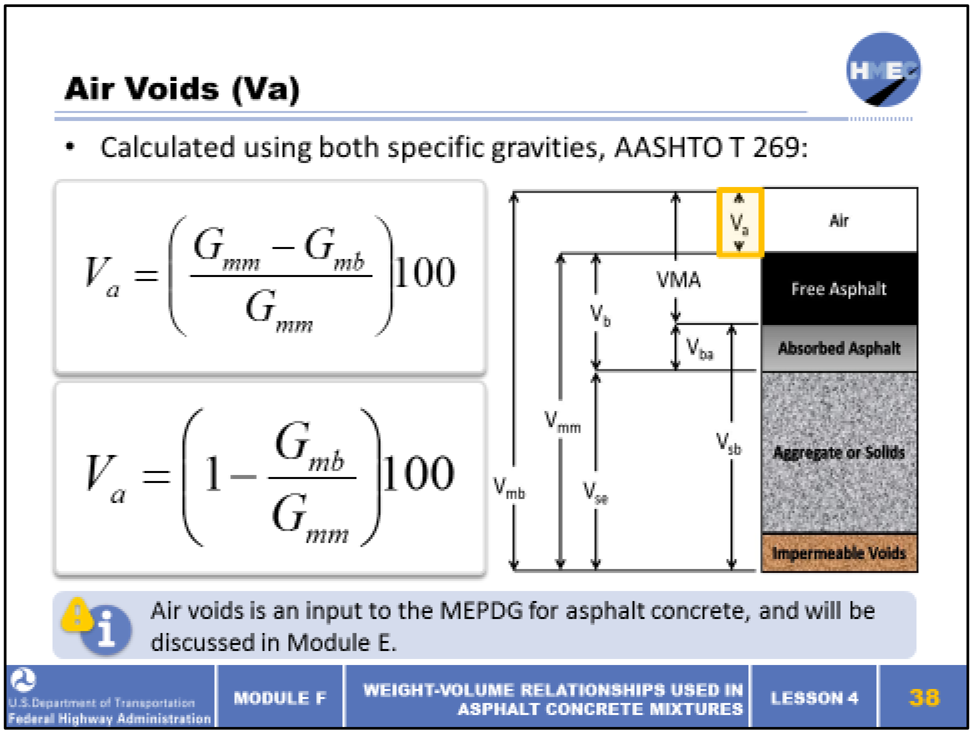
Air Voids (Va)
HMEC
• Calculated using both specific gravities, AASHTO T 269:
$V_a = \left(\frac{G_{mm} - G_{mb}}{G_{mm}}\right)100$
$V_a = \left(1 - \frac{G_{mb}}{G_{mm}}\right)100$
Air
Free Asphalt
Absorbed Asphalt
Aggregate or Solids
Impermeable Voids
V_a
VMA
V_b
V_{ba}
V_{mm}
V_{sb}
V_{mb}
V_{se}
Air voids is an input to the MEPDG for asphalt concrete, and will be discussed in Module E.
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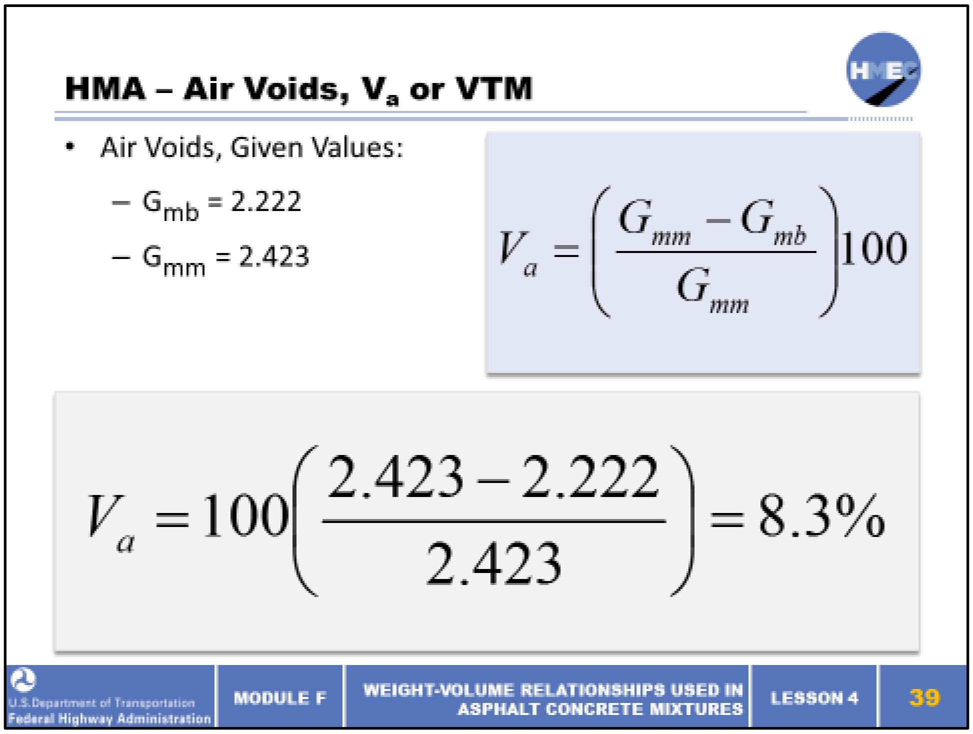
Slide 39


HMA – Air Voids, V_a or VTM
• Air Voids, Given Values:
– G_{mb} = 2.222
– G_{mm} = 2.423
$V_a = \left(\frac{G_{mm} - G_{mb}}{G_{mm}}\right)100$
$V_a = 100\left(\frac{2.423 - 2.222}{2.423}\right) = 8.3\%$
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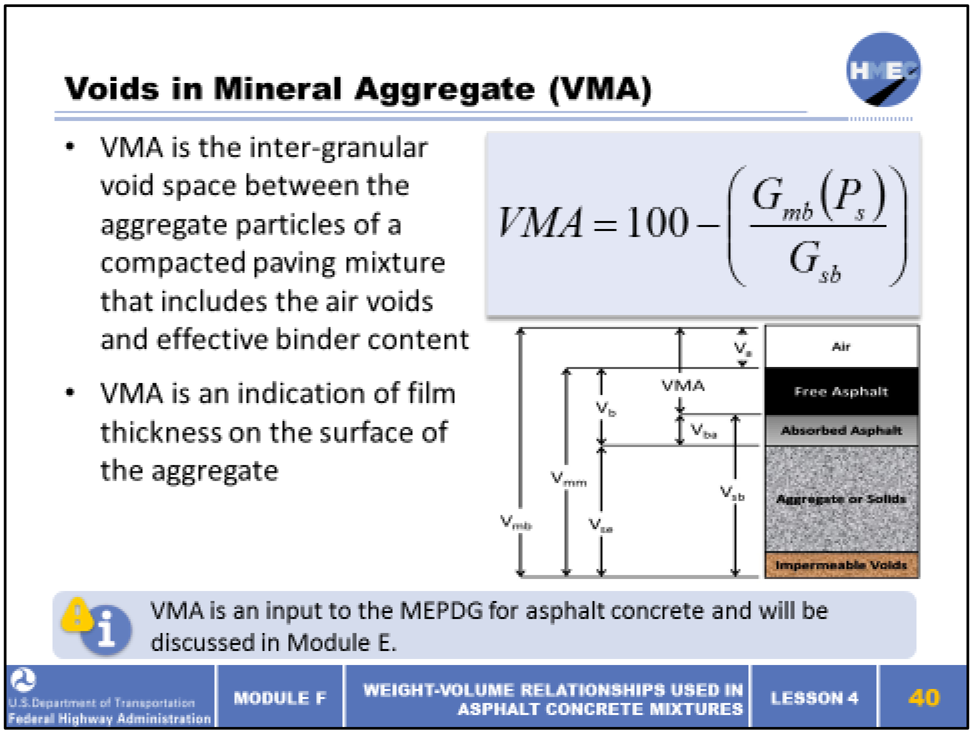
Slide 40



Because it is difficult and time-consuming to obtain the bulk specific gravity of the aggregate, a number of States are substituting the effective specific gravity of the aggregate. This value is simply a calculation using test results that have to be determined anyway.

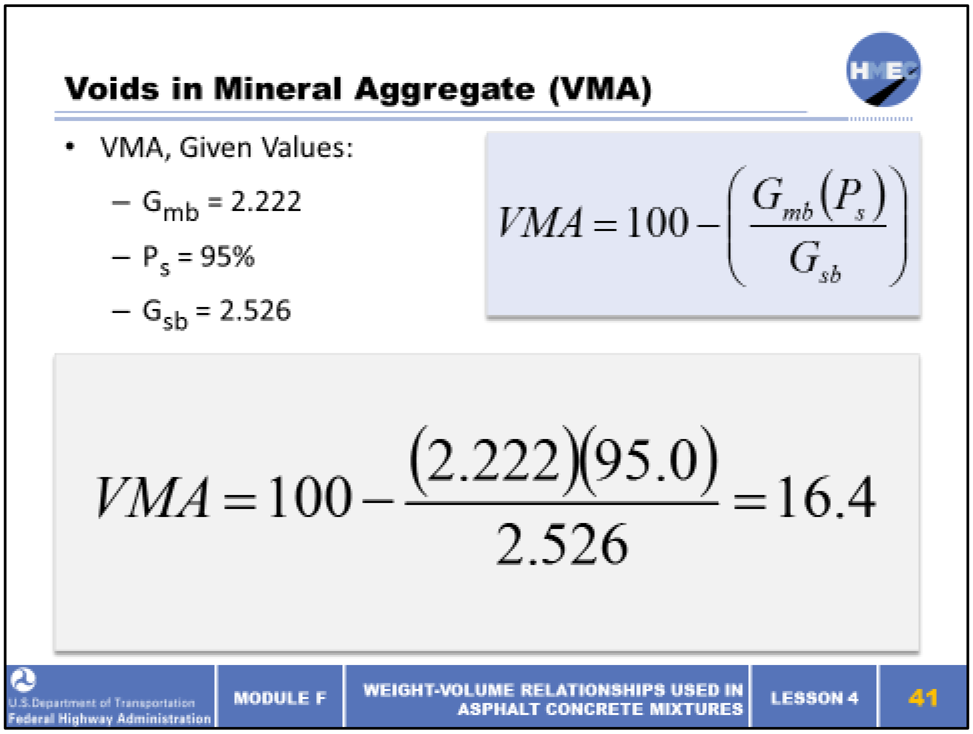
Altering the VMA calculation this way will result in an increase in the value. If this substitution is made, then either a correction factor is needed or the VMA requirements need to be adjusted for local aggregate properties.

Explain the equation and diagram shown on the screen.

Notes: Also note that the typical range of VMA for mixture design will be provided in Lesson 5.

“Free asphalt” is noted in the illustration, which is the “effective” binder. Thus, free = effective. The word effective was not used in the first illustration because it was not defined until in the discussion and presentation of VMA and in the latter slides on effective asphalt content.

Slide 41


Voids in Mineral Aggregate (VMA)
HMEC
• VMA, Given Values:
– G_{mb} = 2.222
– P_s = 95%
– G_{sb} = 2.526
$VMA = 100 - \left(\frac{G_{mb}(P_s)}{G_{sb}} \right)$
$VMA = 100 - \frac{(2.222)(95.0)}{2.526} = 16.4$
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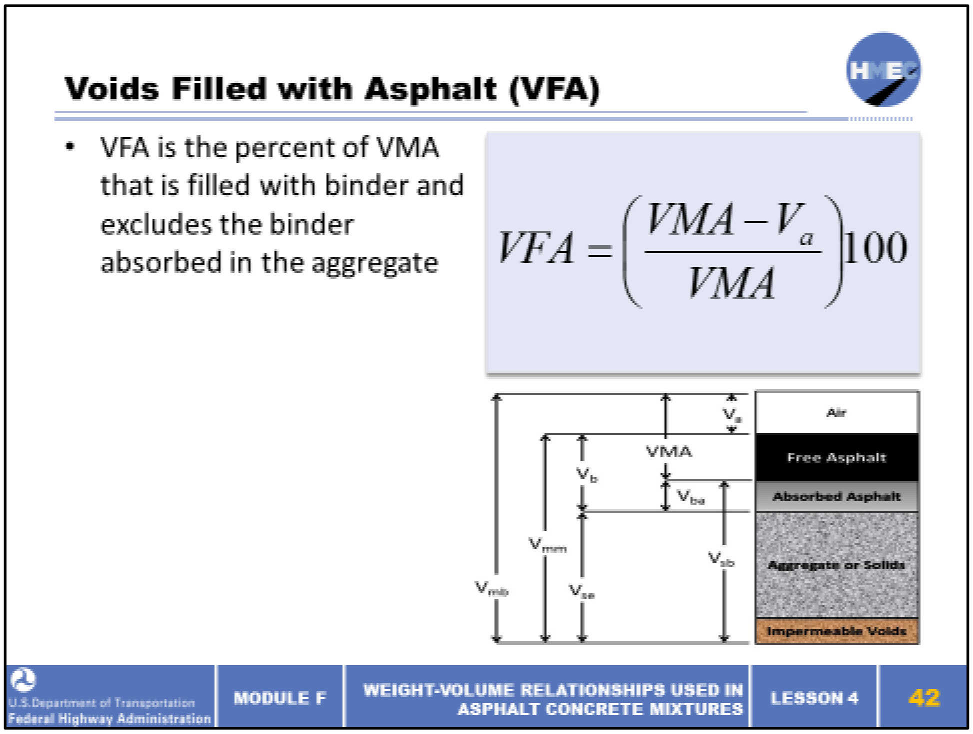
Slide 42


Voids Filled with Asphalt (VFA)
HMEC
• VFA is the percent of VMA that is filled with binder and excludes the binder absorbed in the aggregate
$$VFA = \left(\frac{VMA - V_a}{VMA}\right)100$$
Air
Free Asphalt
Absorbed Asphalt
Aggregate or Solids
Impermeable Voids
V_a
VMA
V_b
V_{ba}
V_{mm}
V_{sb}
V_{mb}
V_{se}
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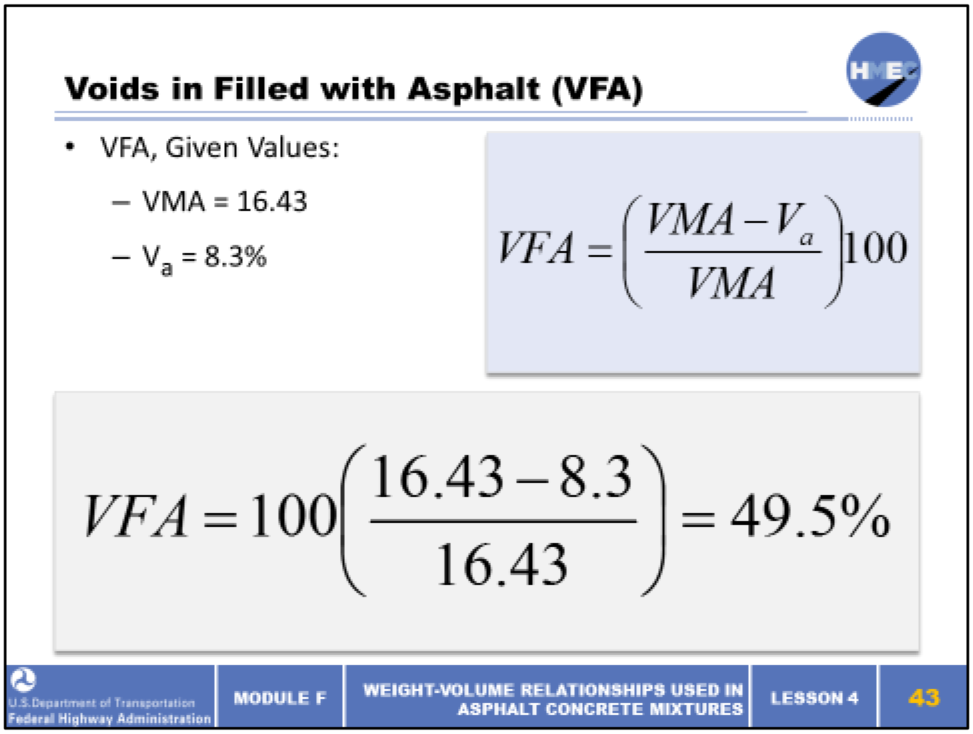
Slide 43


Voids in Filled with Asphalt (VFA)
VFA, Given Values:
– VMA = 16.43
– V_a = 8.3%
$VFA = \left(\frac{VMA - V_a}{VMA} \right) 100$
$VFA = 100 \left(\frac{16.43 - 8.3}{16.43} \right) = 49.5\%$
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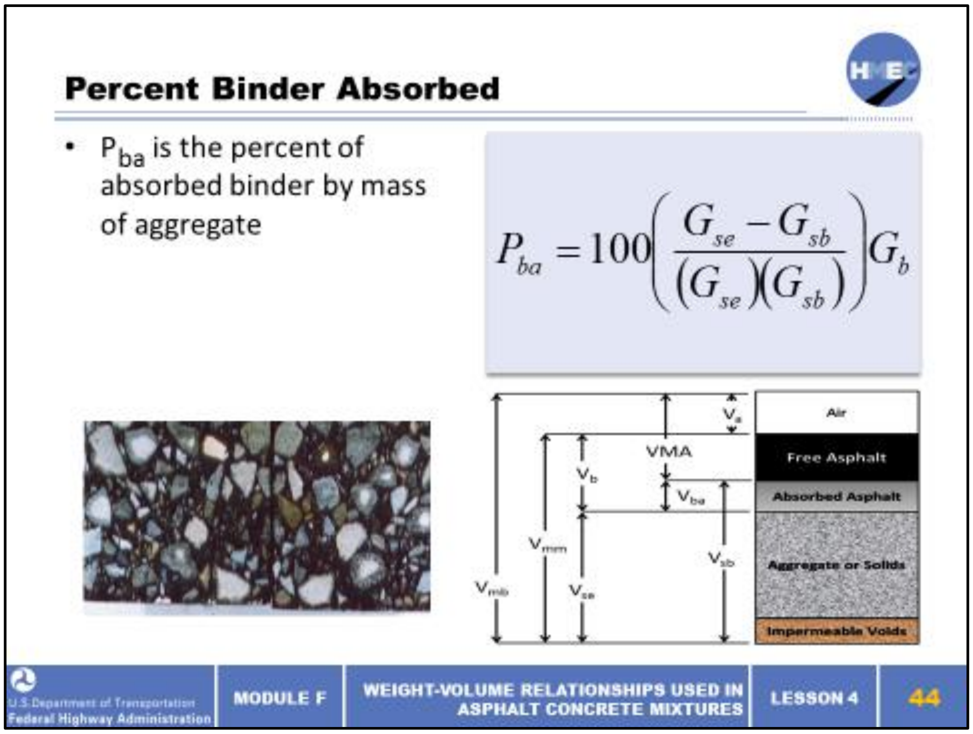
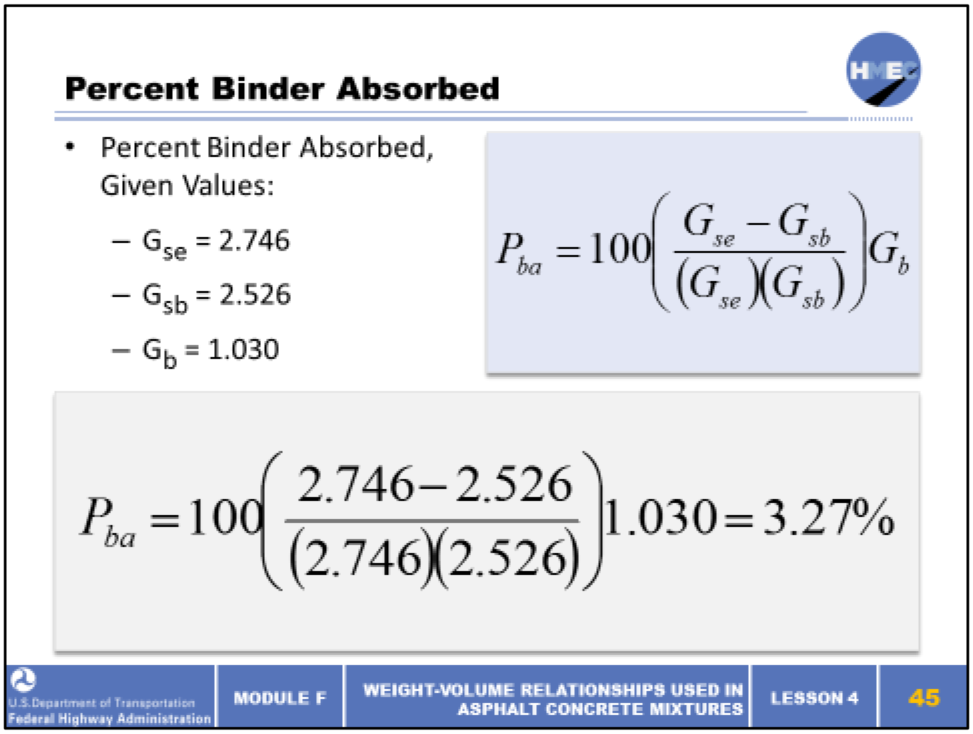
Slide 44



The volume of the absorbed binder is the difference between the bulk volume of the aggregate and the effective volume. The percent of binder absorbed by the aggregate is expressed as a percent of the mass of aggregate. The effective and bulk specific gravities of the aggregate and binder specific gravity are needed to calculate the absorbed binder by weight.

Some mix properties are defined or based on total weight of mix. However, the percent binder absorbed into the aggregate is defined or based on the weight of the aggregate. The reason for this is that as more and more total binder is added to the mix, similar to adding different increments of binder to the aggregate blend during the mixture design process, the percent binder absorbed would vary depending on how much total binder is added because the weight of the total mix is going to change. However, basing the percent binder absorbed of just the aggregate weight means that percent binder absorbed remains relatively constant because the amount of binder absorbed will be relatively constant with different amounts of total binder added. Also, the effective specific gravity of the aggregate blend is an aggregate property and should not change with varying amounts of binder. In addition, the bulk specific gravity of the stone and the specific gravity of the binder are all constant values in the equation. Thus, the percent binder absorbed will be a constant value for a specific aggregate blend.

Slide 45


Percent Binder Absorbed
• Percent Binder Absorbed, Given Values:
– G_{se} = 2.746
– G_{sb} = 2.526
– G_b = 1.030
$P_{ba} = 100\left(\frac{G_{se} - G_{sb}}{(G_{se})(G_{sb})}\right)G_b$
$P_{ba} = 100\left(\frac{2.746 - 2.526}{(2.746)(2.526)}\right)1.030 = 3.27\%$
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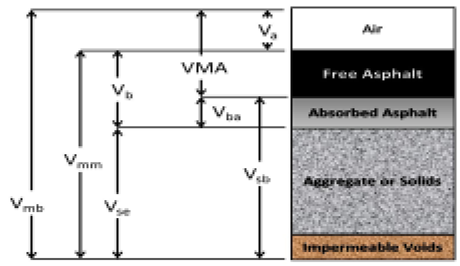
Slide 46

Effective Binder Content by Weight

- The effective binder content by weight is the total binder content minus the percent lost to absorption (based on mass of total mix)



$$P_{be} = P_b - \left(\frac{P_{ba}(P_s)}{100} \right)$$

Q&A: Does the effective binder content change with time or is it a constant value, and if it changes, why does it change?

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Effective Binder Content by Weight

- Effective Binder Content, Given Values:
  - $P_s = 95\%$
  - $P_{ba} = 3.27\%$
  - $P_b = 5.0$

$$P_{be} = P_b - \left(\frac{P_{ba}(P_s)}{100} \right)$$

$$P_{be} = 5.0 - \frac{(3.27)(95.0)}{100} = 1.89\%$$

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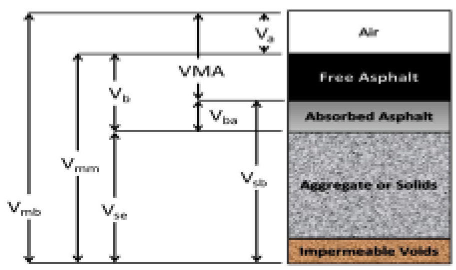
Effective Binder Content by Volume

- The effective binder content by volume is the total binder content minus the percent lost to absorption (based on volume of total mix)



$$V_{be} = VMA - V_a$$

Effective binder content by volume is an input to the MEPDG, and will be discussed in Module E.

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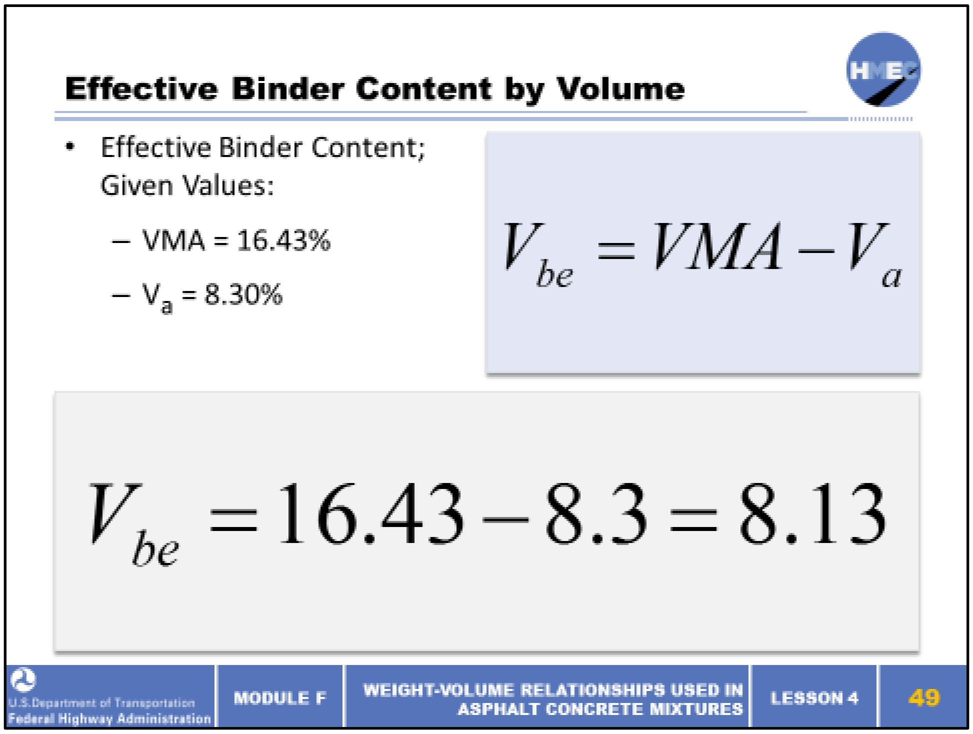
Effective Binder Content by Volume
HMEC
Effective Binder Content; Given Values:
– VMA = 16.43%
– V_a = 8.30%
$V_{be} = VMA - V_a$
$V_{be} = 16.43 - 8.3 = 8.13$
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Exercise 1: Example Problem

- Given:
  - Bulk Specific Gravity of Aggregate, G_{sb} = 2.705
  - Bulk Specific Gravity of the Mixture, G_{mb} = 2.329
  - Theoretical Max. Specific Gravity, G_{mm} = 2.521
  - Asphalt Binder Specific Gravity, G_b = 1.015
  - Binder Content, P_b = 5.0 % (by mass of total mix)
- Calculate:
  - Effective Specific Gravity
  - Percent Binder Asphalt
  - Percent Effective Binder
  - Percent Va
  - Percent VMA
  - Percent VFA

Let's walk through an example problem together. The goal of this exercise is to compute selected volumetric properties.

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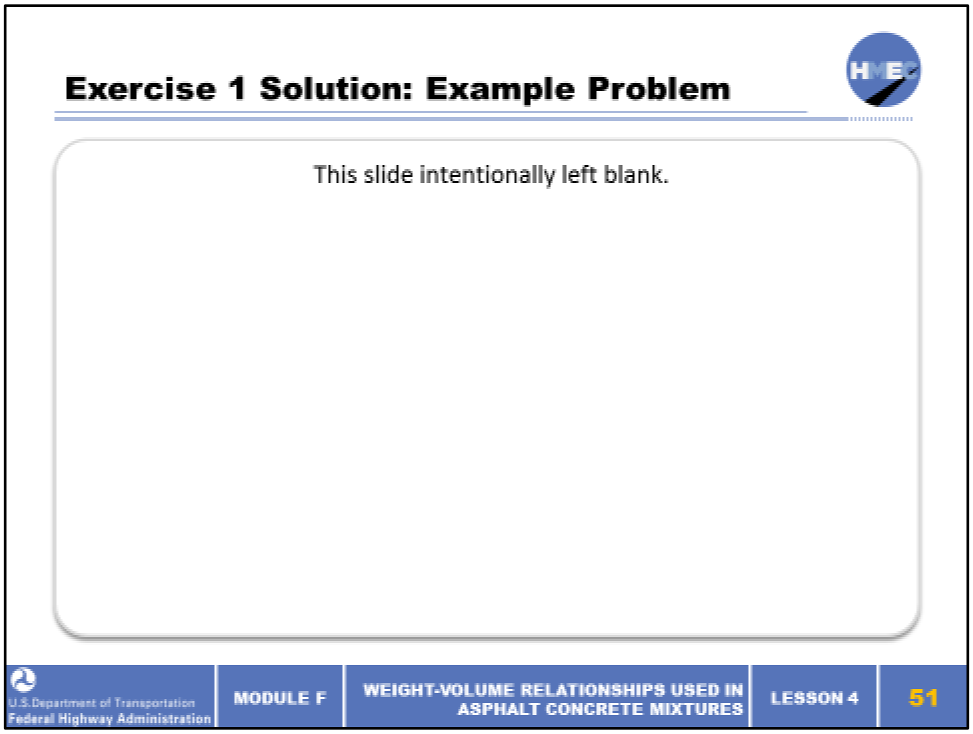
Exercise 1 Solution: Example Problem
HMEC
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U.S. Department of Transportation
Federal Highway Administration
MODULE F
WEIGHT-VOLUME RELATIONSHIPS USED IN ASPHALT CONCRETE MIXTURES
LESSON 4
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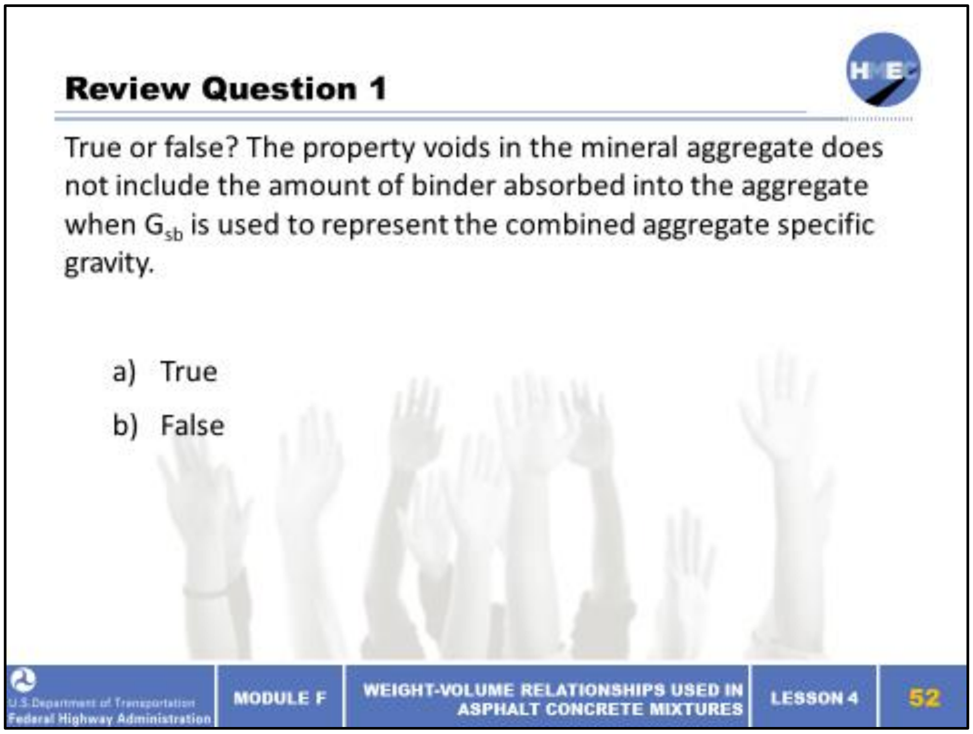
Slide 52


Review Question 1
True or false? The property voids in the mineral aggregate does not include the amount of binder absorbed into the aggregate when G_{sb} is used to represent the combined aggregate specific gravity.
a) True
b) False
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LESSON 4
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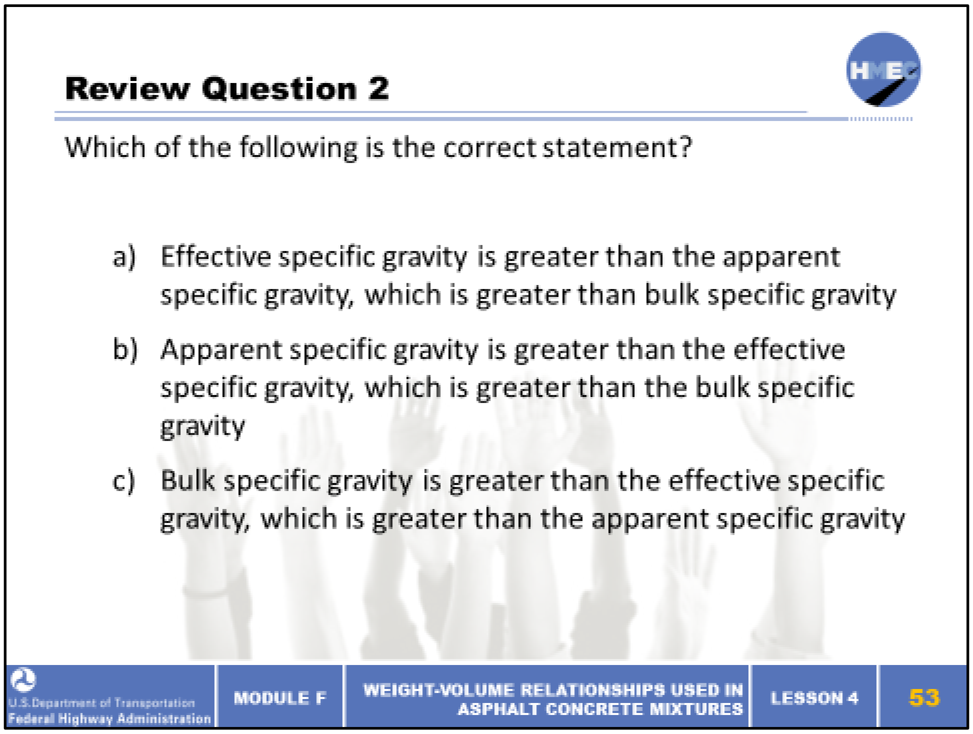
Slide 53


Review Question 2
HMEC
Which of the following is the correct statement?
a) Effective specific gravity is greater than the apparent specific gravity, which is greater than bulk specific gravity
b) Apparent specific gravity is greater than the effective specific gravity, which is greater than the bulk specific gravity
c) Bulk specific gravity is greater than the effective specific gravity, which is greater than the apparent specific gravity
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WEIGHT-VOLUME RELATIONSHIPS USED IN ASPHALT CONCRETE MIXTURES
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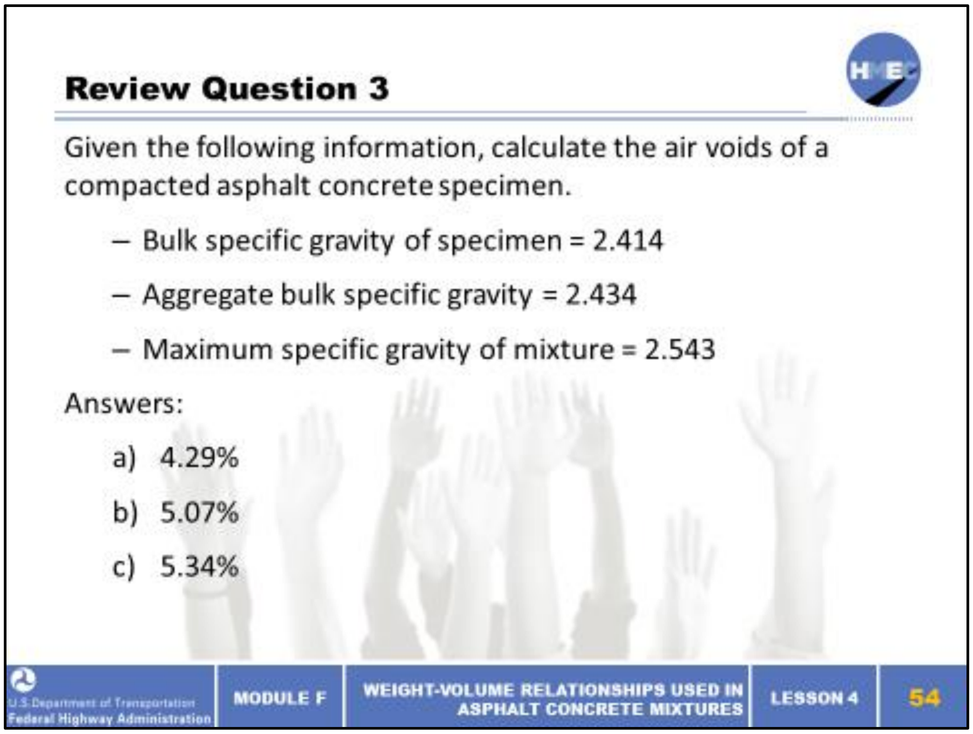
Slide 54


Review Question 3
Given the following information, calculate the air voids of a compacted asphalt concrete specimen.
– Bulk specific gravity of specimen = 2.414
– Aggregate bulk specific gravity = 2.434
– Maximum specific gravity of mixture = 2.543
Answers:
a) 4.29%
b) 5.07%
c) 5.34%
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Federal Highway Administration
MODULE F
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LESSON 4
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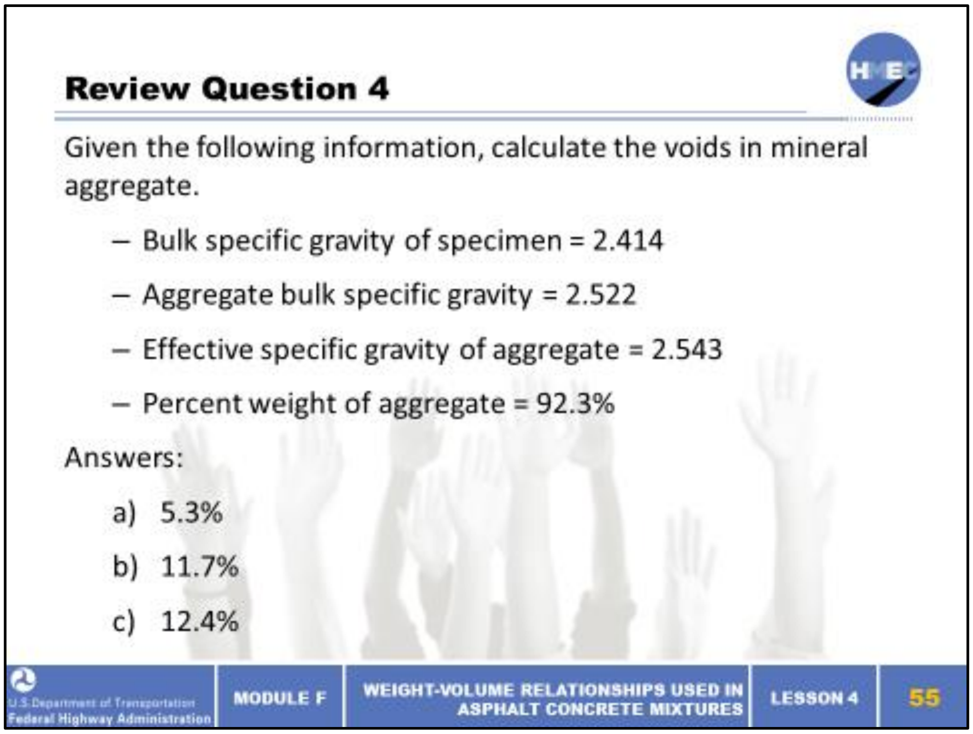
Slide 55


Review Question 4
Given the following information, calculate the voids in mineral aggregate.
– Bulk specific gravity of specimen = 2.414
– Aggregate bulk specific gravity = 2.522
– Effective specific gravity of aggregate = 2.543
– Percent weight of aggregate = 92.3%
Answers:
a) 5.3%
b) 11.7%
c) 12.4%
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MODULE F
WEIGHT-VOLUME RELATIONSHIPS USED IN ASPHALT CONCRETE MIXTURES
LESSON 4
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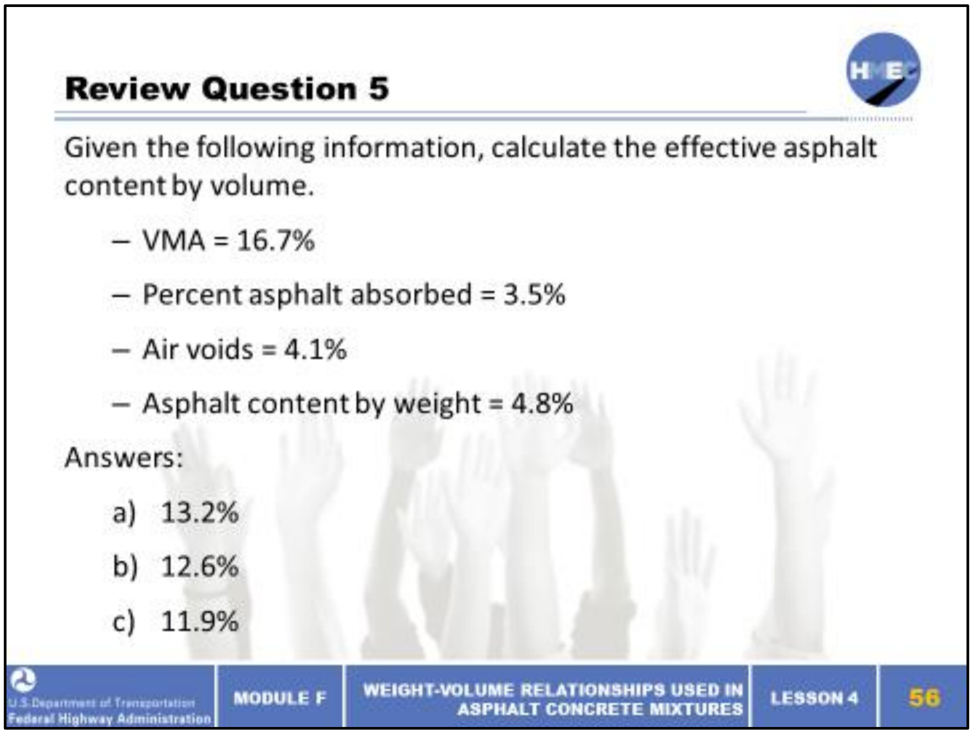
Slide 56


Review Question 5
Given the following information, calculate the effective asphalt content by volume.
– VMA = 16.7%
– Percent asphalt absorbed = 3.5%
– Air voids = 4.1%
– Asphalt content by weight = 4.8%
Answers:
a) 13.2%
b) 12.6%
c) 11.9%
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WEIGHT-VOLUME RELATIONSHIPS USED IN ASPHALT CONCRETE MIXTURES
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Learning Outcomes Review

You are now able to:

- Understand the relationship between the weight and volume of an asphalt mixture
- Calculate volumetric properties

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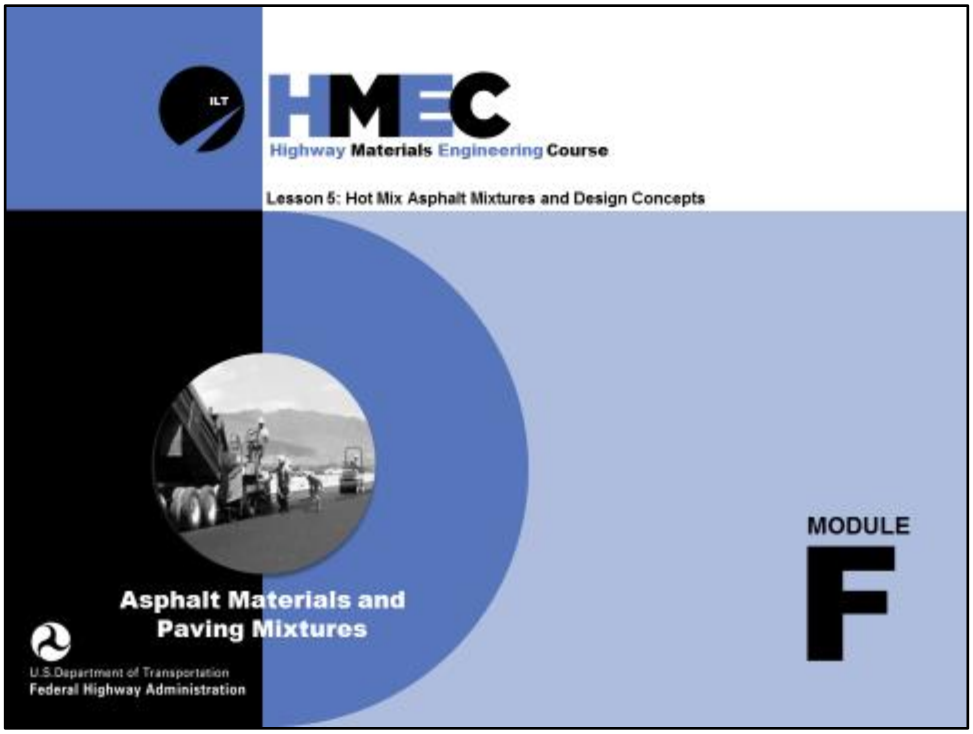
Slide 1


ILT
HMEC
Highway Materials Engineering Course
Lesson 5: Hot Mix Asphalt Mixtures and Design Concepts
Asphalt Materials and Paving Mixtures
U.S.Department of Transportation
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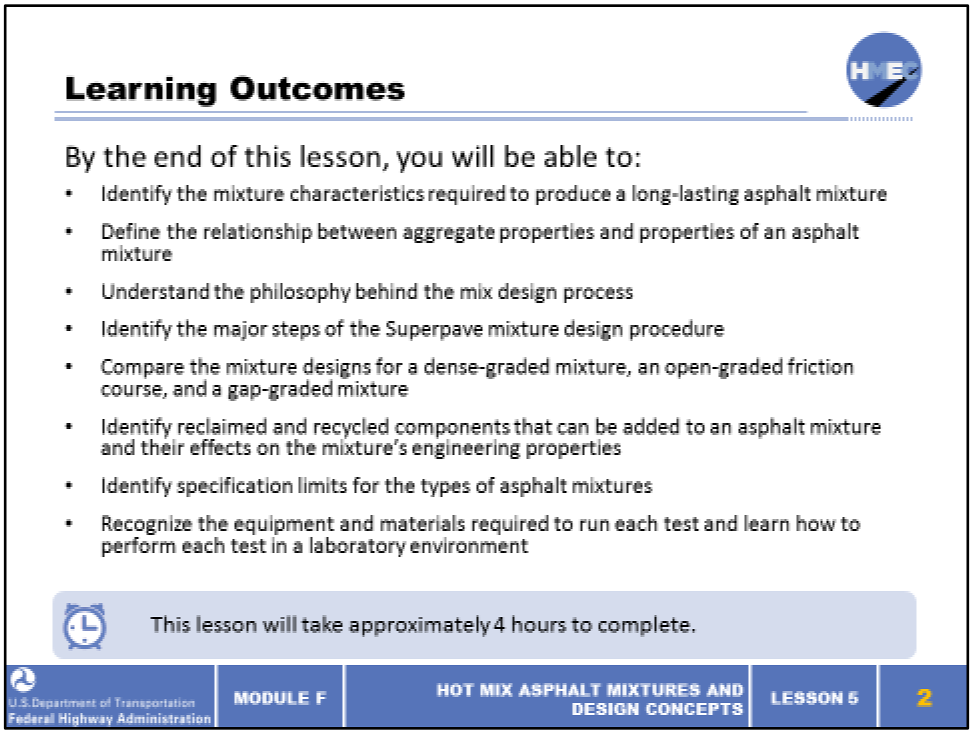
MODULE
F

Slide 2


Learning Outcomes
By the end of this lesson, you will be able to:
• Identify the mixture characteristics required to produce a long-lasting asphalt mixture
• Define the relationship between aggregate properties and properties of an asphalt mixture
• Understand the philosophy behind the mix design process
• Identify the major steps of the Superpave mixture design procedure
• Compare the mixture designs for a dense-graded mixture, an open-graded friction course, and a gap-graded mixture
• Identify reclaimed and recycled components that can be added to an asphalt mixture and their effects on the mixture's engineering properties
• Identify specification limits for the types of asphalt mixtures
• Recognize the equipment and materials required to run each test and learn how to perform each test in a laboratory environment
This lesson will take approximately 4 hours to complete.
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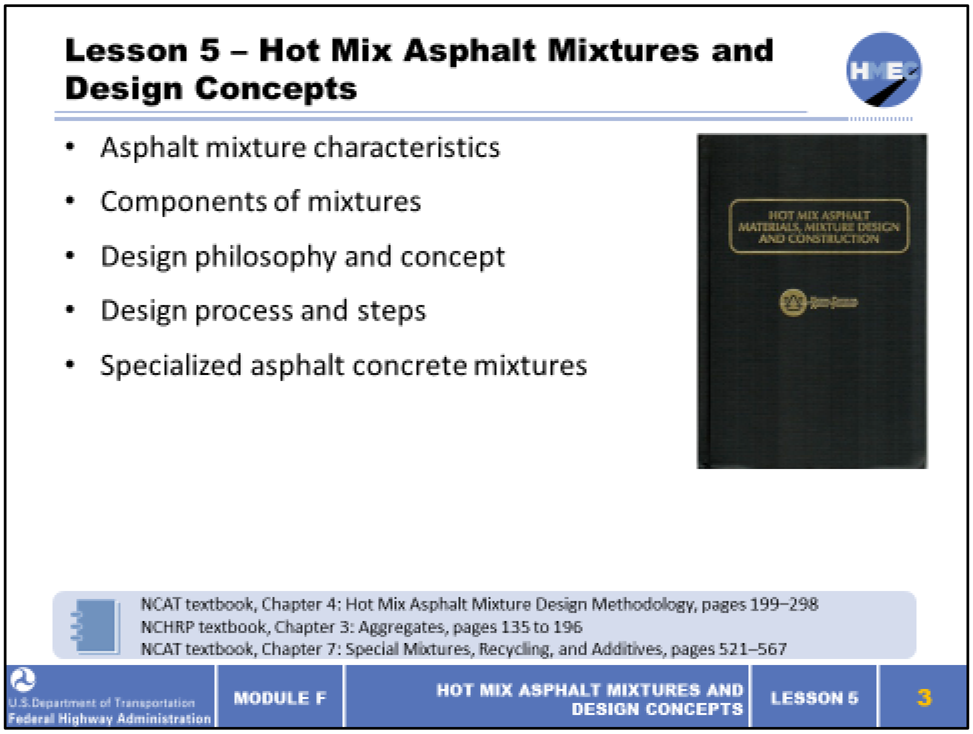
MODULE F
HOT MIX ASPHALT MIXTURES AND DESIGN CONCEPTS
LESSON 5
2

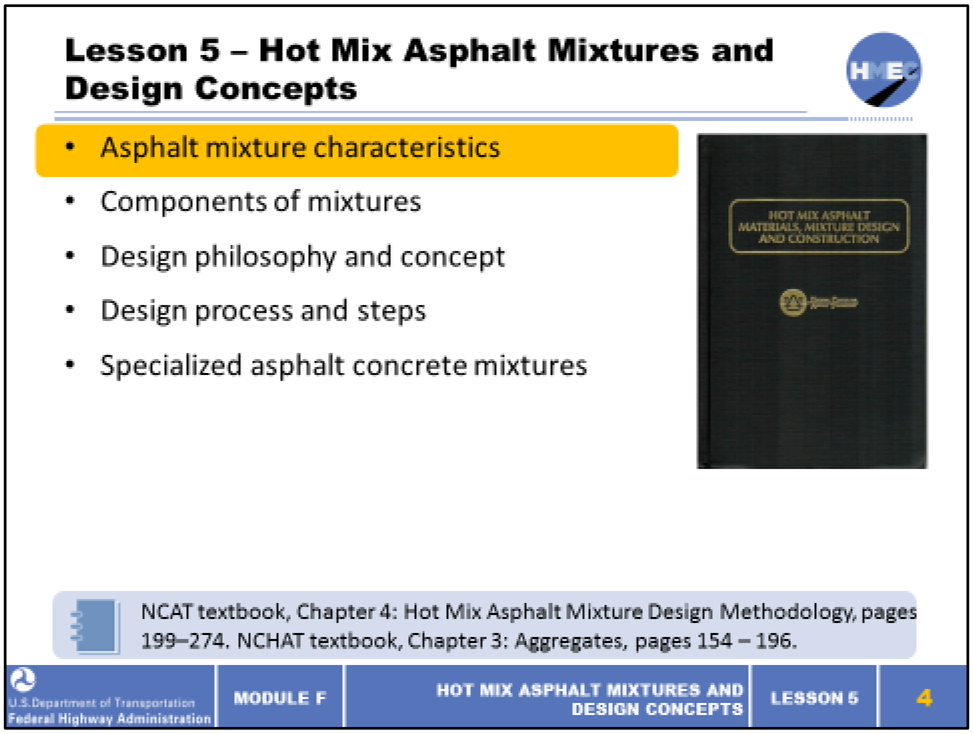
Slide 3



The individual sections or topics of this lesson are listed on this slide.

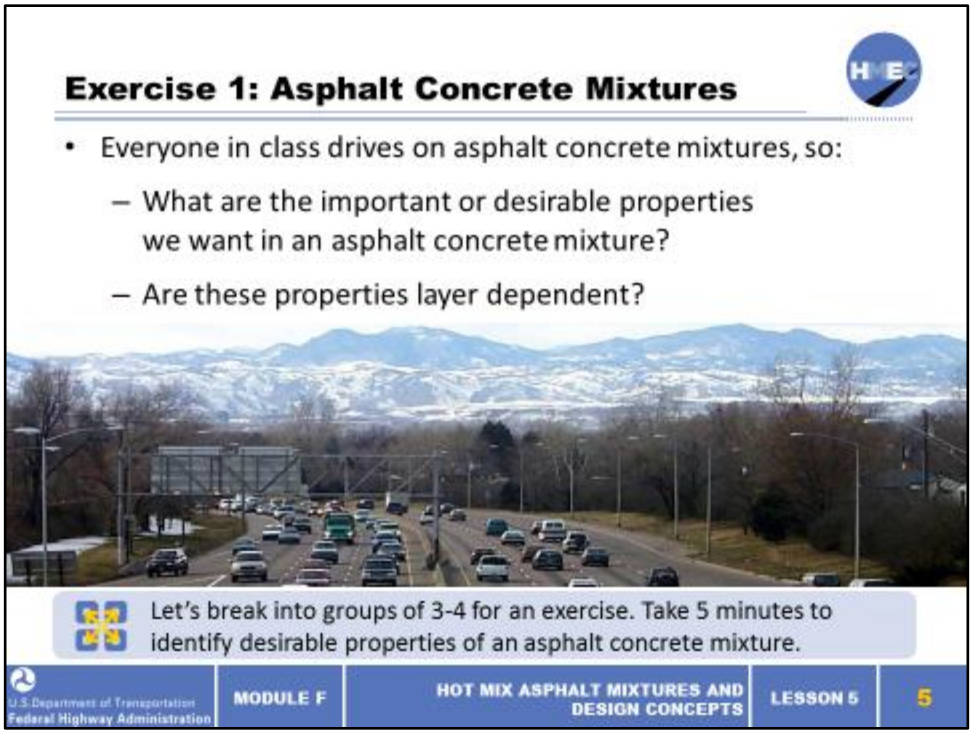
Note some of the information covered within this lesson comes from many different sources but was generally taken from Chapter 4, Hot Mix Asphalt Mixture Design Methodology, Chapter 3, Aggregates, and Chapter 7, Special Mixtures, Recycling, and Additives, of the NCAT textbook and AASHTO test standards and specifications.

Slide 4



Note some of the information covered within this lesson and section comes from many different sources but was generally taken from Chapter 4, Hot Mix Asphalt Mixture Design Methodology, and Chapter 3, Aggregates, of the NCAT textbook and AASHTO test standards and specifications.

Slide 5


Exercise 1: Asphalt Concrete Mixtures
Everyone in class drives on asphalt concrete mixtures, so:
What are the important or desirable properties we want in an asphalt concrete mixture?
Are these properties layer dependent?
Let's break into groups of 3-4 for an exercise. Take 5 minutes to identify desirable properties of an asphalt concrete mixture.
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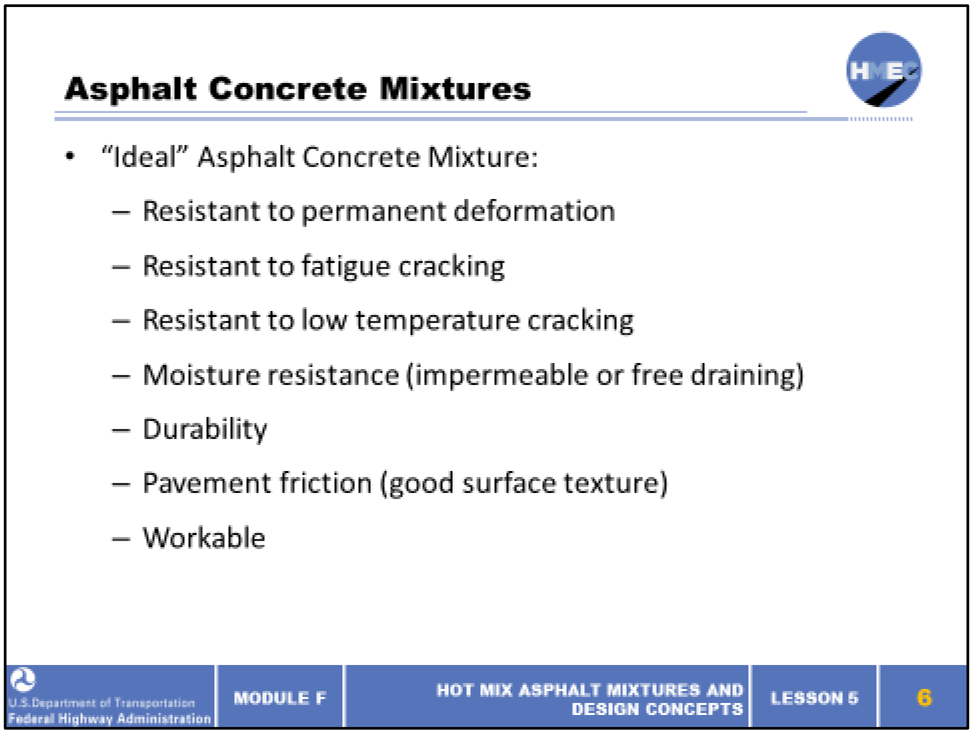
MODULE F
HOT MIX ASPHALT MIXTURES AND DESIGN CONCEPTS
LESSON 5
5

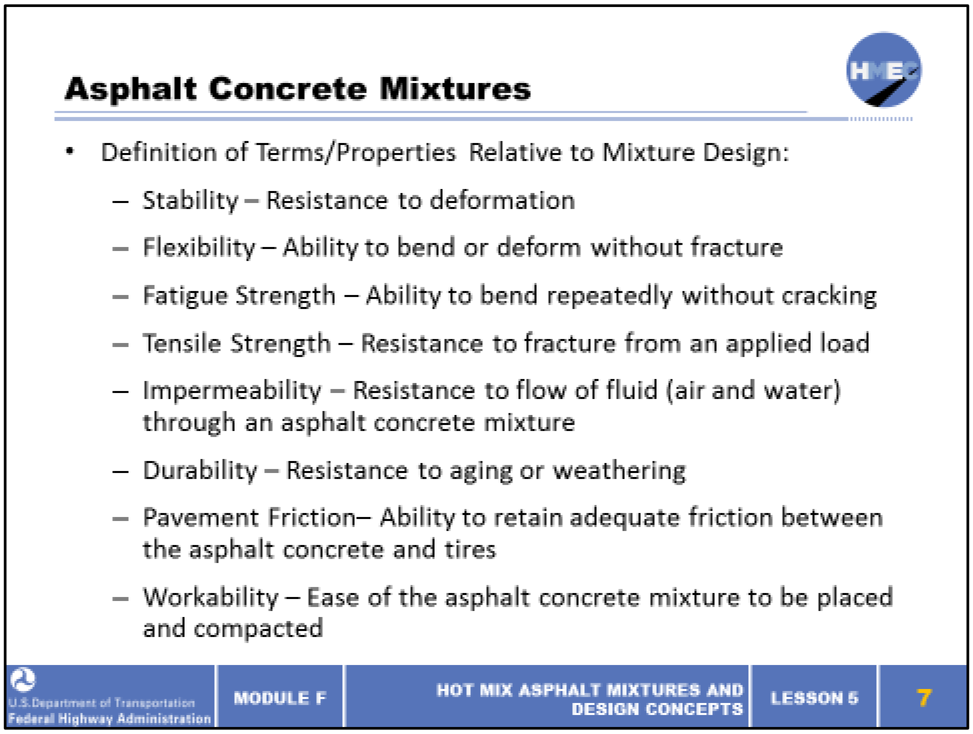
Slide 6



A hot mix asphalt mixture can possess excellent resistance to fatigue cracking and good durability characteristics but poor resistance to permanent deformation.

It is the objective of the mixture design process (covered later in this lesson) to select an aggregate gradation and binder content (with or without modifiers) to achieve these characteristics. Consideration must be given to the pavement structure (covered in Lesson 7) in combination with the mixture design to achieve success in this regard.

Slide 7

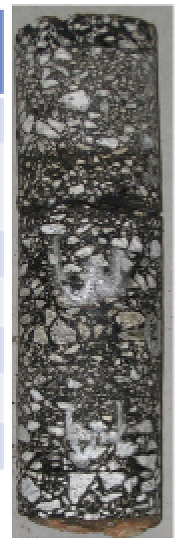


All of these terms are defined in the NCAT textbook, as listed above.

Slide 8

Asphalt Concrete Mixtures

Mix Property	Top Layers	Mid-Depth Layers	Bottom Layers
Stability	✓	✓	
Flexibility, Fatigue Strength		✓	✓
Tensile Strength	✓		
Impermeability	✓	✓	✓
Durability	✓	✓	✓
Pavement Friction	✓		
Workability	✓	✓	✓



Note: Open-graded friction courses are often used as a wearing surface to provide pavement friction.

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The different mixture properties are defined and covered under the section "Objectives and Elements of Mix Design" of Chapter 4, Hot Mix Asphalt Mix Design Methodology, pages 210 to 212. Specifically, mixtures can be designed to optimize specific types of performance and the properties for the wearing surface versus base layers are different.

For example, in the uppermost portion of a pavement structure, the mix designer should strive to maximize stability, pavement friction, durability, and tensile strength (in that order) in most situations. In the lowest portion of a pavement structure, the mixture should be designed to maximize fatigue resistance and durability.

In between the upper- and lower-most layers, it is common to design for stability and durability. This is good in theory, and the practice may be followed with newly designed thick pavements or when thick overlays will be used. It is even possible to use different materials (binders and aggregates) in mixtures placed in different layers to optimize performance and minimize cost.

Slide 9

Asphalt Concrete Mixtures

Mix Property	Component and Construction Effects on Mix Properties				
	Aggregate		Asphalt		Air Voids
	Texture	Gradation	Content	Stiffness	
Stability	Rough	Dense	Low	High	Low
Fatigue Strength	Rough	Dense	High	High	Low
Tensile Strength	Rough	Dense	Low	High	Low
Impermeability	---	Dense, Fine	High	Low	Low
Durability	Rough	Dense, Fine	High	Low	Low
Pavement Friction	Rough	Open to Coarse	Low	High	High
Workability	Smooth	Fine	High	Low	---

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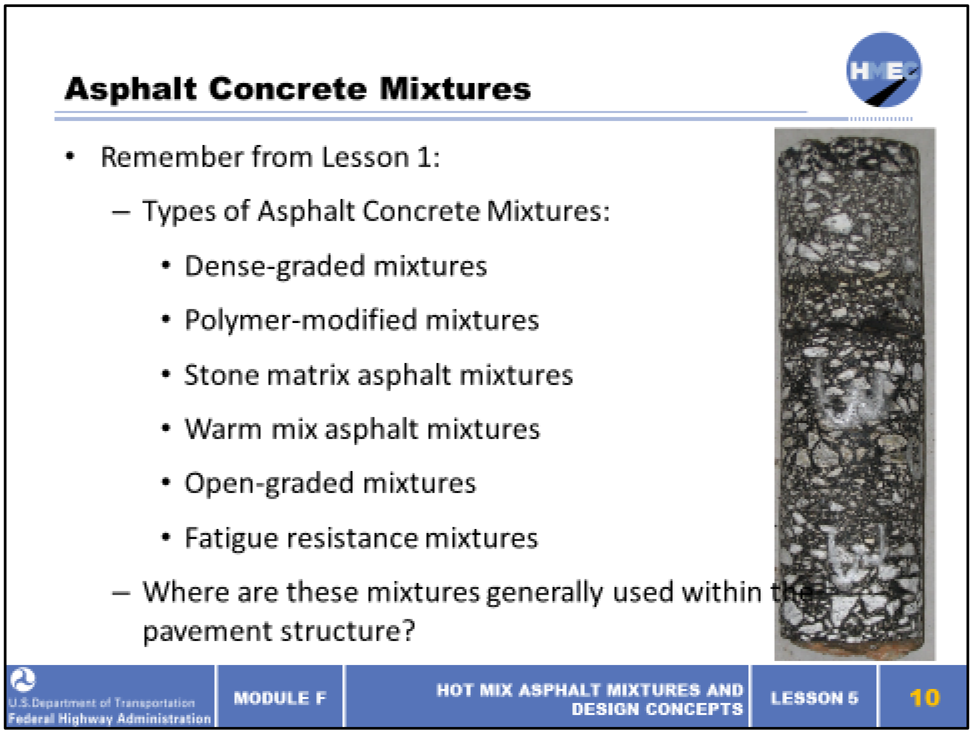
As an example, in order to improve stability, relatively high asphalt stiffness, a dense aggregate gradation, low asphalt content, and high degree of compaction (in-place density) are desirable.

To improve durability, a dense gradation, high asphalt content, and a high degree of compaction are desirable.

In general, denser gradation and a high degree of compaction are desirable for all mixture properties. However, desirable relative asphalt stiffness and content levels vary for different mixture properties. A key issue to mix design is striking the correct balance.

Slide 10


Asphalt Concrete Mixtures
HMEC
• Remember from Lesson 1:
– Types of Asphalt Concrete Mixtures:
• Dense-graded mixtures
• Polymer-modified mixtures
• Stone matrix asphalt mixtures
• Warm mix asphalt mixtures
• Open-graded mixtures
• Fatigue resistance mixtures
– Where are these mixtures generally used within the pavement structure?
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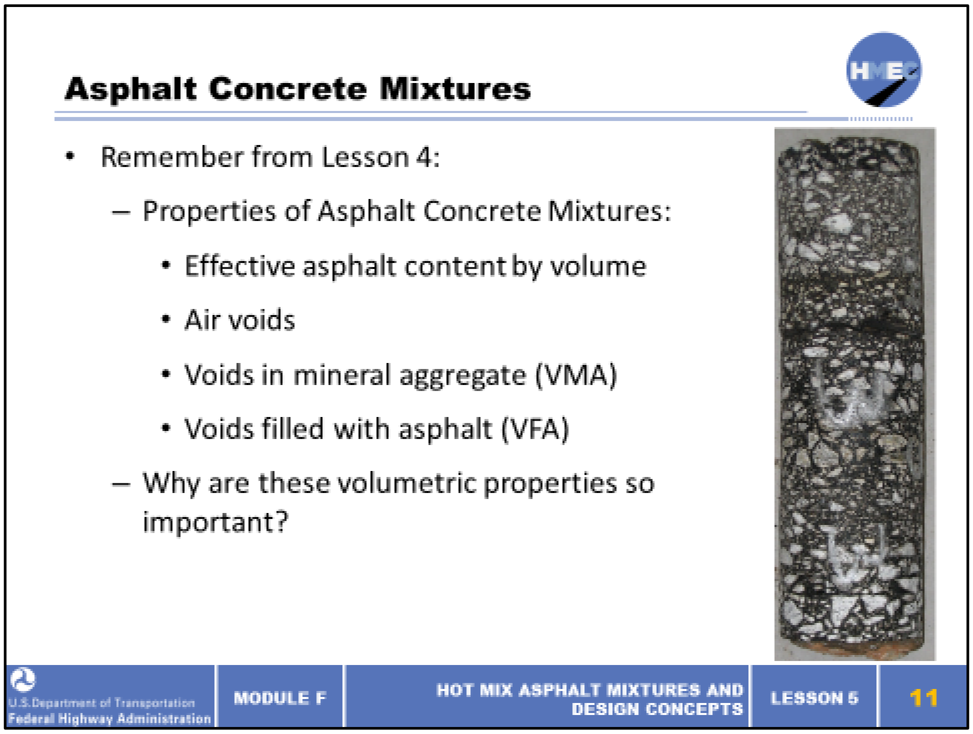
Slide 11


Asphalt Concrete Mixtures
HMEC
• Remember from Lesson 4:
– Properties of Asphalt Concrete Mixtures:
• Effective asphalt content by volume
• Air voids
• Voids in mineral aggregate (VMA)
• Voids filled with asphalt (VFA)
– Why are these volumetric properties so important?
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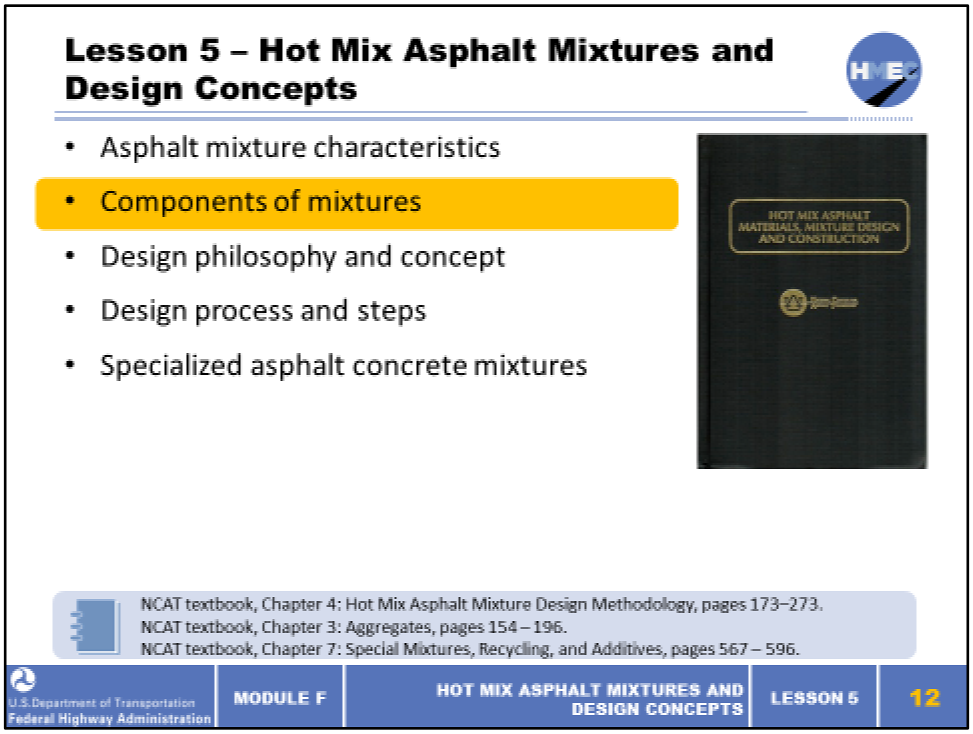
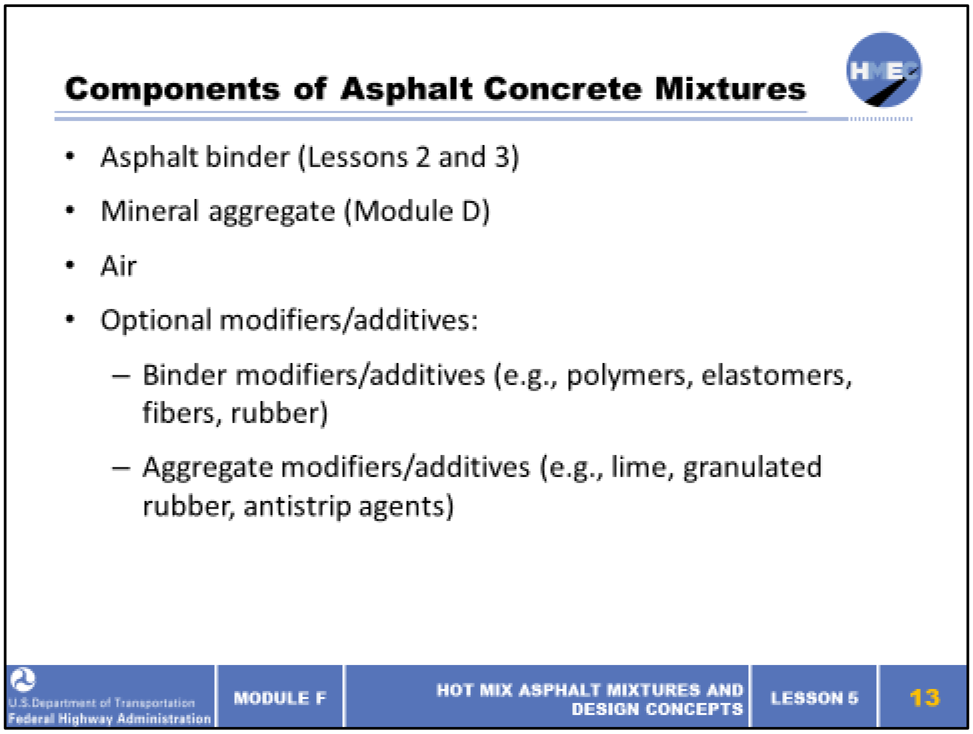
Slide 12



The information from Module D on aggregates and Lesson 3 from this module on asphalt binders. Both Module D and Lesson 3 from Module F were focused on the important properties of these material components.

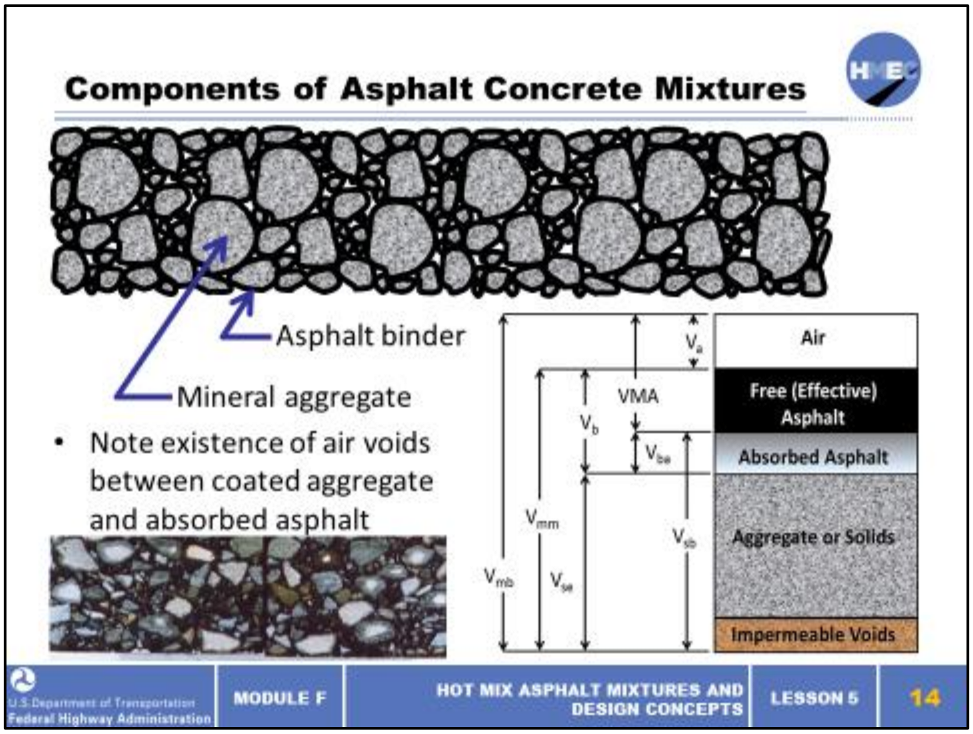
Slide 13



Asphalt concrete is composed of asphalt binder, mineral aggregate, and air.

Lessons 2 and 3 focused on the asphalt binder and module D focused on the mineral aggregate. This section of Lesson #5 will focus more on the aggregate grading and other properties that can have a significant impact on the mixture design and the amount of asphalt binder from the mixture design process, which will be discussed later within this lesson.

Slide 14



This slide also includes a picture of the cross-section of a core or compacted asphalt concrete mixture. The void space between aggregate particles (voids in mineral aggregate or VMA) is comprised of asphalt binder (with or without modifier) and air.

The schematic indicates the approximate proportions (not to scale) of the components in the mix. Note that asphalt binder is typically absorbed by the aggregate; for some of the coarser aggregate particles, only a portion of the center of the aggregate particle has not absorbed any asphalt. Absorption occurs rapidly during the mixing process and storage process when the mixture temperature is high. Absorption can still occur during the hotter summer months earlier in the life of the pavement, but it occurs at a much slower rate.

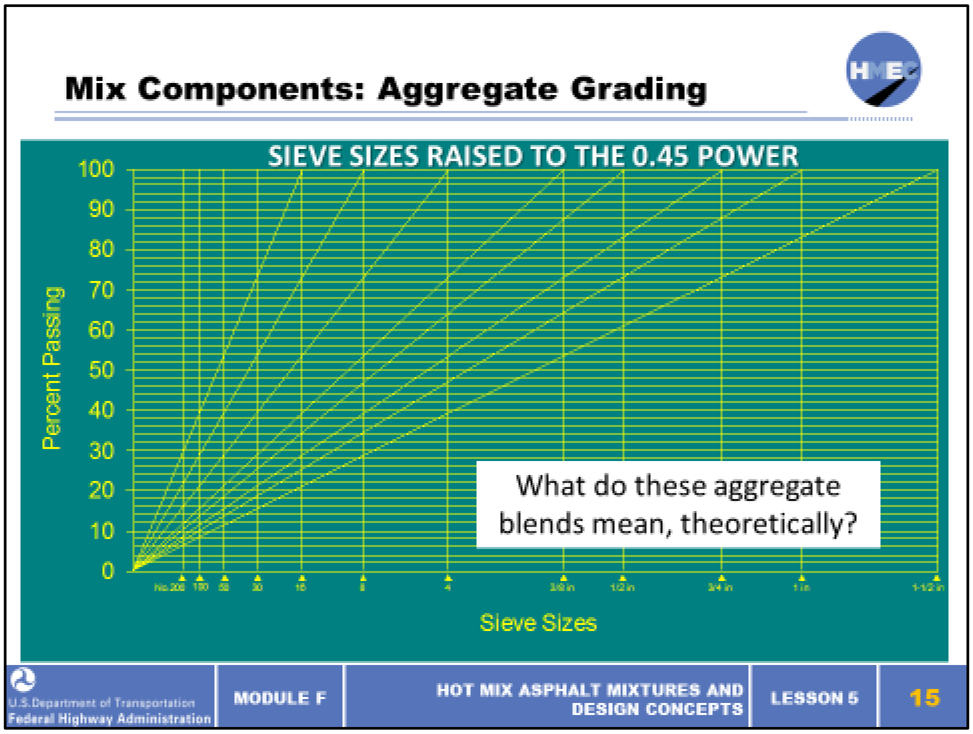
Percentages by volume at design:
• Air = 4%
• Effective asphalt binder = 6 to 16%
• Aggregate = 80 to 90%

The following summarizes the key properties that have a significant impact on the mixture design, as well as performance:

• Asphalt binder – stiffness across all temperature ranges, especially the stiffness in the higher temperature range that occurs during production and compaction. From Lessons #2 and #3, however, the asphalt binder grade is selected based on climate and traffic considerations.
• Mineral aggregate – gradation, shape, surface texture, and absorption.

Aggregate properties are largely responsible for the behavior of HMA mixtures, and largely control the stability or permanent deformation characteristics of mixtures as well as the friction number and durability. Shape, absorption, and surface texture determine, to a large degree, the binder content of the mixture. Tensile strength, fatigue resistance, and flexibility are also controlled to a degree by aggregate properties. Hard, tough, and sound aggregates are desired for durability and resistance to breakdown during handling and loading. Angular aggregates with rough surface texture should contribute to internal mixture friction and improve mixture stability.

Slide 15


Mix Components: Aggregate Grading
HMEC
SIEVE SIZES RAISED TO THE 0.45 POWER
100
90
80
70
60
50
40
30
20
10
0
Percent Passing
What do these aggregate blends mean, theoretically?
Sieve Sizes
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MODULE F
HOT MIX ASPHALT MIXTURES AND DESIGN CONCEPTS
LESSON 5
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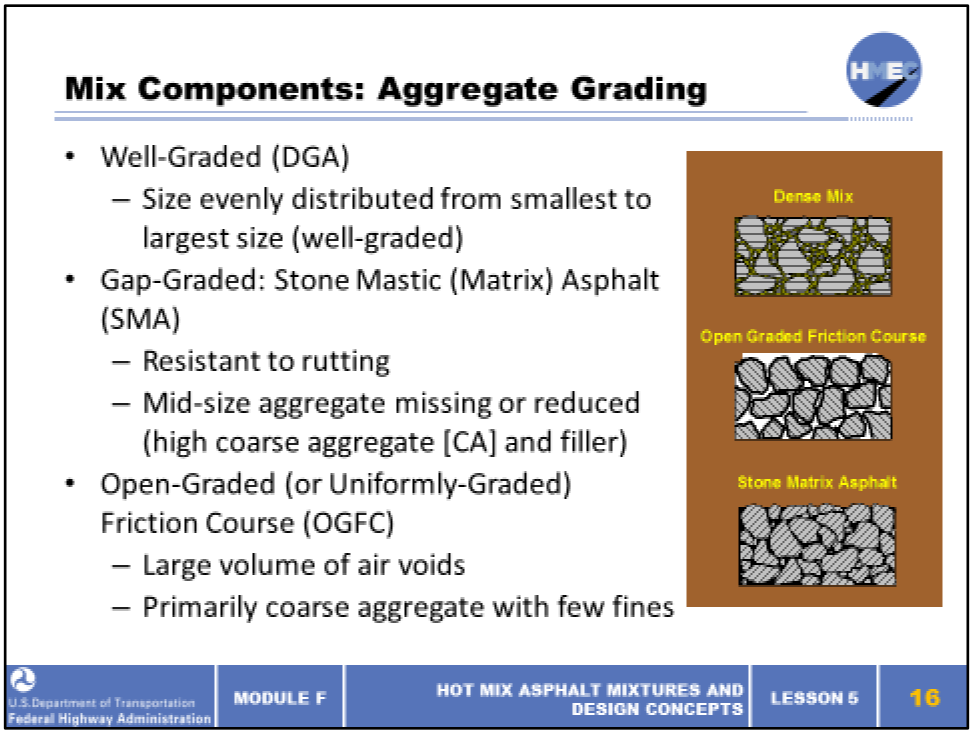
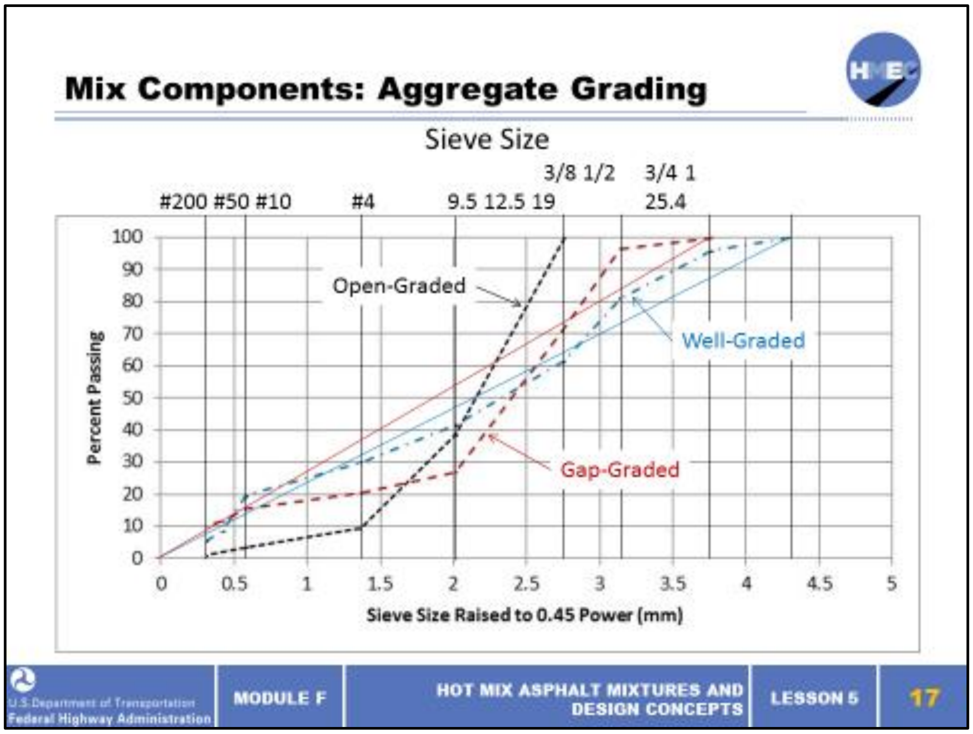
Slide 16



Mixture types are defined by the gradation of the aggregate comprising the mixture, and these are the three most common mixture types used in the United States are listed. Of these, dense well-graded mixtures have seen the widest use in the U.S.

As defined in Module D, aggregate gradation is the distribution of particle sizes expressed as a percentage of the total weight of aggregate.

Slide 17

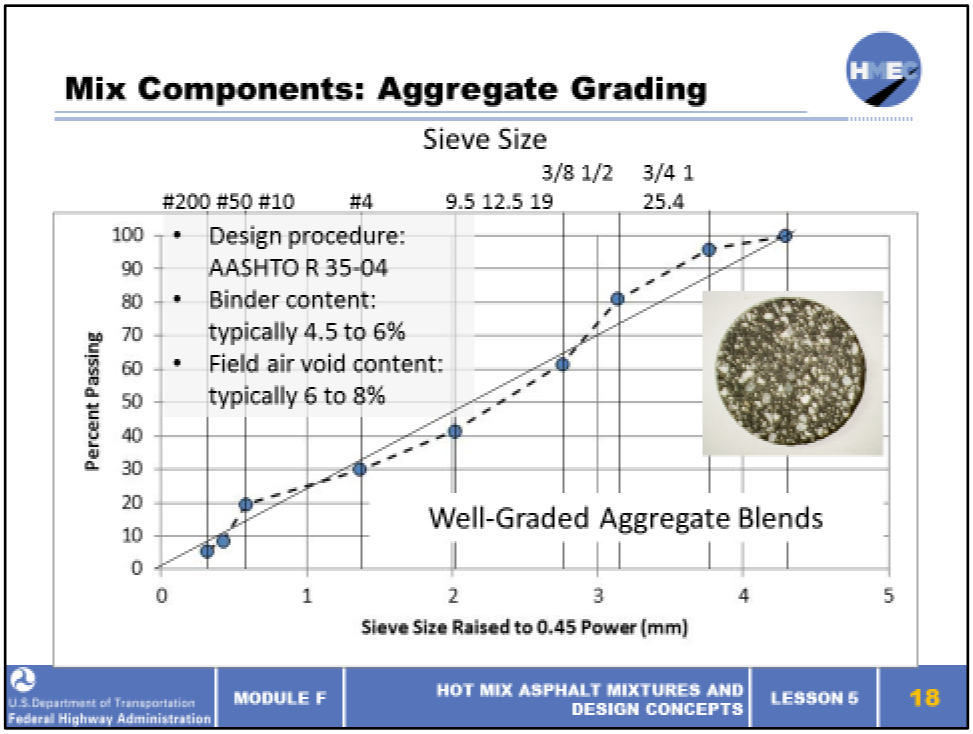


Note that the dense gradation closely follows the maximum density line (i.e., aggregate particle sizes are, for the most part, distributed along the maximum density line). The reason for deviating from the maximum density line is to provide some space or room for the asphalt binder. The greater the deviation from the maximum density line, theoretically, the greater the higher the amount of asphalt binder—assuming the same design air void content.

The open gradation falls entirely below the maximum density line. That is, it consists of aggregate particles that are coarser (larger) than prescribed by the maximum density line.

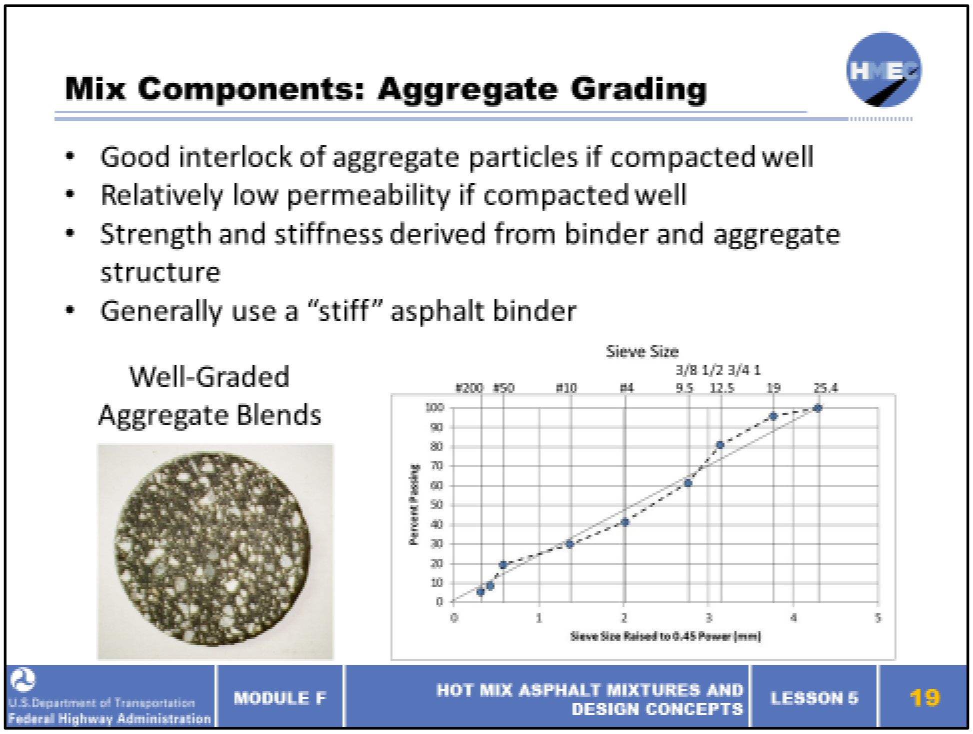
For the gap gradation, the mid-sized aggregates have been reduced from that prescribed by the maximum density line, thus producing a gap in these sizes.

Slide 18



Well-graded mixtures have been used extensively in the United States as a wearing course (surface layer), more so than other mixture types. Asphalt binder contents for these mixtures have typically ranged from 4.5 to 6% by weight, and in-place air voids have typically ranged from about 6 to 8%.

Slide 19



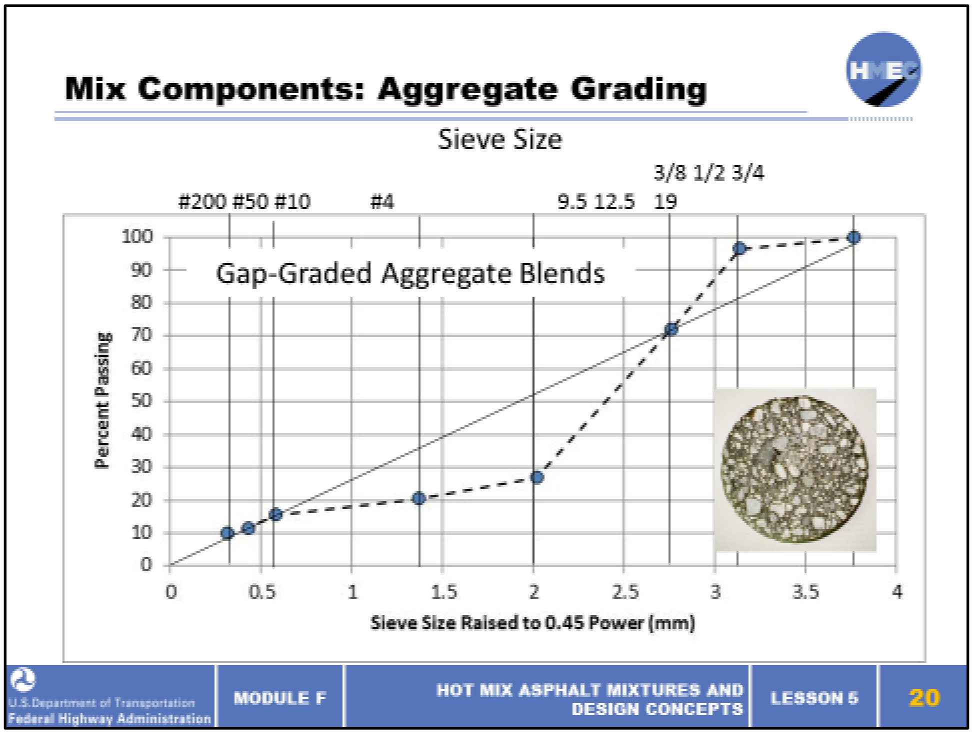
The aggregate gradation provides good interlock between particles due to the evenly distributed sizes. If these mixtures are compacted well, they possess relatively low permeability, resulting in good resistance to moisture damage and age hardening. Lower permeability of the mix results in minimal infiltration of moisture/runoff that can introduce moisture damage issues, along with degradation of the HMA through freeze-thaw cycling.

The strength and stiffness of the mixture are derived from a combination of the aggregate gradation and binder, which is typically relatively stiff. A stiff binder is typically chosen to mitigate permanent deformation.

Conversely, the principal disadvantages include difficulty in selecting the design binder content and the fact that in-place air void content and permeability are not optimum for good moisture damage resistance. Selection of the design or target binder content involves balancing resistance to permanent deformation with resistance to fatigue cracking. Also, good durability is typically associated with higher binder contents (good for fatigue cracking resistance), but the selected target binder content generally results in thin binder films on the aggregate and, consequently, not optimal for good durability. Another disadvantage of the well-graded aggregate blends is that the larger aggregate particles may not have stone on stone contact—

some state that the "larger aggregate particles are floating in a "sea of fines," as illustrated in the core photo of this slide and the illustration in the previous slide.

Slide 20



One of the more common mixture types which uses the gap-graded aggregate blend is the stone mastic (matrix) asphalt (SMA).

Gap-graded and SMA mixtures have a reduced percentage of mid-size aggregates as compared with well-graded mixtures. Gap-graded mixtures are also a form of dense-graded mixtures. They are composed of aggregates that are, for the most part, coarser than that prescribed by the maximum density line and contain a relatively high percentage of minus 75 μm material.

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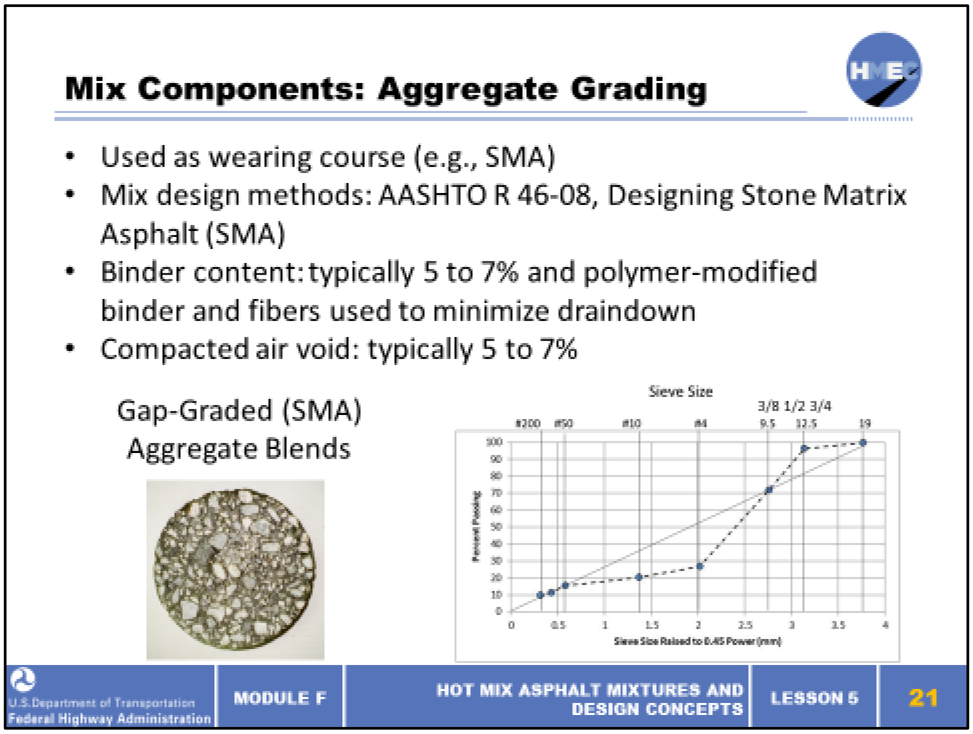
Slide 21



Mix design methods for typical State mixtures use the methods for dense-graded mixtures as a basis for design of gap-graded mixtures. For SMA mixtures, European methods are available and NCAT has recently developed a procedure for use in the United States, which is included in AASHTO R 46-08, Designing SMA Mixtures.

Conversely, as for well-graded blends, the principal disadvantages include difficulty in selecting the design or target binder content and the fact that in-place air void content and permeability are not optimum for good moisture damage resistance. Selection of the design binder content involves balancing resistance to permanent deformation with resistance to fatigue cracking.

These mixtures are typically produced with binder contents ranging from about 5 to 7% by weight and are compacted to air void contents ranging from about 5 to 7%. Good durability is typically associated with higher binder contents which are good for fatigue cracking resistance and fracture strength. Unlike well-graded aggregate blends, however, the selected design binder content generally results in thicker binder films in comparison to well-graded gradations and consequently have good durability.

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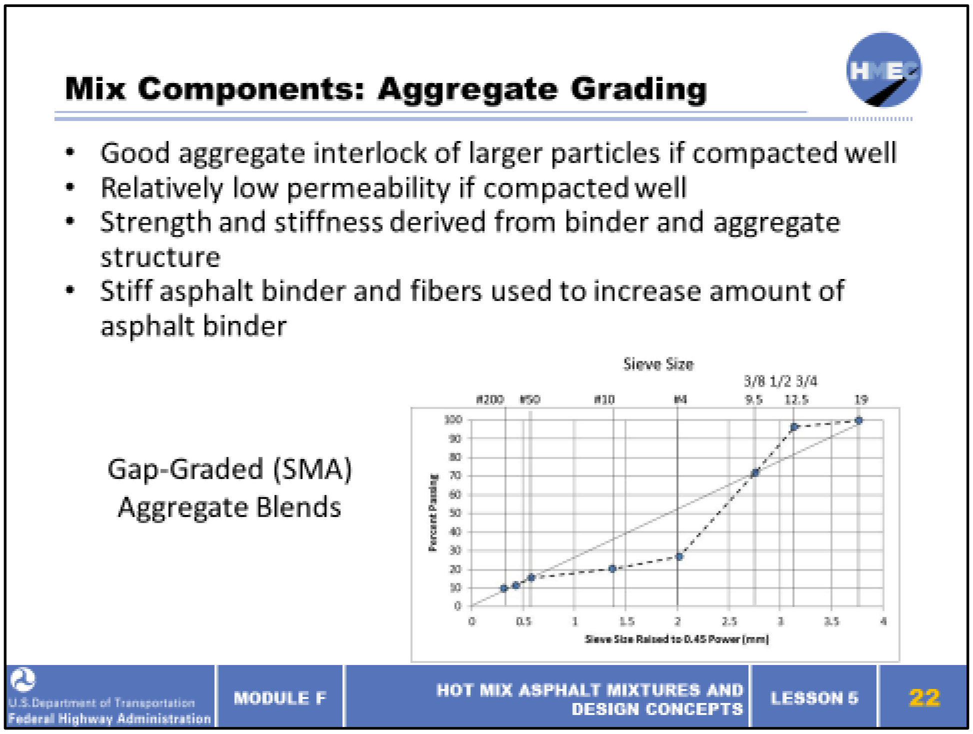
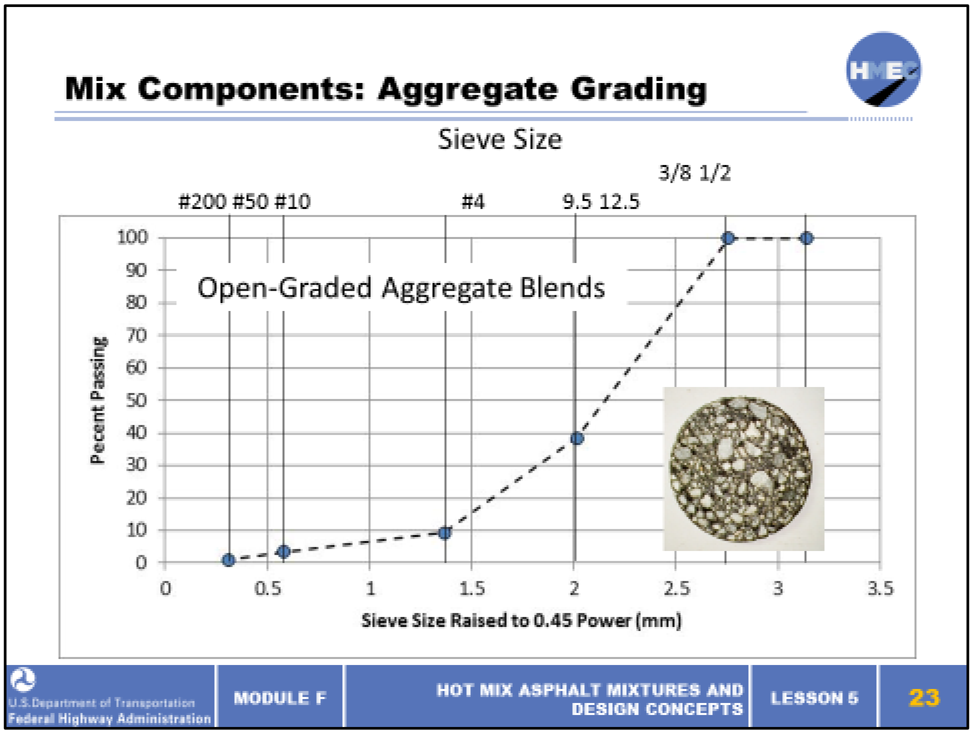
Slide 22



The materials used the produce gap-graded mixtures are usually the same as those for dense-graded mixtures.

Where possible, locally available materials are used to minimize cost.

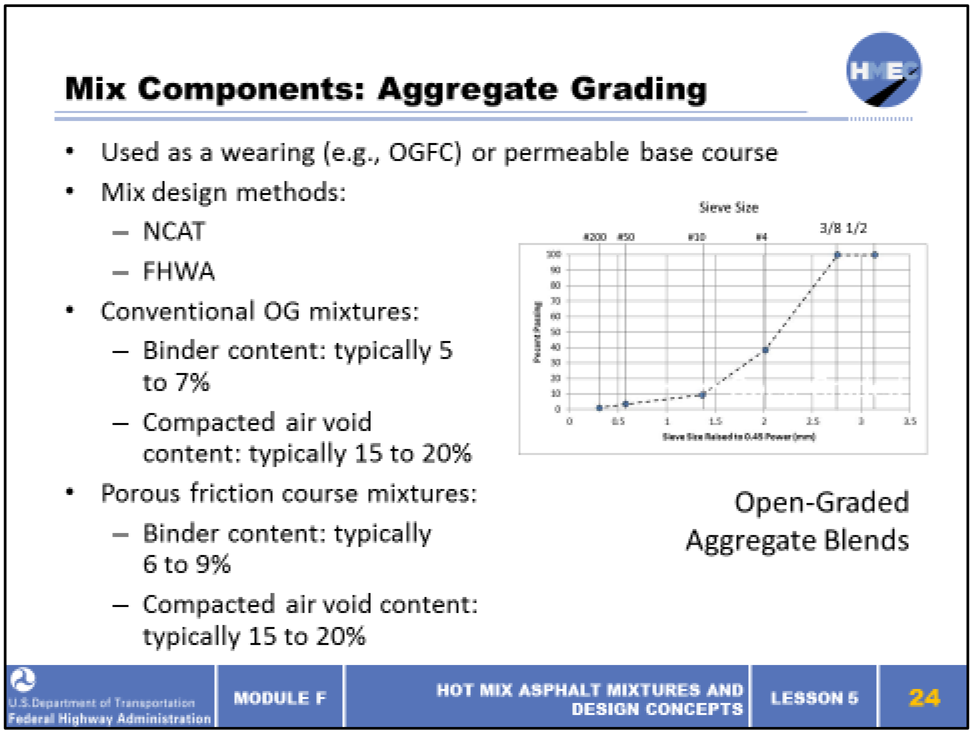
Slide 23



Open-graded mixtures have gradations that generally fall below the maximum density line on the 0.45 power chart. Such mixtures have aggregate sizes that are coarser than that prescribed by the maximum density line (i.e., a higher percentage of aggregate sizes retained on sieves, or equally, a lower percentage of aggregate sizes passing sieves, relative to the percentage prescribed by the maximum density line).

Such mixtures have predominately coarse aggregate with relatively few fines.

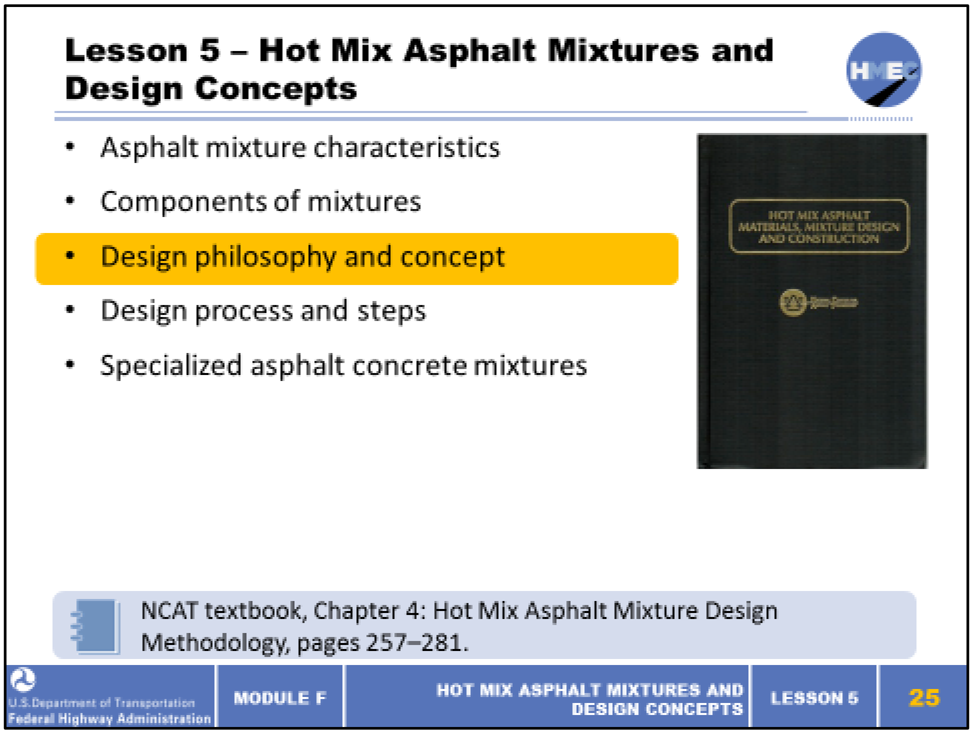
Slide 24



As with dense-graded mixtures, locally available aggregates are usually utilized in open-graded mixtures. These can be unmodified or modified with lime or antistrip liquids. Locally available binders are also usually utilized, but these are typically modified to prevent draindown during storage and transport.

As noted in the previous discussion, draindown is the physical separation of the asphalt binder from the aggregate surface during storage and transport. When it occurs, it results in segregated areas of over and under-asphalted conditions. This, in turn, can lead to flushing in some areas and raveling in others.

Slide 25



Note some of the information covered within this lesson and section comes from many different sources but was generally taken from Chapter 4, Hot Mix Asphalt Mixture Design Methodology, of the NCAT textbook and AASHTO test standards and specifications.

Slide 26

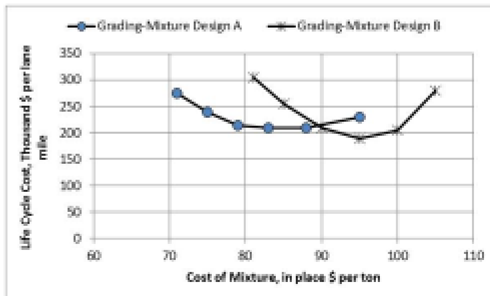
This section of the lesson will be limited to a description of mix design methods for dense-graded mixtures. The primary objective of all asphalt concrete mix design methods is to develop an economical blend of available materials that meets specific project design requirements and specifications. The aggregate composes between 93 to 96 % by weight, while the binder composes about 70% of the cost of the mixture. Thus, contractors responsible for mixture design tend to select an aggregate grading that minimizes the target binder content while still meeting the project specifications.

This sounds like a simple process, as the materials are limited to aggregates, binder, and in some cases modifiers. However, because economics must be given due consideration, asphalt concrete mix design is not always a simple process.

Slide 27

Asphalt Concrete Mix Design Philosophy

- Factors in selecting an economical blend of mixture components:
  - Types of materials; aggregate, binder, additives, etc.
  - Cost of materials
  - Design/service life
  - Risk or reliability
  - Layer thickness requirements
  - Traffic
  - Foundation or supporting layers
  - Climate
  - Highway classification



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The primary objective of all asphalt concrete mix design methods is to develop an economical blend of available materials that meets specific project design requirements and specifications. The issue is how do we define the economical blend and what factors should be considered. Many different factors should be considered in selecting the mixture design to be used so that the life cycle cost to the pavements is minimized.

To make this judgment requires the use of performance prediction models to evaluate different mixture designs and target binder contents and estimate the service life of each proposed mixture. This slide lists the different factors that need to be considered in estimating the cost to produce and place the mixture as well as the life cycle cost estimated through the service life estimates.

Each of the factors should be noted and discussed in terms of how it affects the cost of the mixture and/or overall cost of the pavement.

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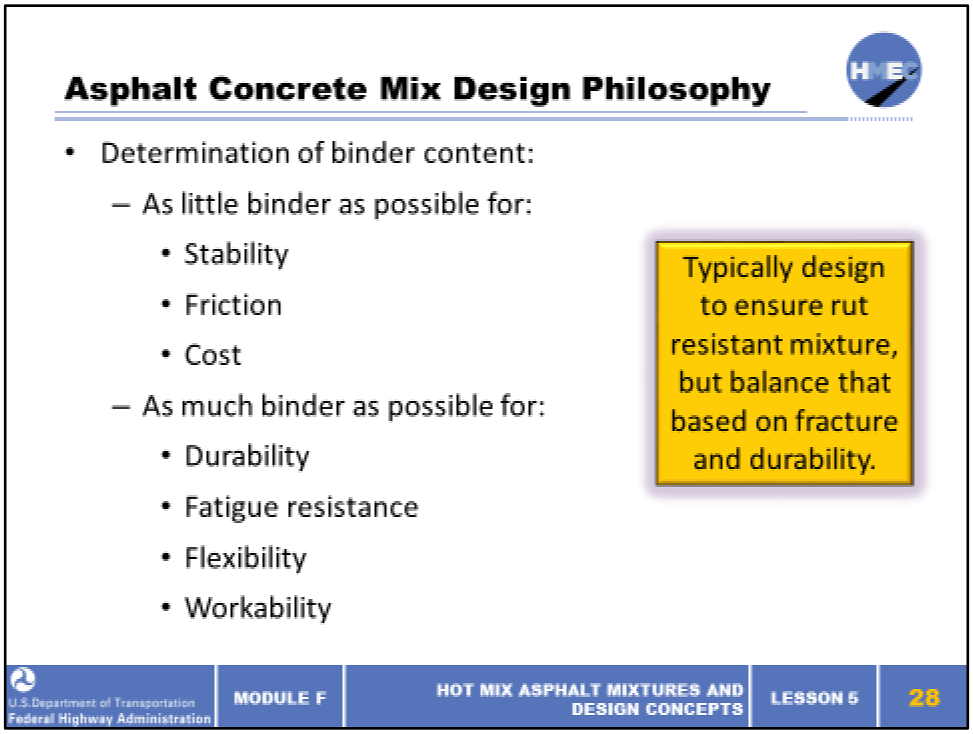
Slide 28


Asphalt Concrete Mix Design Philosophy
HMEC
• Determination of binder content:
– As little binder as possible for:
• Stability
• Friction
• Cost
– As much binder as possible for:
• Durability
• Fatigue resistance
• Flexibility
• Workability
Typically design to ensure rut resistant mixture, but balance that based on fracture and durability.
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HOT MIX ASPHALT MIXTURES AND DESIGN CONCEPTS
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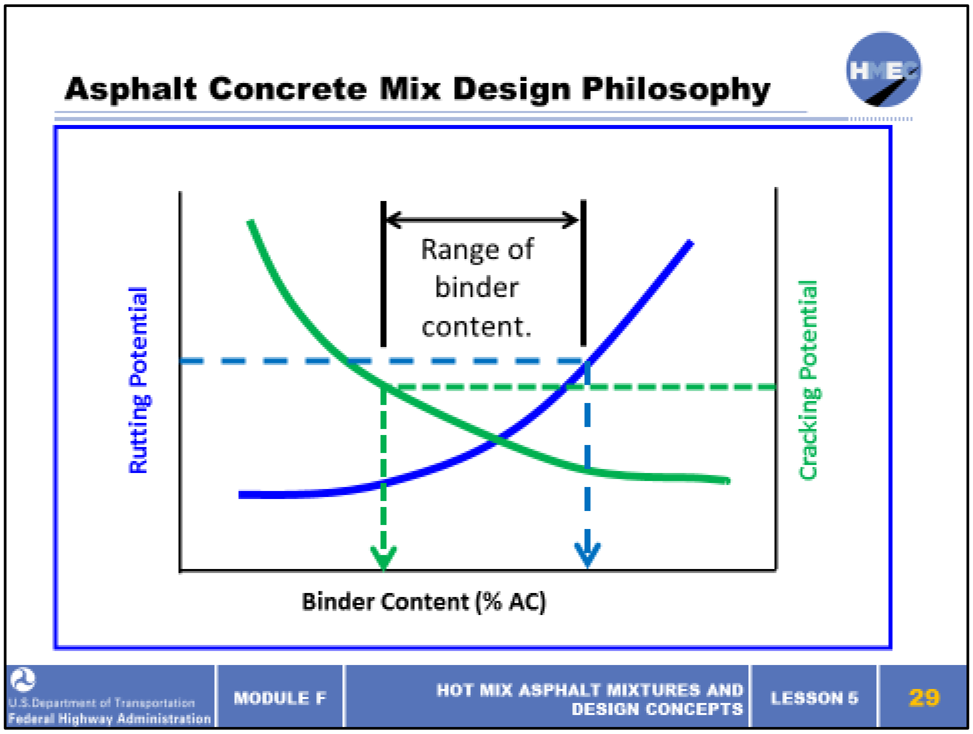
Slide 29



The target binder content is selected to minimize the amount of distress. Cracking distresses help define the limit for the lower binder content, while rutting or distortion distresses help define the limit for the higher binder content. Unfortunately, there are cases and aggregate types and blends where the lower limit based on cracking is higher than the higher limit based on rutting. This condition is very difficult to recognize without cracking and rutting tests used to actually optimize the mixture design process.

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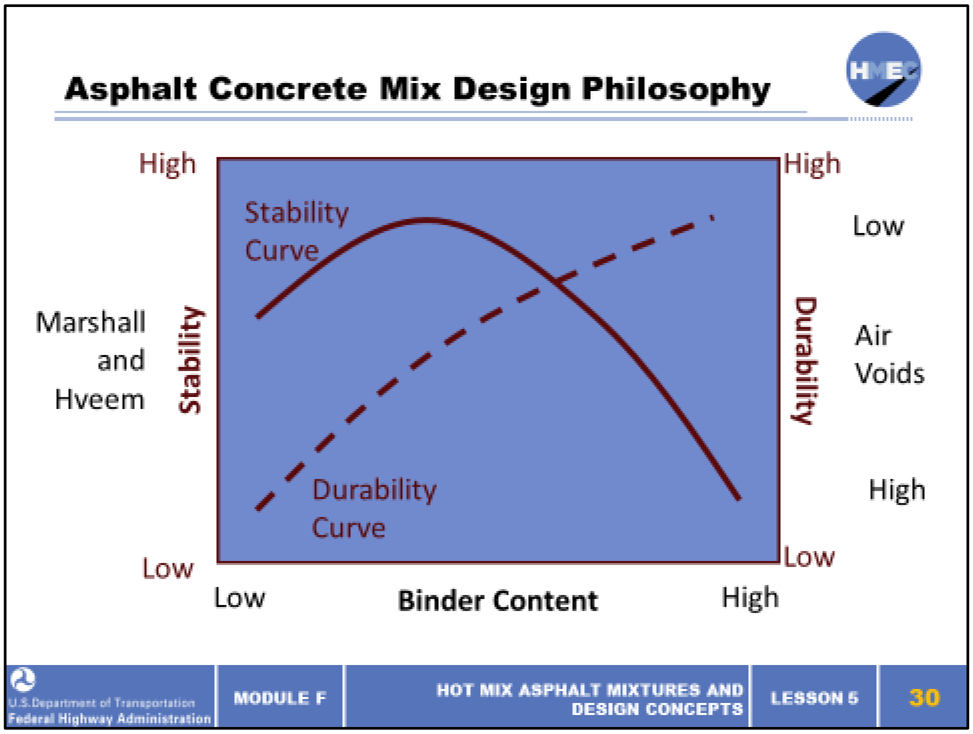
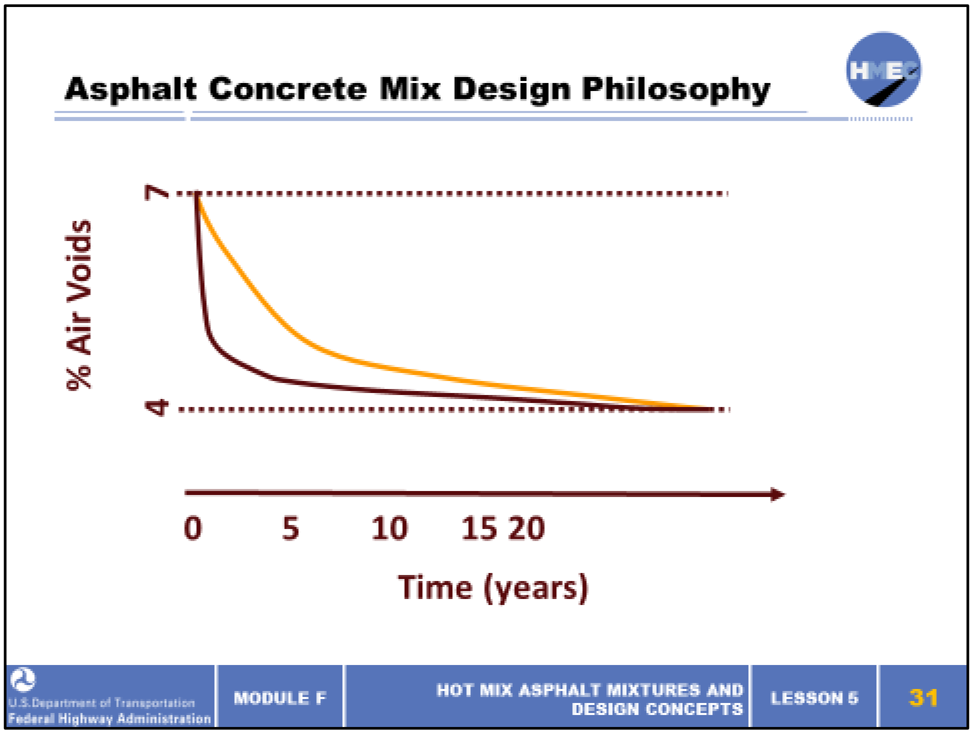
Slide 30



Maximum stability (i.e., resistance to permanent deformation) occurs in compacted HMA mixtures at binder contents that correspond to relatively low durability qualities (and fatigue resistance).

Selection of the "optimum" binder content requires a trade-off between good durability characteristics (and fatigue cracking resistance) and rutting resistance. However, the optimum binder content is difficult to determine or define when only volumetric properties are used within the mixture design process.

Slide 31



The additional densification that takes place is dependent on the amount and type of truck traffic, as well as on the asphalt properties or grade. The asphalt concrete mixture design philosophy or concept is to determine the amount of asphalt based on the initial compaction and future densification from truck traffic so that the air void level will be the same. The design air void level for most mixture design procedures is 4%.

Several unsuccessful efforts have been made to quantify the impact of traffic loading on in-place air voids.

It is simply assumed during the mixture design process that initial in-place air void level (typically 6–8%) will be reduced to the air void level at which the target binder content selection was made (3–5%) prior to the end of the service life of the pavement.

It is also assumed or hypothesized that the in-place air void level will be reduced below the initial in-place level to reduce aging and durability problems, but not below approximately 3% because a dense, well-graded mixture typically becomes very unstable (loses shear strength) below this air void level and/or may exhibit bleeding and flushing because there is an insufficient level of air void for the binder to occupy when it expands during the hotter months of the year.

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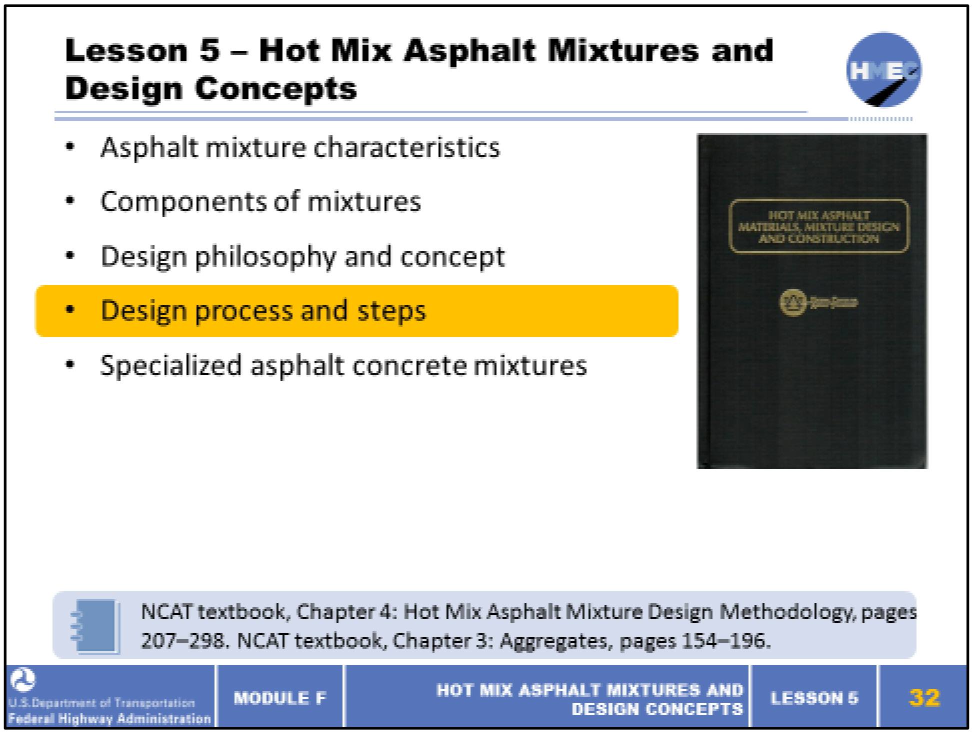
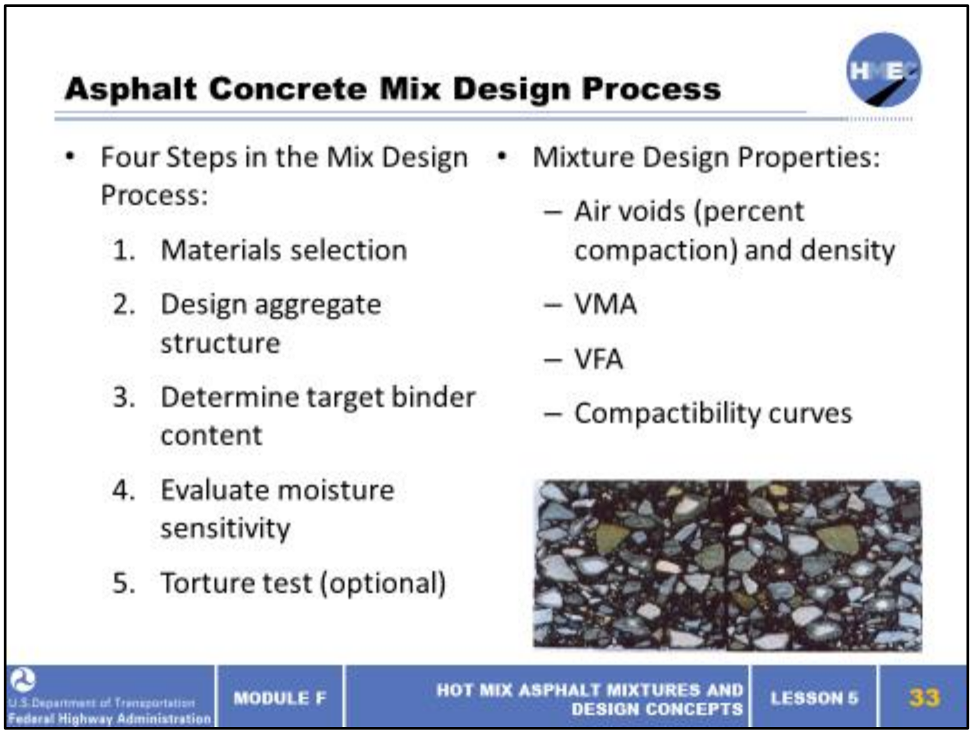
Slide 32



Note some of the information covered within this lesson and section comes from many different sources but was generally taken from Chapter 4, Hot Mix Asphalt Mixture Design Methodology, and Chapter 3, Aggregates, of the NCAT textbook and AASHTO test standards and specifications.

Slide 33



It is important to note that the conventional mix design methods essentially left materials selection up to the specifying agency. Superpave, on the other hand, utilizes a completely new system for testing, specifying, and selecting asphalt binders that was developed during SHRP. No new aggregate tests were developed, so current methods of selecting and specifying aggregates were refined and incorporated into the Superpave mix design system in the material selection step. These specification and inherent properties were defined and explained in Module D.

Superpave volumetric asphalt mixture requirements were established from currently used criteria. The materials selected in this step include a performance-graded binder, mineral aggregates, and modifiers and/or additives if necessary. It should be noted that very limited emphasis was placed on the use of modifiers during SHRP. Thus, efforts are currently underway to address the use of modifiers within the Superpave system.

The design aggregate structure step consists of initial development and evaluation of several trial blends.

In the design binder content step, the most desirable trial blend identified in step two is further evaluated to determine the optimum binder content for that blend.

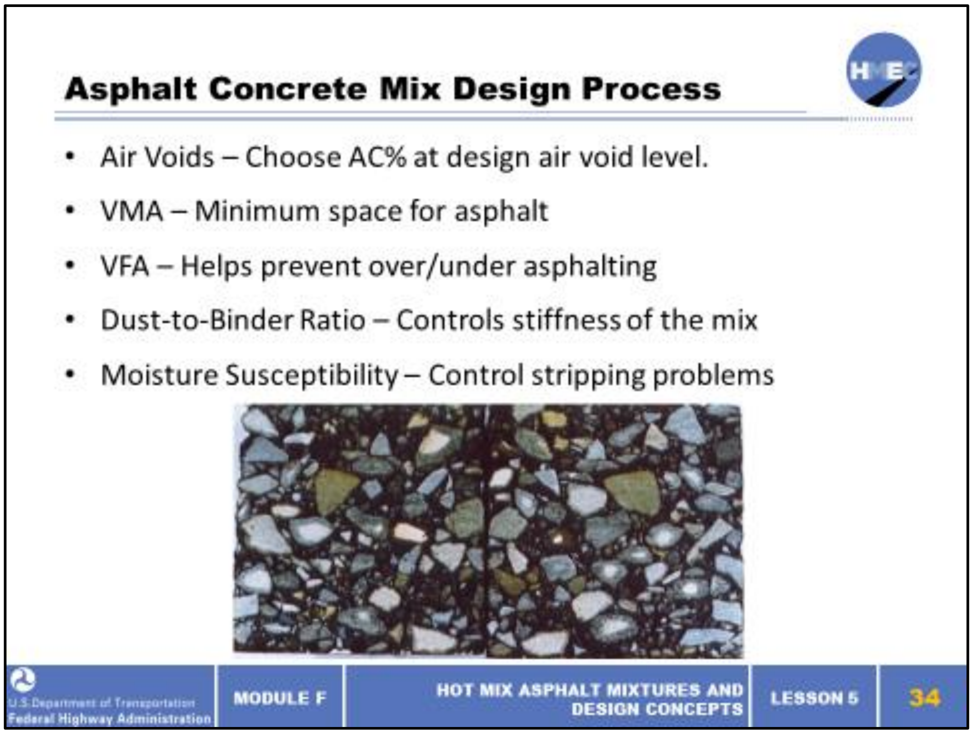
Optimum binder content is referred to as the design binder content in Superpave.

The final step is an evaluation of the mixture sensitivity to moisture, essentially using AASHTO T 283 (the modified Lottman procedure). Some agencies use the Hamburg wheel tracking test (see Lesson #6) to ensure the mixture will not exhibit excessive rutting in the presence of water.

Some agencies use or require a torture test to verify the final asphalt concrete mixture will exceed some minimum criteria relative to permanent deformation or rutting resistance. The more common torture tests used are the Hamburg wheel tracking test (also used to confirm the asphalt concrete mixture will be resistant to stripping or moisture damage, as noted above) or the Asphalt Pavement Analyzer.

At present, no strength or fracture and stability or distortion test is included in the mixture design procedure. This issue has been debated for years and there are performance (fracture and plastic deformation) tests that can be used but to date there is no consensus within industry on which test(s) should be used. Performance tests will be presented and discussed in Lesson #6.

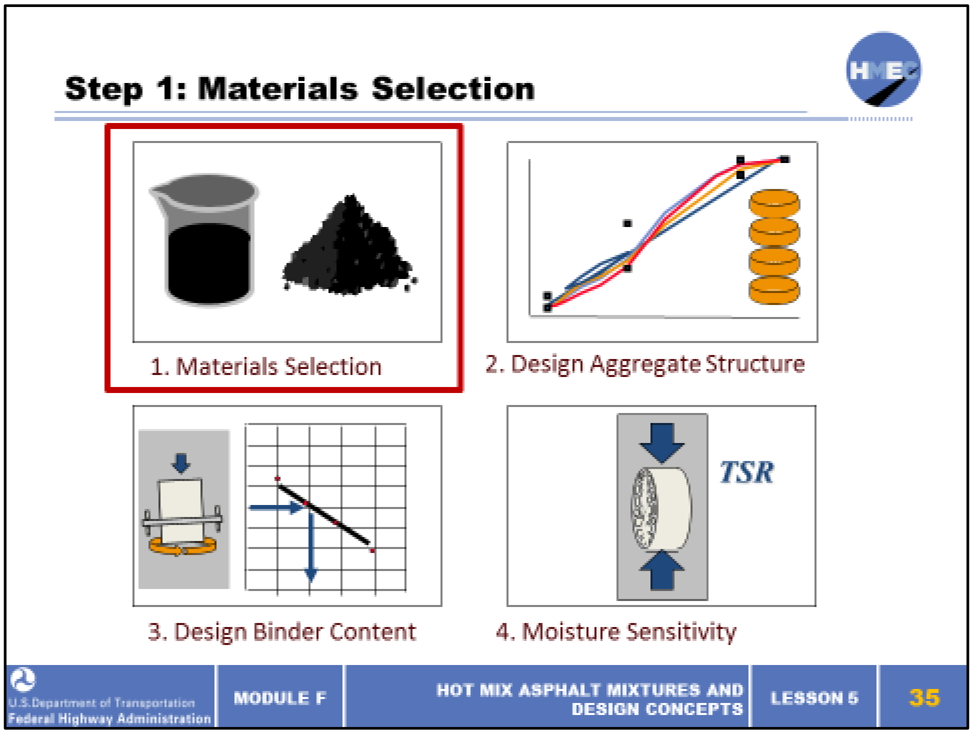
Slide 34



VMA is used to ensure there is sufficient space for the design binder content, while VFA is used to prevent against over and under asphalt mixtures. The dust to effective binder ratio is used to ensure the mix stiffness is not too high or low. Moisture susceptibility is measured to control and/or prevent stripping issues.

The dust-to-binder (DB) ratio is simply the percent passing the number 200 sieve divided by the effective binder by weight. The effective binder content by weight does NOT include the binder absorbed into the aggregate. Effective binder content was defined and calculated in Lesson #4.

Slide 35



There are two key features to the Superpave mix design method that are accomplished in four basic steps, which are:

1. Rigorous materials selection to be discussed in this section.
2. Volumetric design using the Superpave gyratory compactor (SGC), which was discussed in a previous section of this lesson.

Superpave volumetric asphalt mixture requirements were established from currently used criteria. The materials selected in this step include a performance graded binder, mineral aggregates, mineral filler, and modifiers and/or additives, if necessary. The performance-grade binder specification and selection process for project climate and traffic conditions was addressed within Lesson 3.

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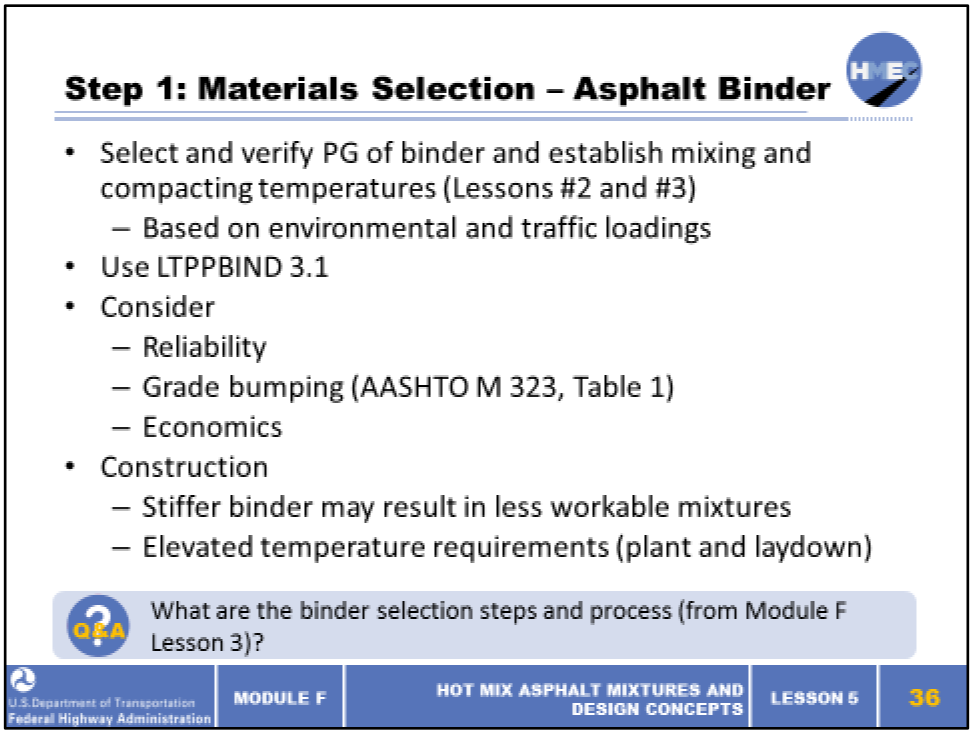
Slide 36



This information is used to select and evaluate potential binders and aggregates. The process for selecting the asphalt performance grade (PG) was discussed in Lesson 3 but is also summarized within this slide.

The easiest method of selecting a binder is to use the Superpave or the LTPPBind 3.1 software. It is further suggested that the Long Term Pavement Performance (LTPP) temperature algorithms, rather than the original SHRP algorithms, be employed.

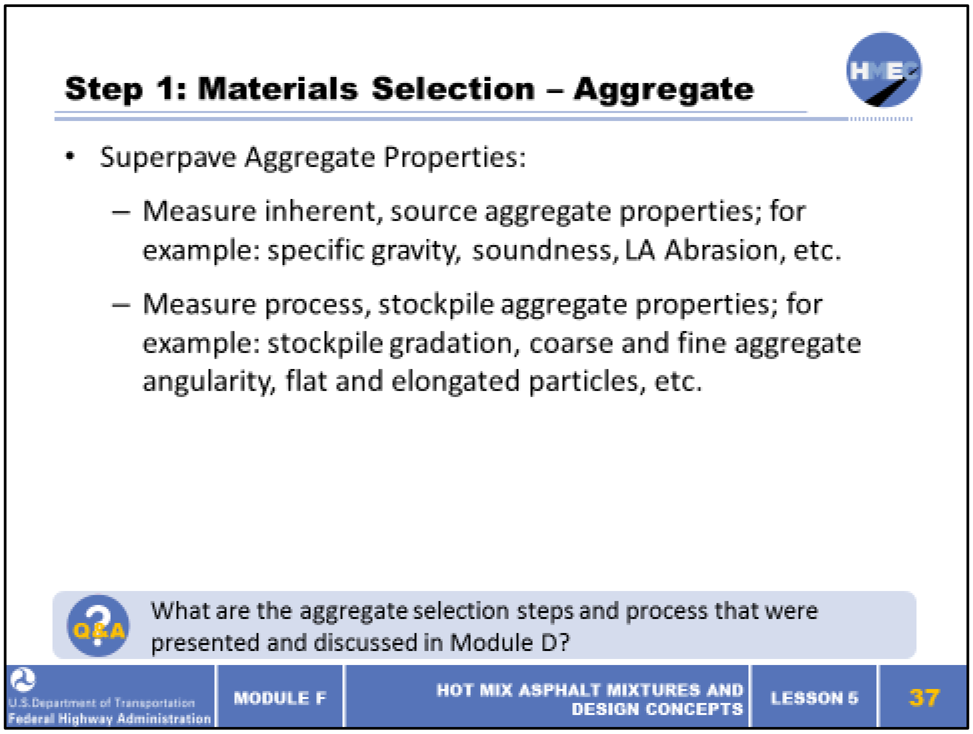
In either software, the user identifies one or more NOAA weather stations near the project location from a database. As discussed earlier, the required binder graded is determined.

In the selection process, reliability is considered along with the potential need for grade bumping to account for heavy traffic and/or slow moving loads.

Of course, economics should be considered by the mix designer in ultimately selecting a binder grade. It is also important for the mix design to consider the construction process when selecting a binder. Selection of a stiffer binder may result in a less workable mixture that many contractors are not accustomed to placing and compacting. There may also be the need to used elevated temperatures at the hot plant and for laydown to ensure adequate time for compaction.

Refer to Module F Lesson 3.

Slide 37



During SHRP, a group of 14 experts called the Aggregate Expert Task Group used a method called a Delphi method to develop aggregate specifications. The Delphi method relies on the opinion of experts rather than actual experimentation (testing) and analysis. SHRP researchers relied on the opinion of these experts and their own to identify two categories of aggregate properties to be used in the Superpave system: consensus properties and source properties. Aggregate properties for which experts were able to come to a consensus as well as to what the specification limits should be were used in step one for the materials selection for the Superpave mixture design process.

Slide 38

Step 1: Materials Selection – Aggregate

Source or Inherent Aggregate Criteria – Recommended Values

ESAL (10^6)	Los Angeles Abrasion (Max Loss) AASHTO T 96	Soundness (Max Loss for 5 cycles) AASHTO T 104	Clay Lumps (Max Allow., %) AASHTO T 112
<0.3	35 to 40%	10 to 20%	0.2 to 10%
0.3 to 3			
3 to <30			
≥30			

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Toughness is the percent loss of materials from an aggregate blend in the Los Angeles Abrasion test.

Soundness is the percent loss of materials from an aggregate blend during the sodium or magnesium sulfate soundness test. This test estimates the resistance of aggregate to weathering while in-service.

Deleterious materials are defined as the weight percentage of contaminants (undesirable materials) such as shale, wood, mica, and coal in the blended aggregate.

The specification limits placed on these tests are independent of traffic (ESALs). Specified values are established by state agencies, although FHWA does recommend some criteria. While these properties are relevant during the mix design process, they may also be used as source acceptance control.

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Slide 39

Step 1: Materials Selection – Aggregate

Process Aggregate Criteria

ESAL (10^6)	CAA; AASHTO T 326		FAA; AASHTO T 304, Method A		SE (%) Min; AASHTO T 176	FEP (%) Max; ASTM D4791
	≤ 100mm	≥100mm	≤100mm	≥100mm		
<0.3	55/-	-/-	-	-	40	-
0.3 to 3	75/-	50/-	40	40	40	10
3 <10	85/80	60/-	45	40	45	
10 to <30	95/90	80/75	45	40	45	
≥30	100/100	100/100	45	45	50	

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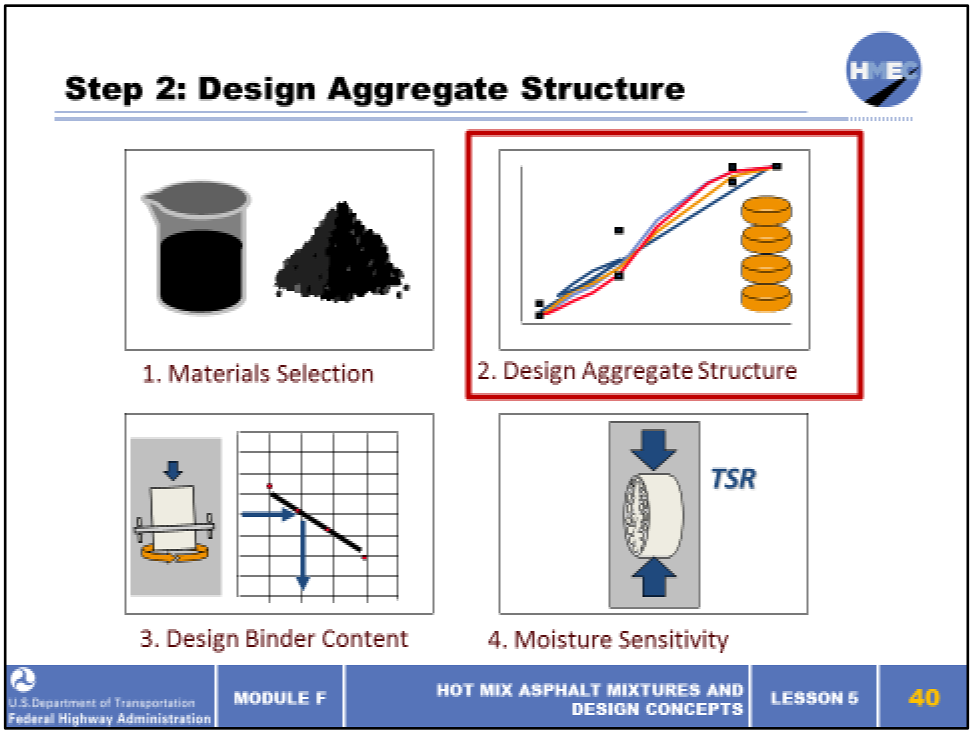
Refer to Table 4-4 on page 263 for the tests used to measure these properties which are:

• Coarse aggregate angularity (CAA – AASHTO T 326; ASTM D 5821) – the coarse aggregate angularity determines the percentage of coarse aggregate particles with two or more fractured faces;
• Fine aggregate angularity (FAA – AASHTO T 304 method A) – the fine aggregate angularity is the void content in a fine aggregate column of material deposited in a container from a specified height, the higher the value the higher the void content and the more angular the fine aggregate;
• Flat and/or elongated particles (FEP – ASTM D 4791) – the percentage of coarse particles that are flat or rectangular with a maximum to minimum dimension ratio of more than 3 to 1. However, some agencies use a dimension ratio of 3 to 1 while others use a ratio of 5 to 1 to define FEP; and
• Sand equivalent (SE – AASHTO T 176; ASTM D2419) – the relative proportion of sand to clay like or plastic fines and dust or the sand reading divided by the clay reading, the higher the SE value the cleaner the lower the amount of clay in the aggregate.

The process aggregate property specifications are included in this slide, which are a function of design traffic or design traffic and the depth below the pavement surface at which the material will reside.

Slide 40


Step 2: Design Aggregate Structure
HMEC
1. Materials Selection
2. Design Aggregate Structure
3. Design Binder Content
TSR
4. Moisture Sensitivity
U.S. Department of Transportation
Federal Highway Administration
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LESSON 5
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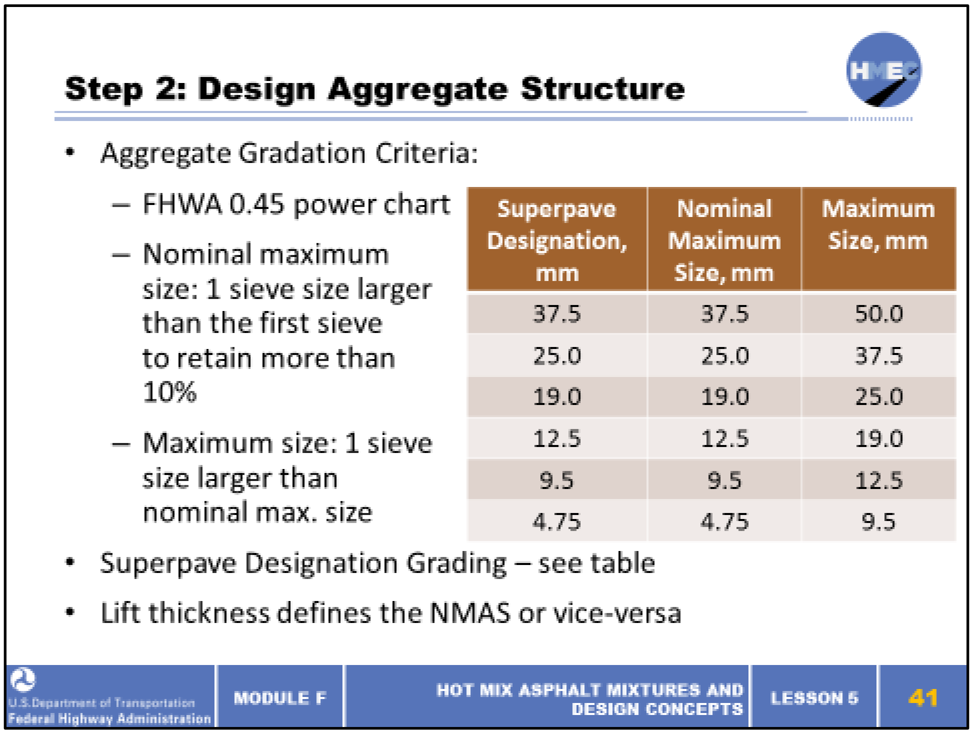
Slide 41



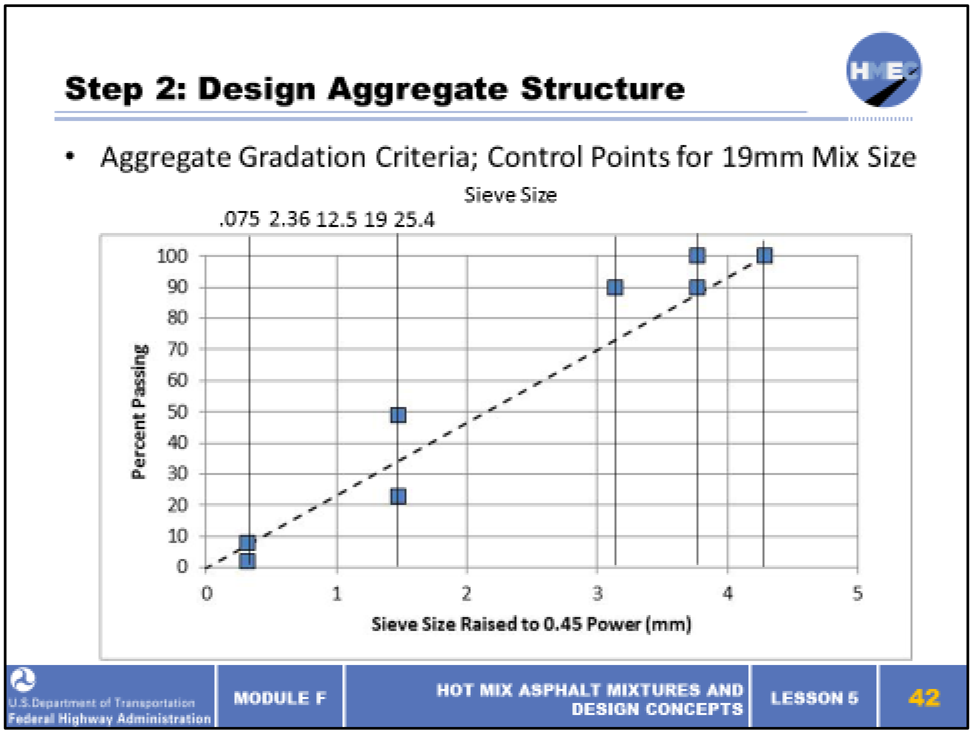
There are five nominal maximum sizes: 37.5, 25.0, 19.0, 12.5, and 9.5mm. Nominal maximum aggregate size (NMAS) is defined as one sieve size larger than the first sieve to retain more than 10%. The maximum size is one sieve size larger than the nominal maximum size.

The Superpave aggregate property specifications listed and defined in previous slides strive to improve internal mixture friction and thus rutting performance. As a result, these mixtures may be less workable than mixtures designed without these aggregate requirements.

This can contribute to field compaction problems for contractors unfamiliar with these mixtures. These dense, well-graded mixtures are compactable, but may have to be treated differently than mixtures that do not meet the aggregate criteria listed in previous slides.

The lift thickness is used to define the nominal maximum aggregate size (NMAS) for establishing the gradation.

Slide 42



Refer to table 4-5 on page 266 and figure 4-11 on page 264 in the NCAT textbook. The control points vary for different size mixtures. The control points for different mix sizes are included on the next slide. The control points are used to try and ensure that there is a sufficient amount of asphalt binder in the mixture for durability. For a specific mix size of designation, four upper and four lower values are used to define the control points for designing the aggregate structure. This is explained in more detail in the following slide for five mix designations.

The design aggregate structure step generally requires the establishment of three trial blends (these are mathematical or theoretical aggregate blends). The trial blends are established by selecting different gradations based on the nominal maximum size aggregate and the associated control points. The NCAT textbook provides the control points for different size mixtures (see Tables 4-6 through 4-10 in Chapter 3 on Aggregates). The trial gradations should be located between each of the control points for the appropriate mix size to meet the Superpave requirements.

The aggregate properties for each blends or gradation are estimated (calculated, not measured) using the relationships that were given in Lesson 4. The estimated values are then compared to the specification criteria for initial compliance prior to preparing and comparing the test specimens.

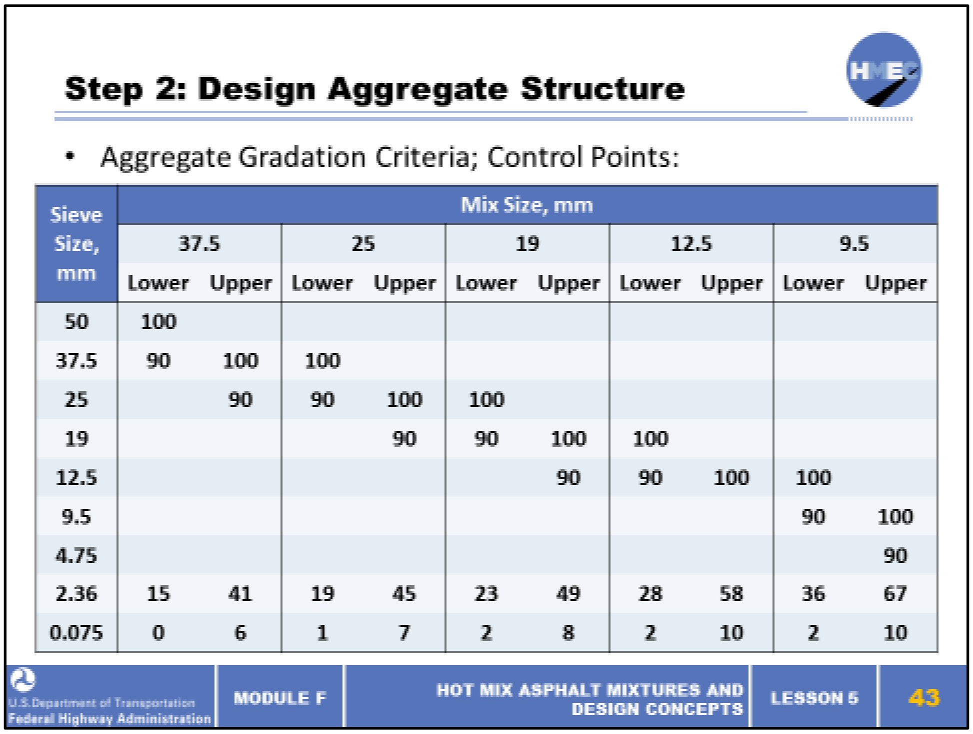
An initial asphalt content (P_{bi}) is then estimated for each trial blend. These estimates are made based on the calculated aggregate properties, which are listed on the next slide. Two specimens are then batched and compacted per trial blend at the initial or trial asphalt content (P_{bi}). There are two methods of estimating the P_{bi} for each trial blend, but it is important to note and recognize that this is simply a starting point for evaluating different blends.

1. It can be estimated from experience, particularly if the mix designer has a long history of use with the aggregate source and blend. This is the more simplistic approach but requires historical data.
2. If historical data are unavailable, P_{bi} can also be estimated based on aggregate properties of the individual stockpiles or aggregates. For this technique, effective aggregate specific gravity (G_{se}) is estimated from the bulk and apparent aggregate specific gravities using an equation which was provided in Lesson 4. The volume of absorbed asphalt (V_{ba}) is also estimated using the equation provided in Lesson 4. With these two estimates, the effective volume of asphalt (V_{be}) can be calculated and finally P_{bi} is calculated.

A volumetric analysis of the compacted specimens is then conducted to identify a most appropriate blend from the three considered.

In the third step of the Superpave volumetric mix design process, specimens are actually compacted over a range of asphalt contents, which will be presented and discussed after Step 2.

Slide 43



Sieve Size, mm	Mix Size, mm									
	37.5		25		19		12.5		9.5	
	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper
50	100									
37.5	90	100	100							
25		90	90	100	100					
19				90	90	100	100			
12.5						90	90	100	100	
9.5									90	100
4.75										90
2.36	15	41	19	45	23	49	28	58	36	67
0.075	0	6	1	7	2	8	2	10	2	10

An example of the control points for a 19 mm size mix was shown on the previous slide. The following lists the lower and upper control points for five different size mixtures. The instructor should simply note the critical sieve sizes used for setting the control points for a specific mix size and refer back to the previous slide as an example relative to the maximum density line.

The values for the control points were taken from AASHTO MP 2, Standard Specifications for Superpave Volumetric Mix Design. The control points for each mix designation are composed of four lower and four upper limits on the gradation. The four upper control points of each mix designation included in this slide are a result of the definition of nominal maximum and maximum sieve size, as defined in an earlier slide. The four lower control points for each mix designation limits the amount of sand in the mixture to exclude sand type asphalt mixes. The four lower control points also ensure an adequate amount of sand is contained in the mix to ensure a dense graded mixture.

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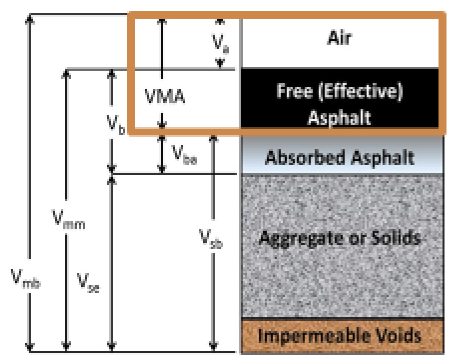
Slide 44

Step 2: Design Aggregate Structure

- Voids in Mineral Aggregate (VMA) Design Criterion:

NMAS, mm	Minimum VMA, %
9.5	15.0
12.5	14.0
19	13.0
25	12.0
37.5	11.0



Q&A Why does the VMA requirement drop as the aggregate size increases?

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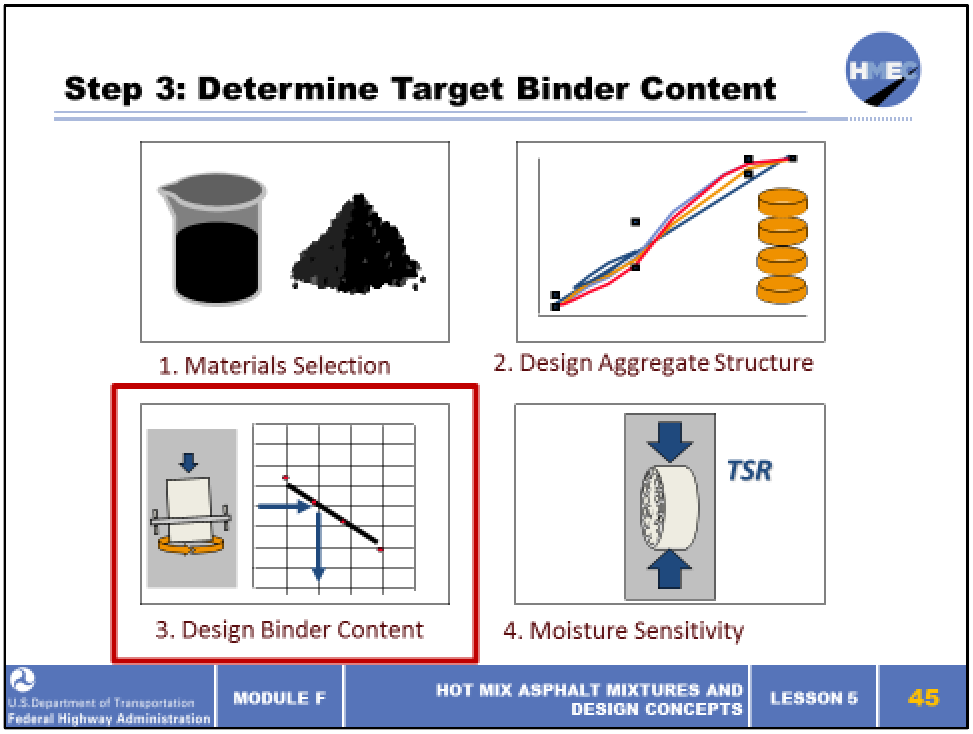
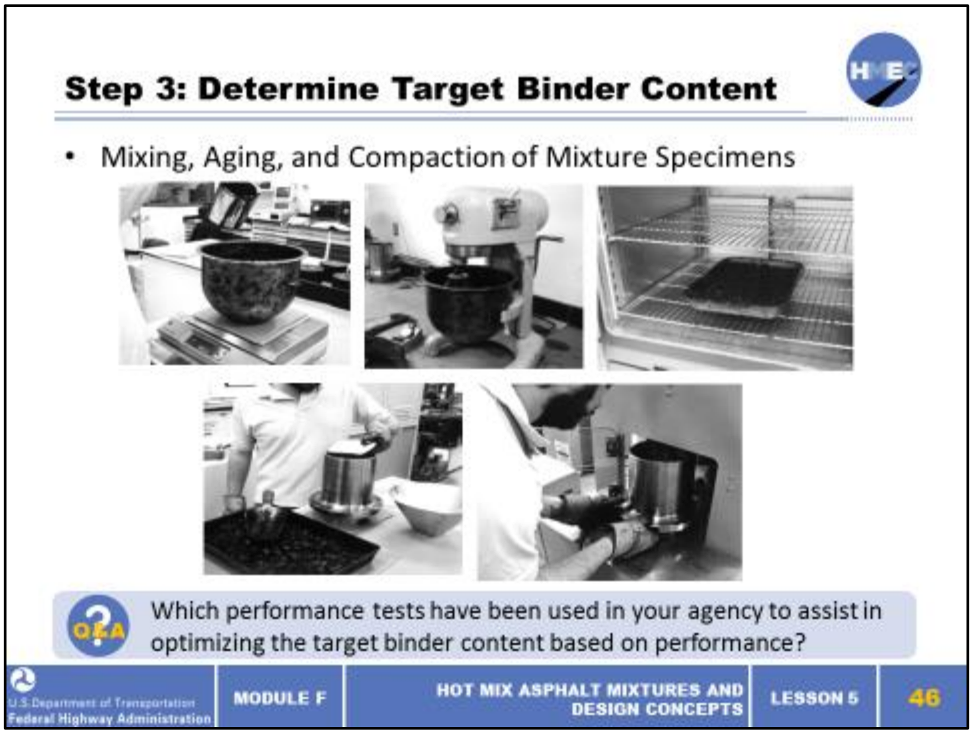
As the aggregate size increases, the VMA requirement drops. Refer to table 4-8 on page 273 of the NCAT textbook.

Slide 45



This section addresses Step 3: Design binder content.

Slide 46



The specimens are mixed at specific viscosity and then short term oven aged at the compaction temperature. The aging of the mix is to simulate the action of the asphalt concrete mixture going through the plant production and laydown process. The key issue with the aging is that asphalt absorption takes place and this directly affects volumetric properties.

Superpave requires all samples mixed and oven-aged at compaction temperature for 2 hours prior to compaction. This is required to:

- Allow time for aggregate to absorb asphalt; and
- Help minimize variability in volumetric calculations.

Most terms are dependent upon volumes which change with changes in the amount (volume) of absorbed asphalt.

After short-term aging, specimens are compacted using the SGC using the N_{design} compactive effort.

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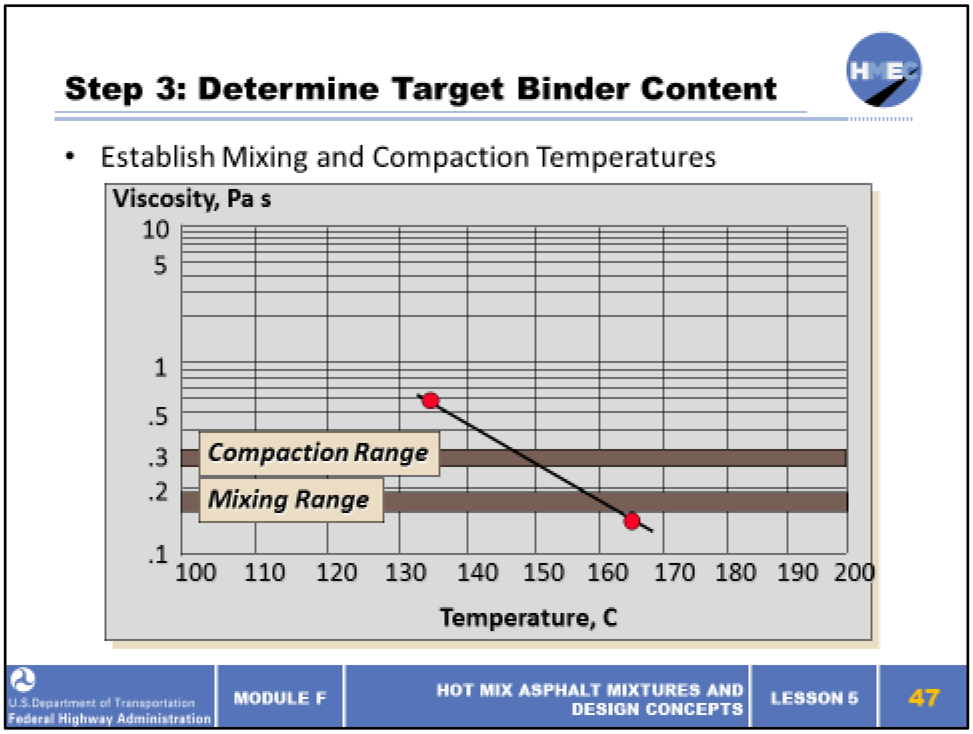
Slide 47



The red dots represent viscosity measurements determined at 135 °C and 165 °C. A straight line is then drawn between the two points to represent the mixing and compaction temperature zones.

For some modified binders, the viscosity-temperature chart may not always work or be appropriate for determining the proper mixing and compaction temperature for highly modified binders. In those cases, use of the modifiers recommended mixing and compaction temperatures may need to be used.

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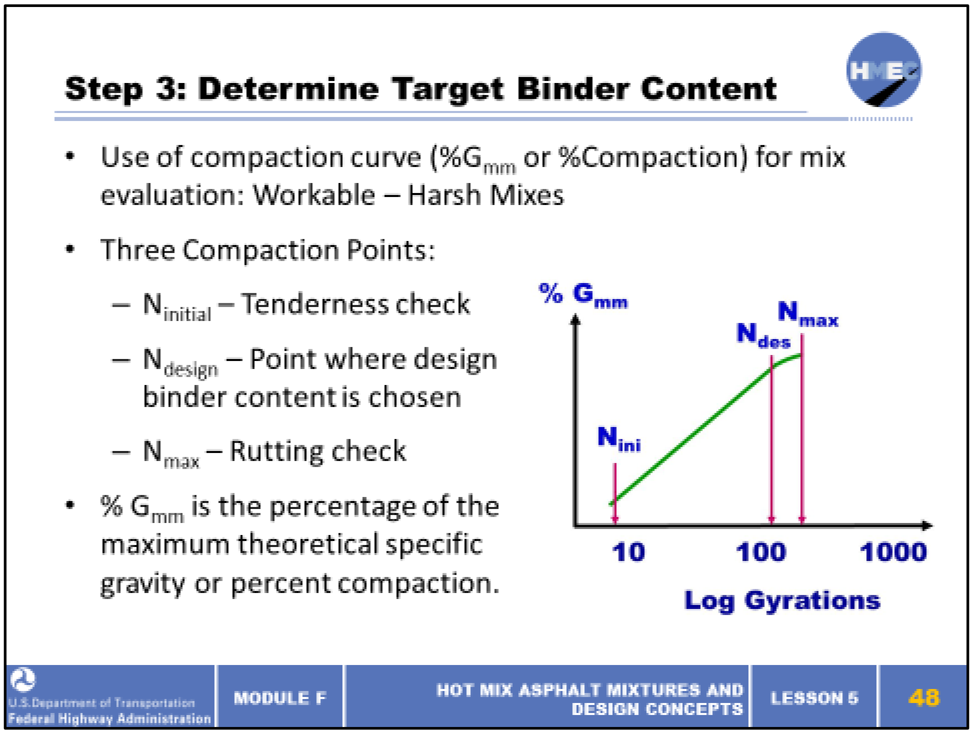
Slide 48



As shown in this slide, there are three points of interest on the SGC relationship - $N_{initial}$, N_{design} and N_{max}. During compaction, the percent compaction is determined through height recordings on the specimen densification in the gyratory mold.

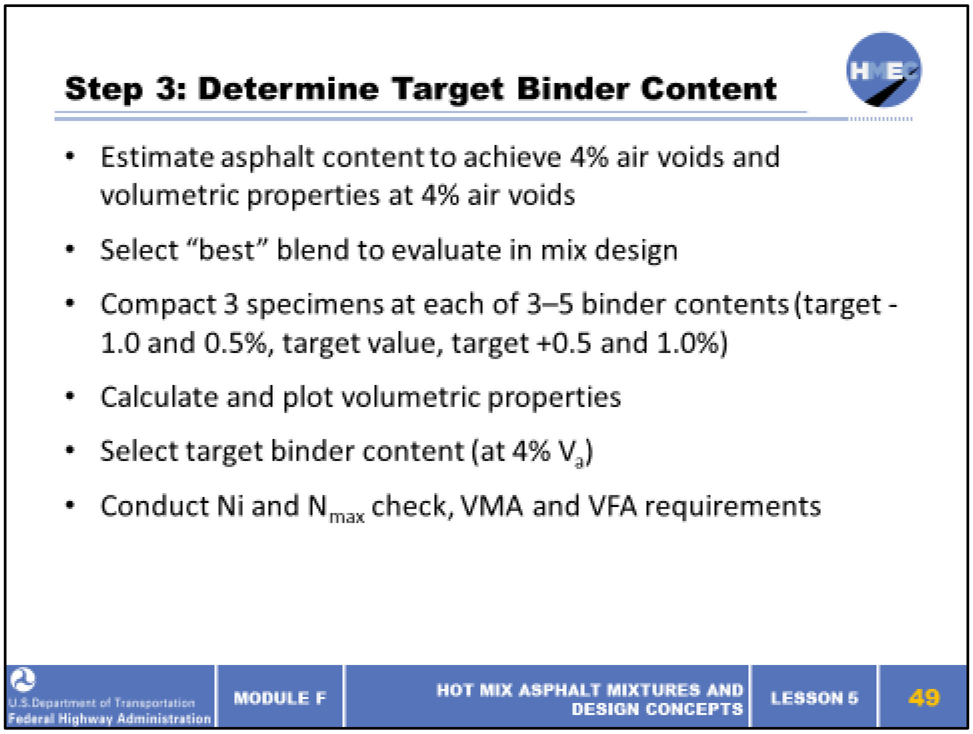
• $N_{initial}$ – This parameter is defined as the tenderness check, a minimum compactive effort is needed to achieve this percent compaction. It is the number of gyrations to identify whether the mixture is too soft or compacts too easily and is susceptible to rutting and shoving. Thus, the percent G_{mm} for $N_{initial}$ is considered a "maximum" to avoid tender mixes. Percent G_{mm} is a function of traffic level, which will be presented in a latter slide.

• N_{design} – This parameter is defined as the point or percent compaction where the design binder content is selected.

• N_{max} – This parameter is defined as the rutting check. It is the number of gyrations for which any additional densification is minimized. It has also been called or termed the equilibrium or refusal air void level. Thus, the percent G_{mm} for N_{max} is considered a “maximum” to avoid rutting prone mixtures. Percent G_{mm} is a function of traffic level, which will be presented in a latter slide In the mix design process, specimens are compacted to N_{design} gyrations for volumetric analysis in accordance with AASHTO T 312. N_{max} was initially or originally used, because it provided the entire compactibility curve. However, most agencies have limited the number of gyrations to N_{design}. The information or data for each aggregate blend includes:

- Specimen height is recorded as a function of gyrations and is standard SGC compactor output (to a printer or directly to software);
- The bulk specific gravity of each compacted specimen is measured in accordance with AASHTO T 166; and
- The theoretical maximum specific gravity of a loose mix sample is also measured in accordance with AASHTO T 209.

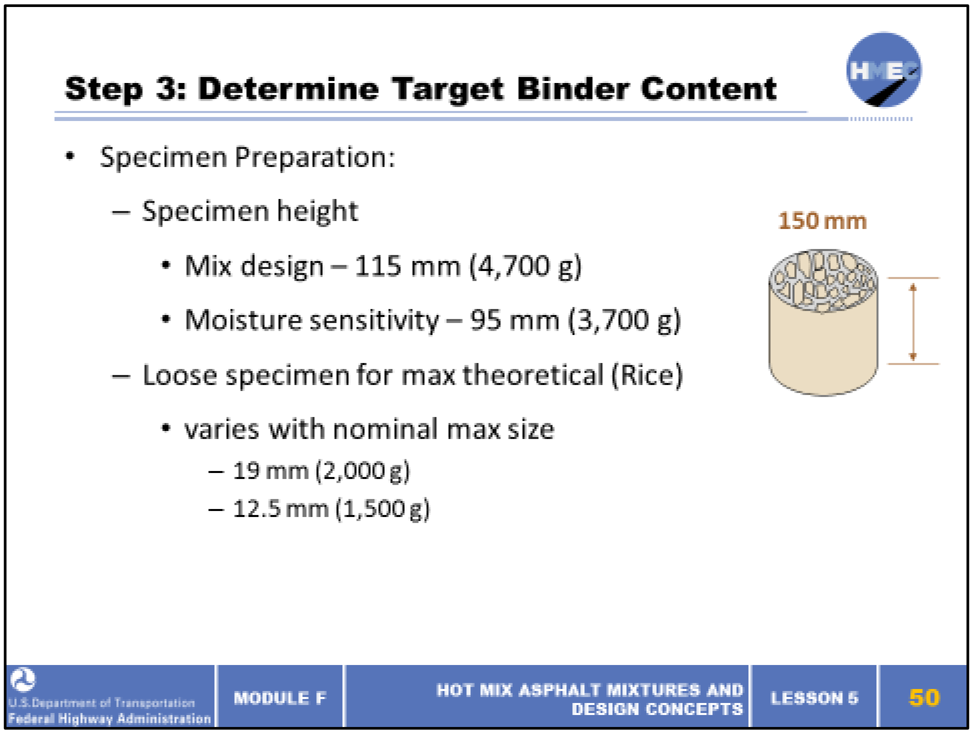
Slide 49



This is a significant difference from traditional mix design methods. Essentially, three potential blends are evaluated and the best blend is selected for binder content optimization. Once the best blend has been identified, two specimens are compacted at each of four binder contents. Volumetric properties are determined and plotted.

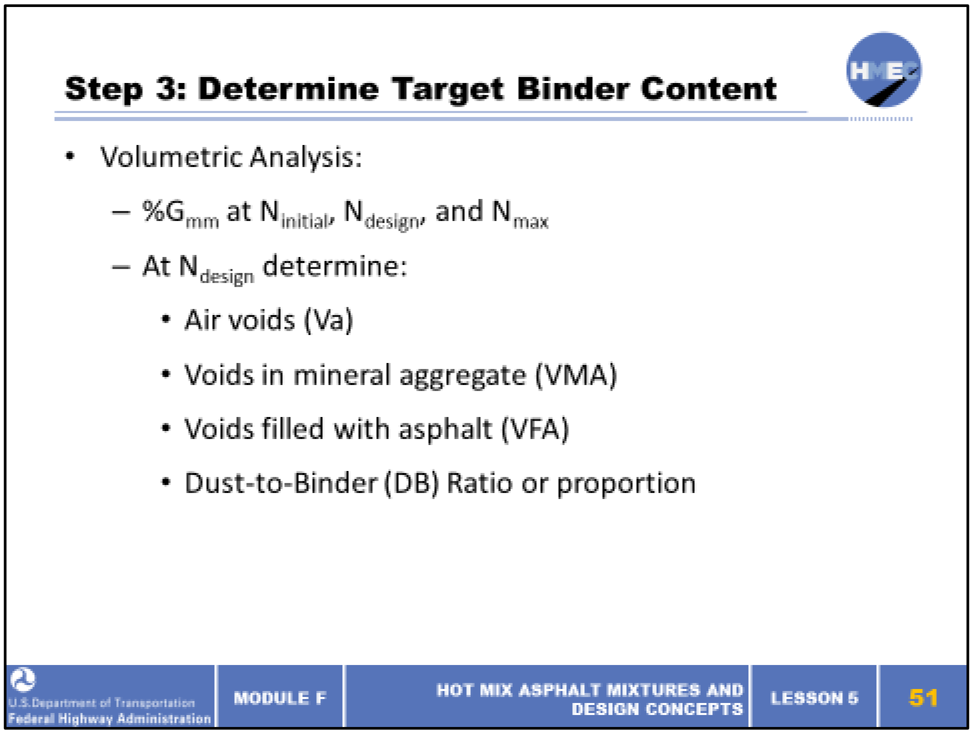
The design binder content is identified at 4% air voids and all other volumetric properties are checked for specification compliance. If all properties are in specification, then a pair of specimens is compacted at the design binder content to N_{max} gyrations to check that the air voids are greater than 2%.

Slide 50



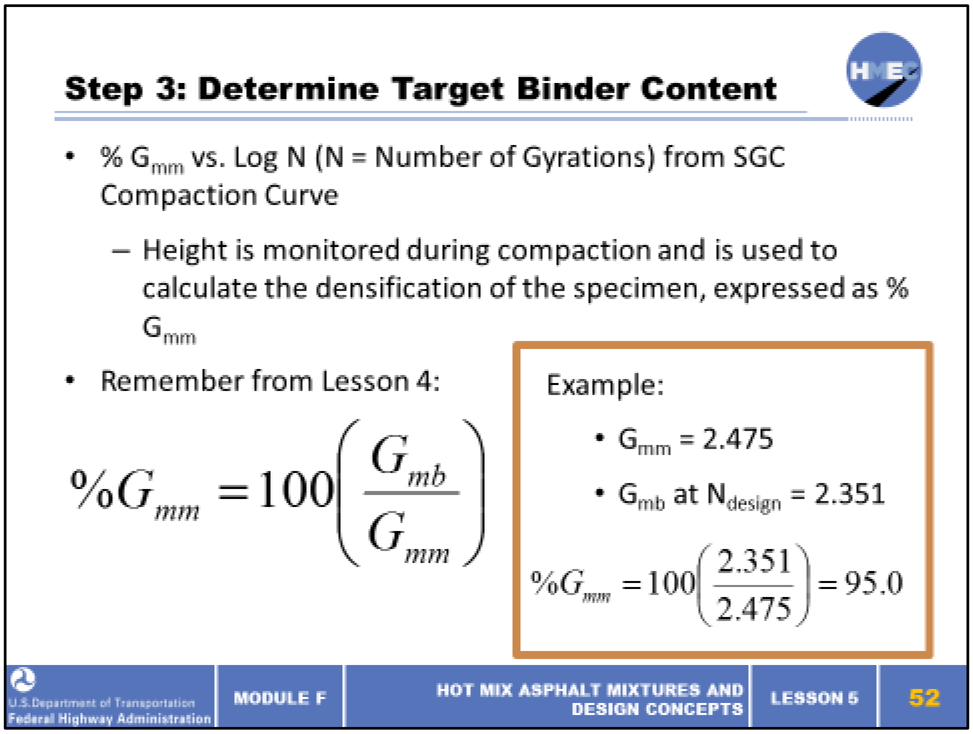
• Mix design specimens must be 115 ±5 mm tall. If a compacted specimen is outside the acceptable range, linear proportions may be used to adjust the aggregate mass to achieve the specified specimen height. Moisture sensitivity specimens must be 95 mm tall and typically require about 3,500 grams of aggregate.
• Loose mixture is also required for theoretical maximum specific gravity (Rice) determinations. The minimum sample size is a function of nominal maximum aggregate size.

Slide 51



Percent compaction at $N_{initial}$ and N_{design} are determined along with the percent air voids ($\%V_a$), voids in mineral aggregate (VMA), voids filled with asphalt (VFA), and dust proportion (DP).

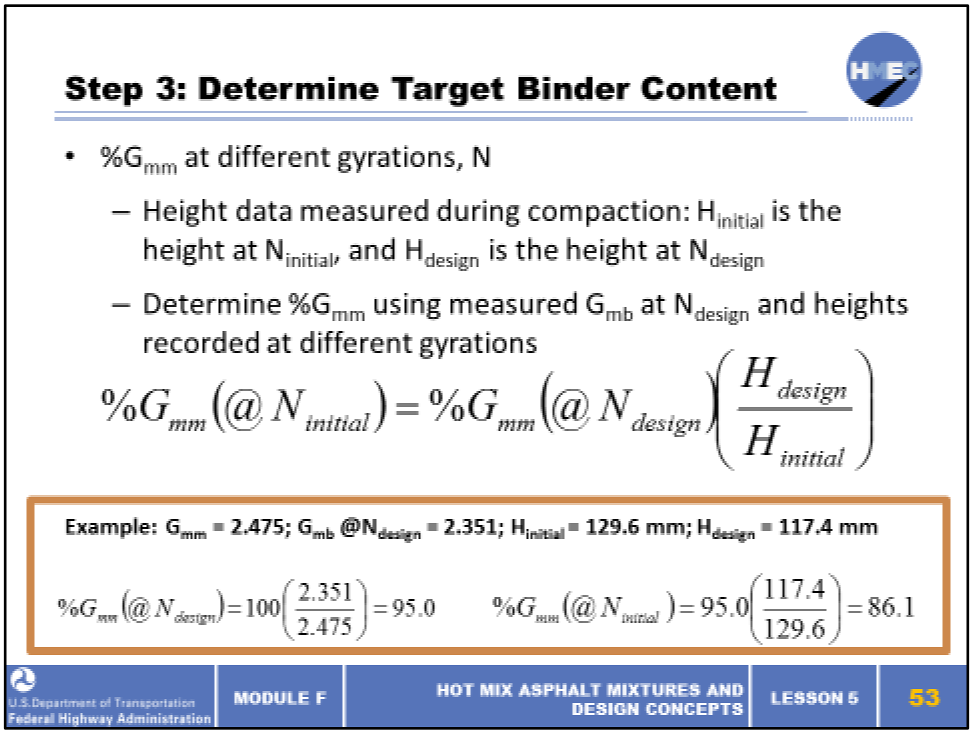
Slide 52



Percent compaction is expressed as a percentage of theoretical maximum specific gravity (%G_{mm}). It is simply the ratio of the compacted mixture bulk specific gravity (G_{mb}) to the Rice or maximum theoretical specific gravity measured in accordance with AASHTO T 209 times 100.

This slide shows an example calculation of determining % G_{mm} from the gyratory compaction data or compactibility curve.

Slide 53



The G_{mb} at $N_{initial}$ cannot be actually measured because the number of gyrations are continued throughout the compaction process. In other words, the compaction process is not stopped at $N_{initial}$. Thus, the $\%G_{mm}$ is estimated based on the height measurements at $N_{initial}$ and N_{design} and G_{mb} measured after the specimen has been compacted and extracted from the mold.

As a reminder:
$\%G_{mm}$ (at N_{design}) = 100*($G_{mb\ (design)}$/G_{mm})

This slide shows an example calculation of determining % G_{mm} at $N_{initial}$ from the gyratory compaction data or compactibility curve.

Slide 54

Step 3: Determine Target Binder Content

- N_{design} and N_{max} Table

Traffic Level; MESALs	Compaction Level				
	Ninitial		Ndesign	Nmaximum	
	Gyrations	%Gmm		Gyrations	%Gmm
<0.3	6	<91.5	50	75	≤98.0
0.3 to <3.0	7	<90.5	75	115	≤98.0
3.0 to <30.0	8	<89.0	100	160	≤98.0
>30.0	9	<89.0	125	205	≤98.0

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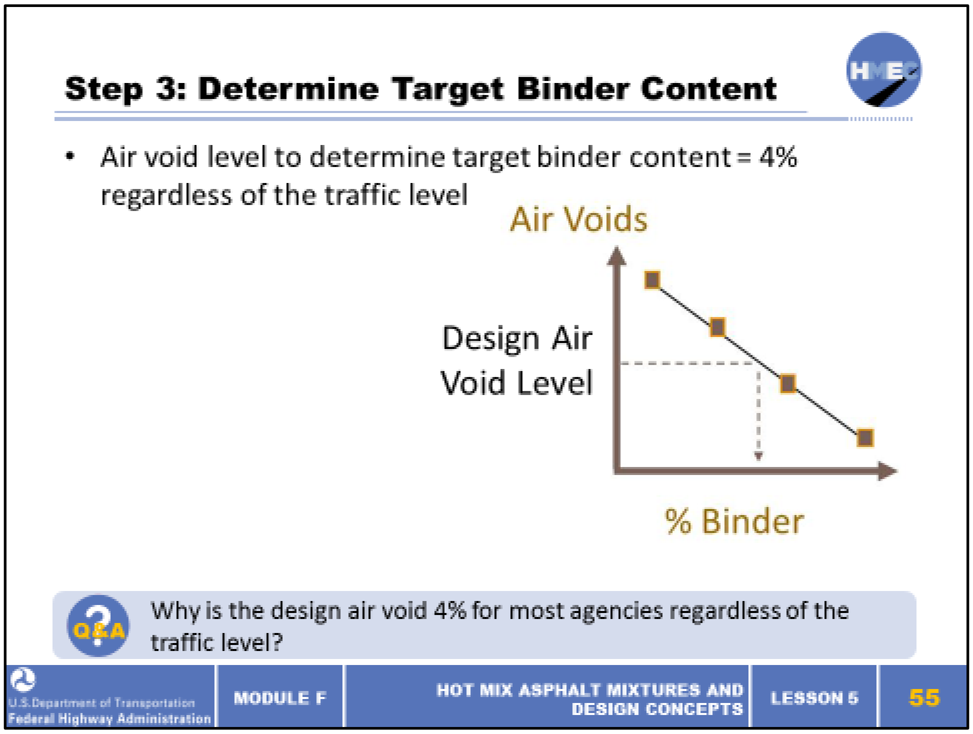
Slide 55



Some agencies, like the Arizona DOT, use a different design air void level. Arizona DOT uses 5% air voids to select the design binder content, which will result in lower target binder contents. The higher design air void level and lower design binder content was selected because of the hot climate, which results in higher levels of distortion and rutting or bleeding.

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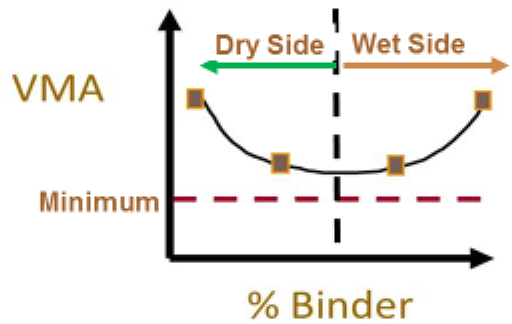
Slide 56

Step 3: Determine Target Binder Content

- Voids in Mineral Aggregate (VMA) Design Criterion is an important volumetric property:

NMAS, mm	Minimum VMA, %
9.5	15.0
12.5	14.0
19	13.0
25	12.0
37.5	11.0



Q&A Do not select target binder content on the "wet side" of VMA curve. Why?

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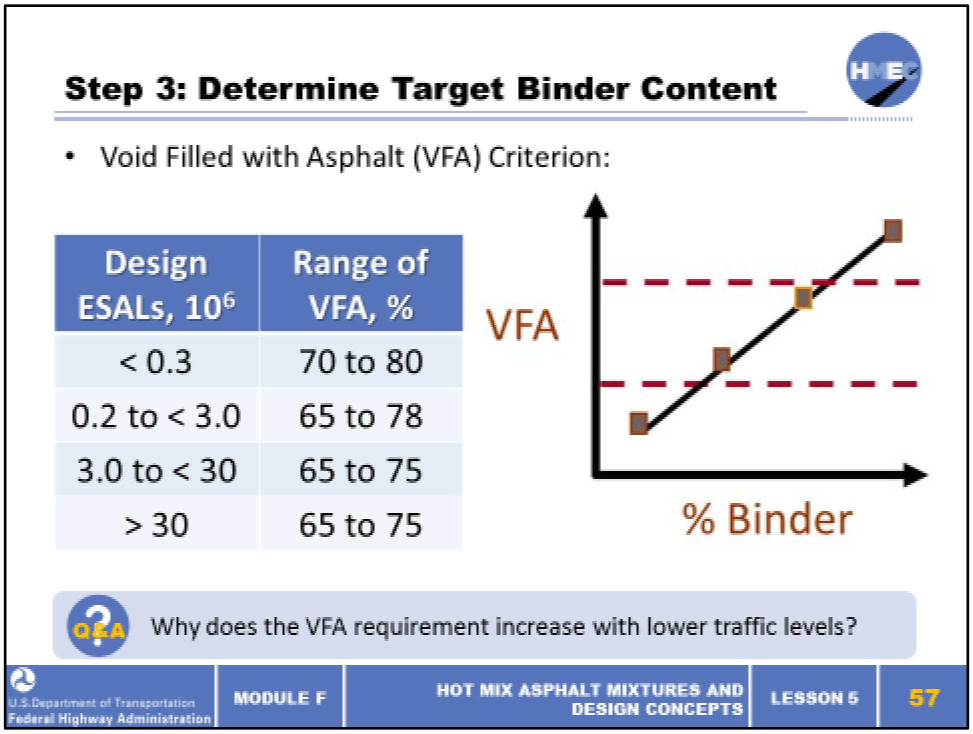
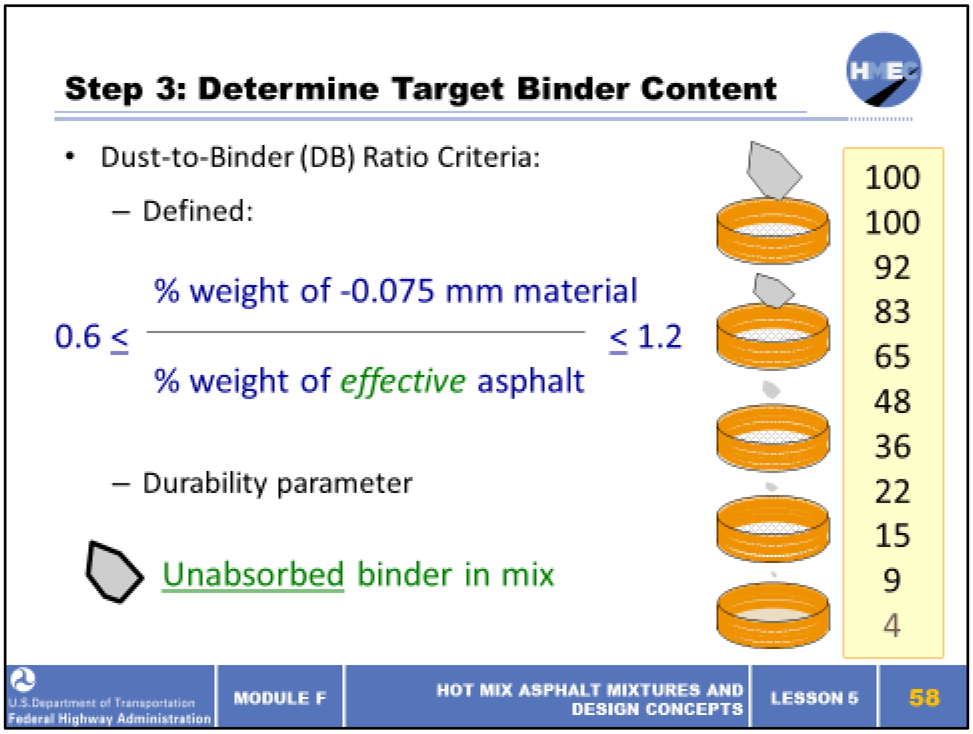
Though it is not explicitly stated in the Superpave design method, design (optimum) binder content should not be selected on the wet side of the VMA versus binder content curve, as noted below. Compacting on the wet side of the VMA curve can result in "flushing" in field, where too high of binder content results in compacting to a very low air void level, "pushing" the binder out of the aggregate skeleton.

Slide 57



This slide shows the range of allowable VFA values for different truck volumes or design traffic levels.

Slide 58



This requirement is included to restrict the amount of fines and thus to stay away from harsh or much stiffer mixes.

AASHTO M 323-13 in Table 6 provides the dust-to-binder ratio criterion is 0.6 to 1.2, as shown in the slide. However, when the gradation passes below the Primary Control Sieve control point the dust-to-binder ratio criterion can be increase from 0.6 to 1.2 to 0.8 to 1.6, which is stated in Note 9 of M 323-13.

Slide 59

Step 3: Determine Target Binder Content

- In Summary, Volumetric Design Criteria (AASHTO M323)

Traffic	%Gmm Criteria			VMA	VFA	DB
ESAL (10^6)	N_{ini}	N_{des}	N_{max}			
<0.3	≤91.5	=96.0	≤98.0	See Table for NMAS	70-80	0.6 to 1.2
0.3 to 3	≤90.5				65-78	
3 to < 30	≤89.0				65-75	
≥ 30	≤89.0					

NOTE: Some States may have modified these parameters (i.e., NYSDOT: N_{des} = 96.5%)

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VMA is a function of nominal maximum aggregate size (NMAS), and percent compaction at the design and maximum number of gyrations, as well as dust-to-binder ratio or proportion, have fixed criteria. This slide summarizes the volumetric design criteria that was discussed in previous sections of this lesson and included in AASHTO M 323.

As noted in the previous slide, however, when the gradation passes below the Primary Control Sieve control point the dust-to-binder ratio criterion can be increase from 0.6 to 1.2 to 0.8 to 1.6, which is stated in Note 9 of M 323-13.

Slide 60

Step 3: Determine Target Binder Content

- Using the SGC to determine target binder content:
  1. All samples compacted to N_{design} or N_{max}
  2. Voids or $\%G_{mm}$ are determined by machine calculations at $N_{initial}$, N_{design}, and N_{max}
  3. Bulk density of specimen is determined from sample weight and sample volume (calculated by machine)
  4. Determine actual bulk density
  5. Determine correction factor, CF
  6. Using Gmm and G_{mb}, the void level can be determined for various compaction levels to ensure project criteria are met

$$CF = \frac{ActualDensity}{CalculatedDensity}$$

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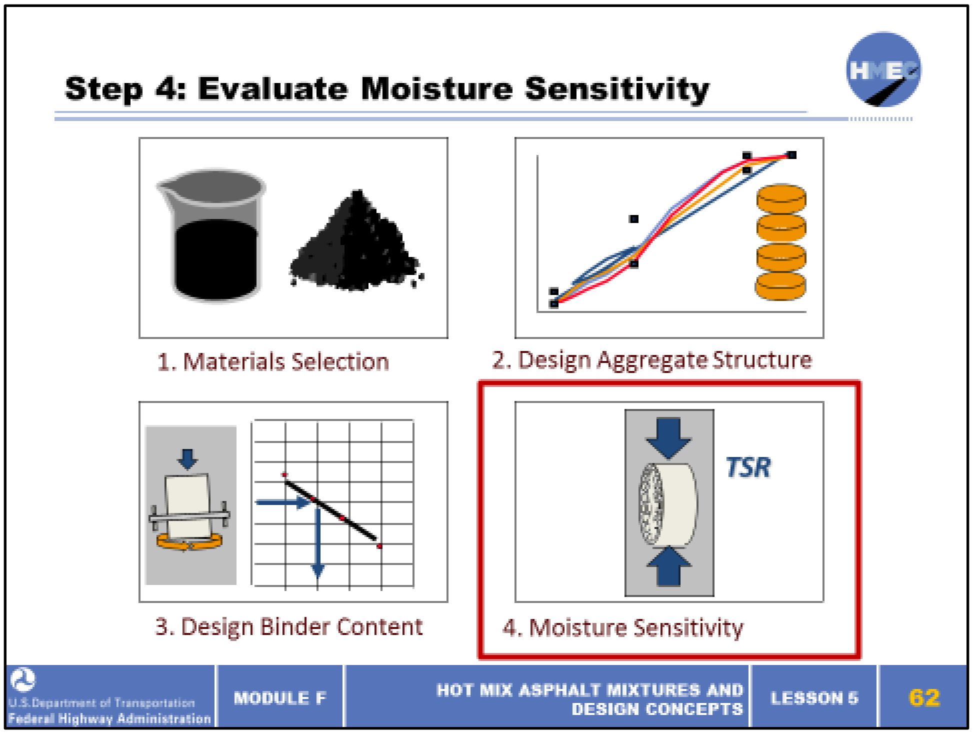
The bulk density or bulk specific gravity of the compacted mixture is measured in step #4. This actual design is compared to the calculated density at the same number of gyrations used to prepare the test specimen. This ratio of the actual to calculation density or bulk specific gravities is the correct factor for determining the percent compaction or density at other gyrations. As noted in the slide, the correction factor is determined and defined as the actual density or bulk specific gravity divided by the calculated density or bulk specific gravity at the same number of gyrations used to prepare the specimen.

Slide 61

Effect of truck traffic on mix design:

• The design ESALs is the expected traffic in the design lane over a 20-year period.
• Regardless of the actual design life of the roadway, determine the ESALs for 20 years and the appropriate N_{design} level.
• If the layer is >100 mm below the pavement surface, decrease the design traffic level by one (unless there will be significant main line and construction traffic prior to being overlaid).

Slide 62

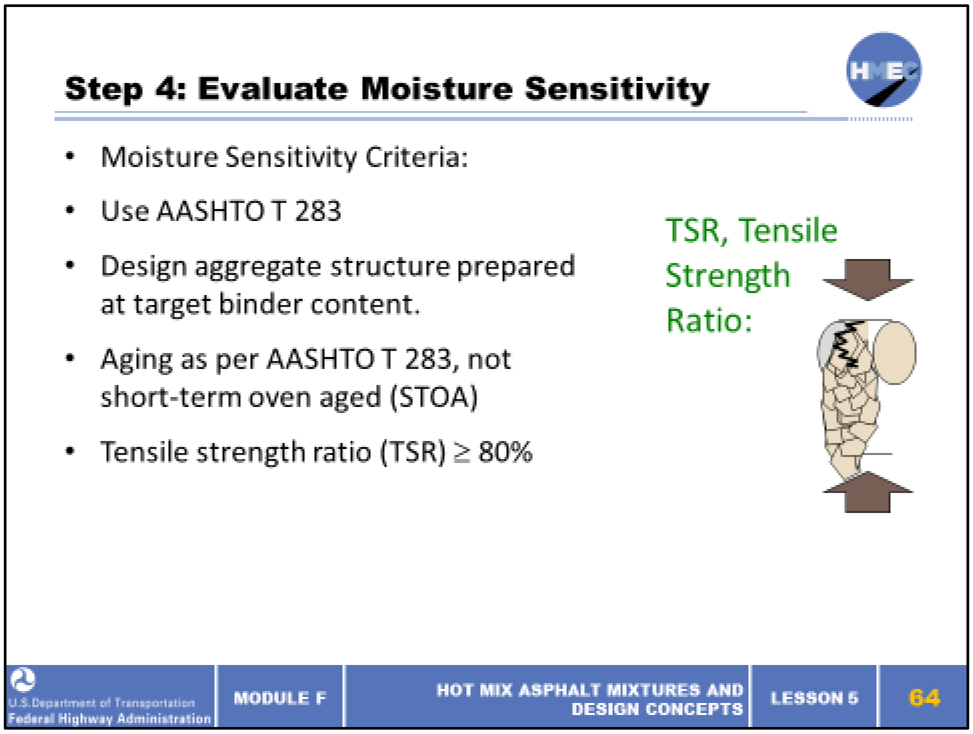


This section addresses Step 4: Evaluate moisture sensitivity.

Slide 63

These photos illustrate the condition of roadways that exhibited stripping and/or moisture damage.

Slide 64



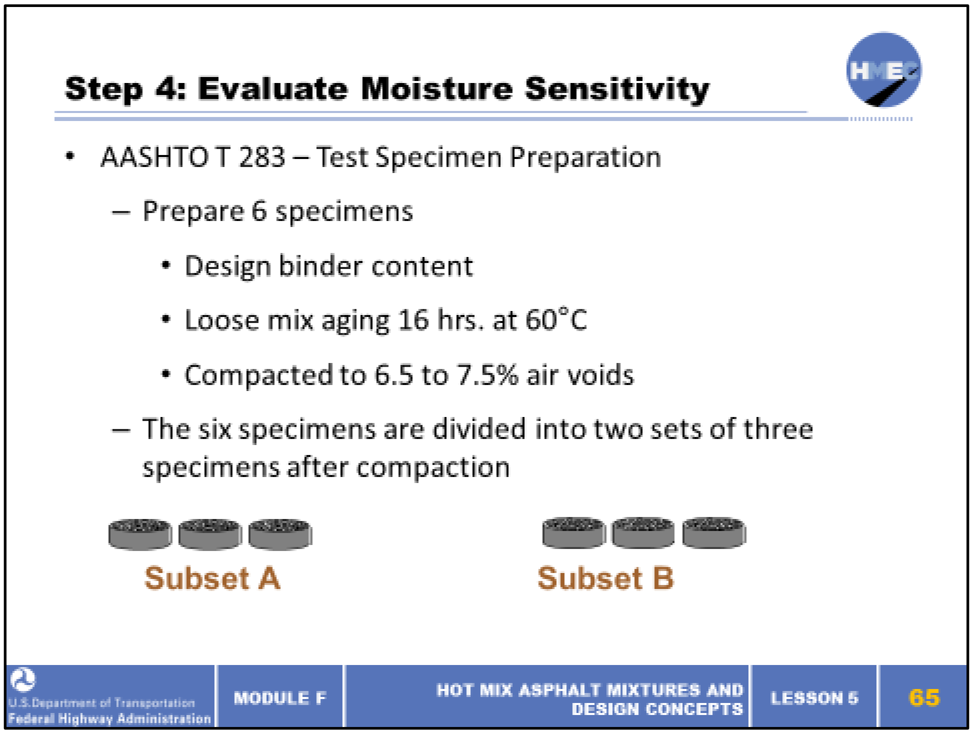
AASHTO T 283 (with or without the modified Lottman conditioning) is used to evaluate moisture sensitivity.

The testing is conducted at the design binder content only.

It is important to note that the loose mixture aging specified in AASHTO T 283 is different that the aging specified in the Superpave volumetric mix design process. The aging specified in AASHTO T 283 should be used.

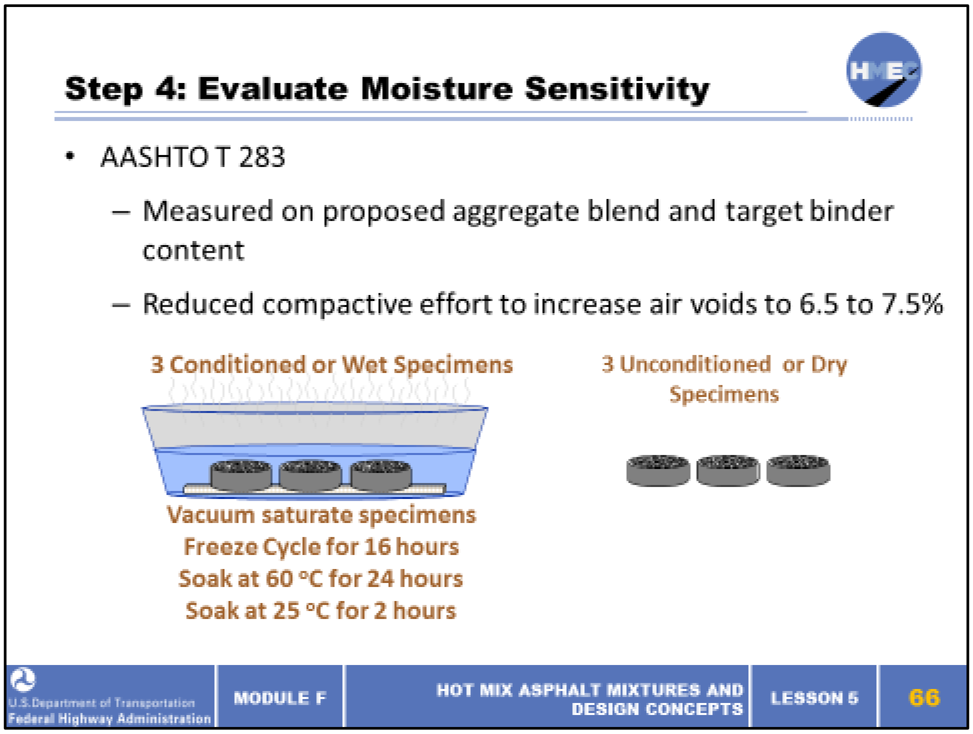
A minimum tensile strength ratio (TSR) of greater than or equal to 80% must be achieved. TSR is the ratio of the average tensile strength of a conditioned set of specimens to the average strength of an unconditioned set of samples. The conditioning procedure in AASHTO T 283 will be reviewed in the following slides.

Slide 65



The specimens are compacted to 6.5 to 7.5% air voids after loose mix aging of 16 hours at 60 °C. The six specimens are divided into two sets of three specimens after compaction. Each set of specimens should have about the same air void level.

Slide 66



This is actually a freezing-hot water (-18 to +60 °C) cycle. The other set is left at room temperature for the two day period, or, in other words, this set is unconditioned.

Many asphalt technologists use the terminology wet and dry for conditioned and unconditioned, respectively. SMA mixture design also requires moisture sensitivity testing in accordance to AASHTO T 283.

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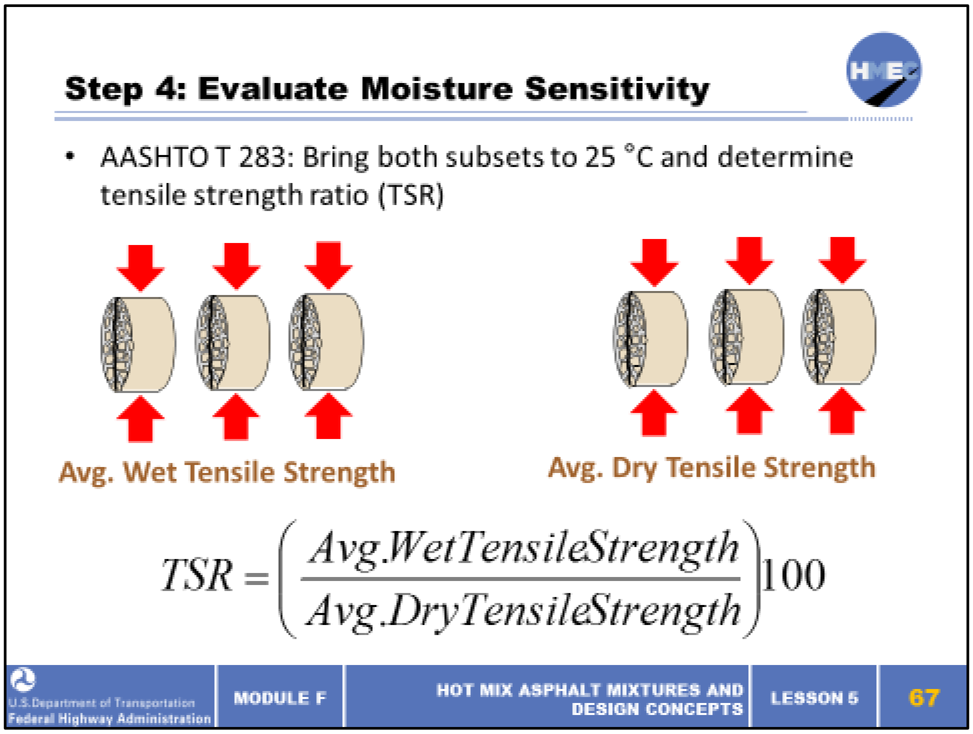
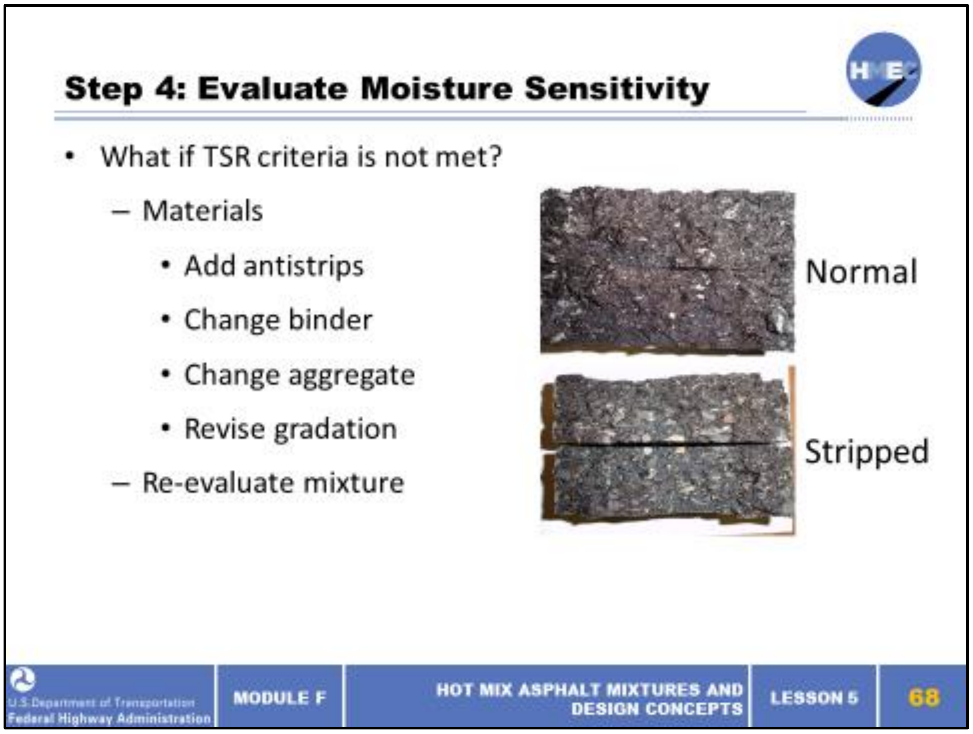
Slide 67



The index of moisture susceptibility obtained with this test is the tensile strength ratio (TSR).

TSR is simply the ratio of the average tensile strength of a set of conditioned to a set of unconditioned specimens expressed as a percent. The minimum allowable TSR is 80%.

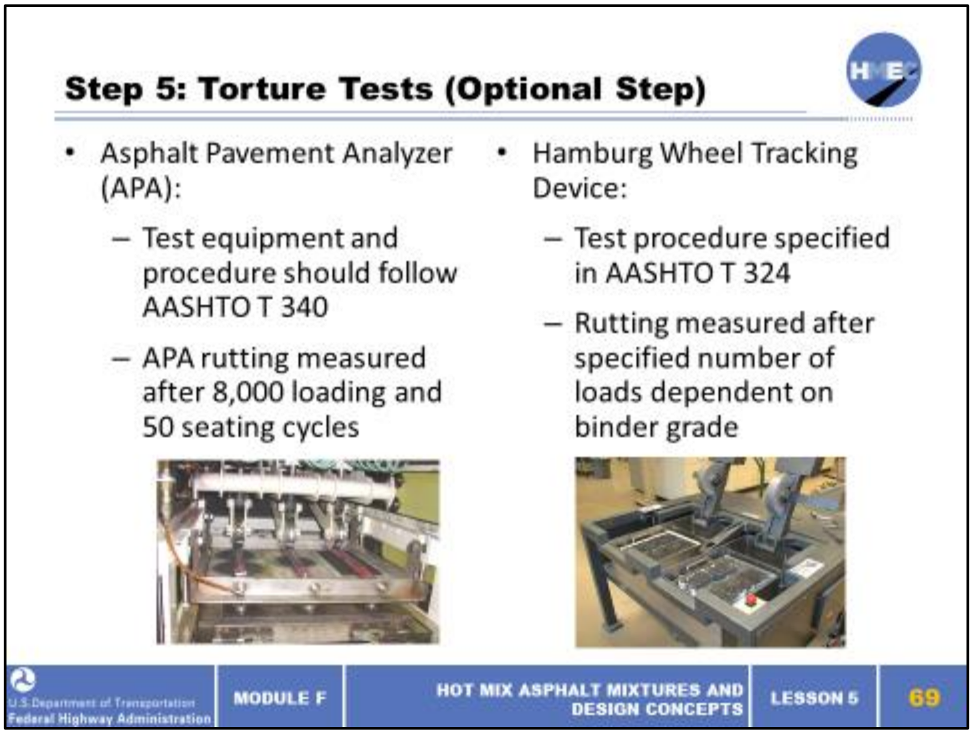
Slide 68



When this occurs, the mix designer must consider techniques to reduce the moisture susceptibility of the materials. This does require starting the mix design process all over. The different options include the following:

• Adding some antistrip additive and/or hydrated lime, which is the best option. The other options are less likely to have a huge benefit to reduce the moisture susceptibility of the mixture;
• Change the aggregate type;
• Change the binder type; and
• Revise the gradation (make it less permeable).

Slide 69



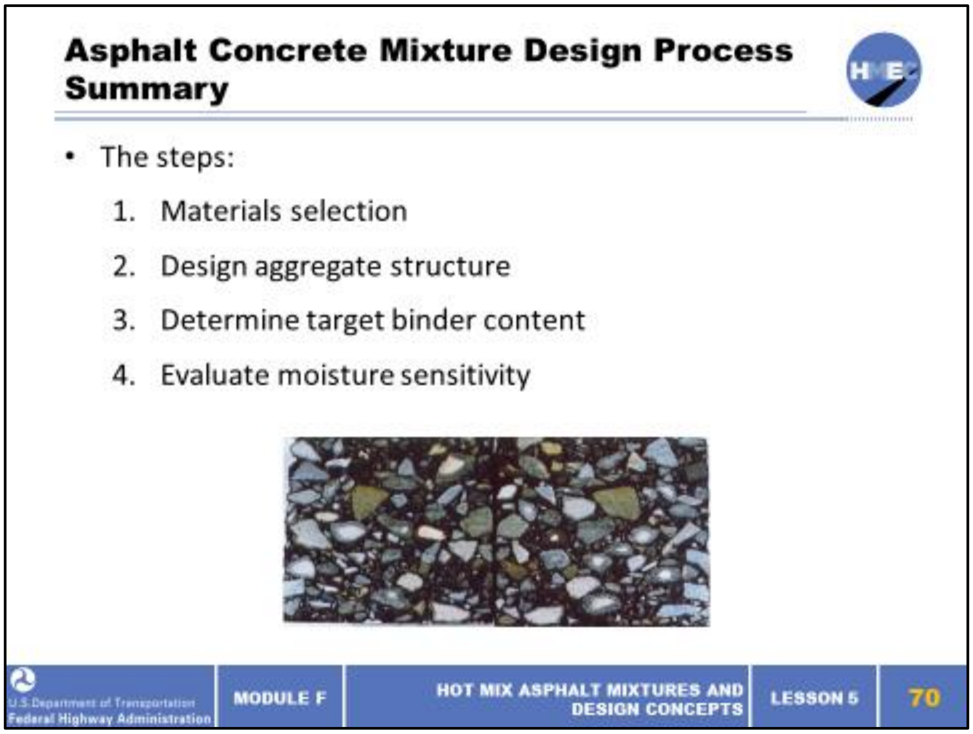
The two more common torture tests used are the asphalt pavement analyzer (AASHTO T 340) and Hamburg wheel tracking device (AASHTO T 324). Both of these devices have been used extensively in selected areas or agencies in the US and provide a mixture's responses or data on the deformation and moisture damage resistance of asphalt concrete mixtures. The APA is normally used to test asphalt concrete specimens in a dry condition (specimens are not submerged in water), while the Hamburg device is normally used to test specimens that are submerged in water for maintaining the temperature of the specimen and obtaining data and information on the mixture's susceptibility to moisture damage. However, both tests can be run in a dry or wet condition.

The failure criteria for the APA device (AASHTO T 340) in terms of deformation of the specimen (varies from 3 to 5 mm) is dependent on the traffic level with a specific number of loading cycles as noted in the slide, while the failure criteria for the Hamburg device (AASHTO 324) in terms of deformation of the specimen (12.5 mm) is constant with the number of loading cycles being dependent on the traffic level (varying from 10,000 to 20,000 passes).

NCHRP Project 9-33 has developed guidelines to determine appropriate testing temperatures, as well as APA rutting criteria (AASHTO T 340) for asphalt concrete mixture design, while

AASHTO T 324 provides the standardized procedure for the Hamburg wheel tracking device. These procedures and their test results are included and discussed in more detail within Lesson 6 on performance tests.

Slide 70



Some agencies are responsible for the mixture design process on a project by project basis. However, many states are handing that responsibility over to the contractor, and are only confirming the mixture design recommended for use by the contractor.

Slide 71

The criteria and volumetric properties listed on the slide in terms of selecting the aggregate structure and binder content for the aggregate structure.

1. Consider the environment in terms of traffic and weather – some of the volumetric properties are traffic volume dependent which are listed on the next slide and the binder grade is climate or weather dependent;
2. Air Voids @ Ndes (number of gyrations for design) – the binder content is selected at a specific air void value to ensure adequate mixture durability (listed on the next slide). Some agencies use a design air void level that is layer and climate dependent. That design air void level is usually based on or determined from experience;
3. Air Voids @ Nmax (number of gyrations for the long term traffic) – the asphalt concrete mixture is checked to ensure the mixture will retain a certain level of air voids over time. In other words, it will not densify under traffic such that rutting (distortion) or bleeding will occur over time;
4. Check VMA – to ensure that there is adequate void space available in the aggregate structure for the asphalt binder. As a reminder the VMA criterion is aggregate size dependent, as listed on the next slide. The selected binder content should be in the range where the VMA is not changing or slightly decreasing as more asphalt binder is added to the aggregate structure. In

other words, the density of the mixture increases with a slight increase in binder content. It is generally good practice to stay away from the range of binder contents where an increase in binder content results in higher VMA and lower mixture density;

5. Check VFA – to ensure that a sufficient amount of binder or binder film thickness is available to properly coat the aggregate particles from a durability standpoint. As a reminder the VFA criterion is traffic level dependent, as included on the next slide;

6. Check Dust to Binder ratio – to ensure the amount of dust is limited or that a sufficient amount of binder is added to the aggregate blend from a coating and durability standpoint.

Slide 72

Asphalt Concrete Mixture Design Process Summary – Volumetric Design Criteria

- Volumetric Design Criteria (AASHTO M323) for binder content determination

Traffic	%Gmm Criteria			VMA	VFA	DB
ESAL (10^6)	N_{ini}	N_{des}	N_{max}			
<0.3	≤91.5	=96.0	≤98.0	See Table for NMAS	70-80	0.6 to 1.2
0.3 to 3	≤90.5				65-78	
3 to < 30	≤89.0				65-75	
≥ 30	≤89.0					

NOTE: Some States may have modified these parameters (i.e., NYSDOT: N_{des} = 96.5%)

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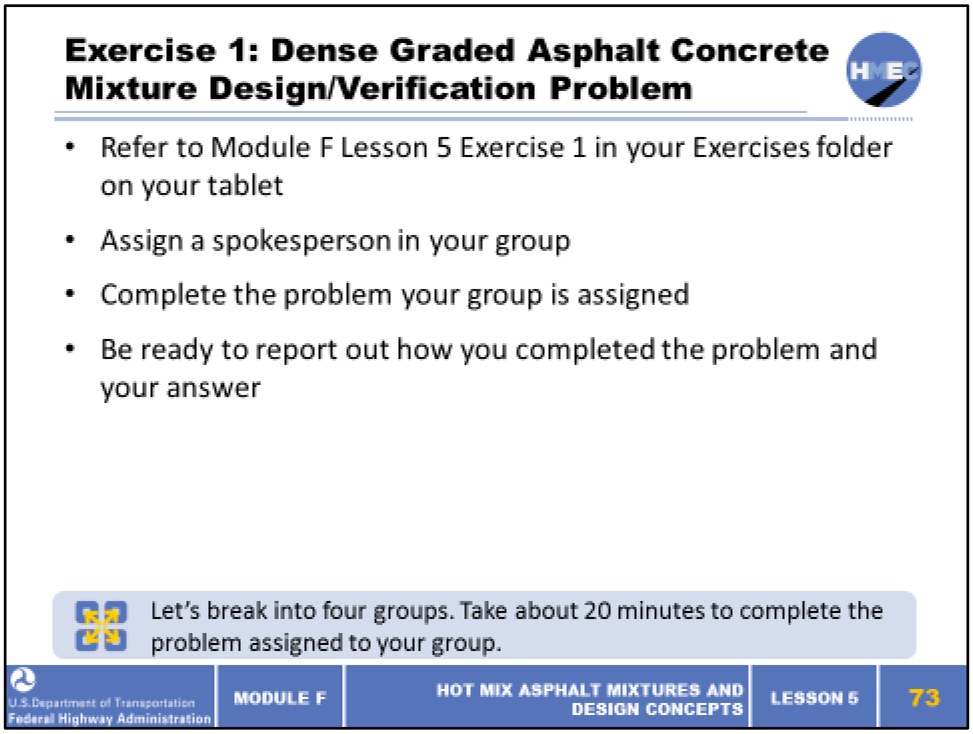
The volumetric criteria listed and summarized on this slide were defined and discussed in the previous two slides. In summary, VMA is a function of nominal maximum aggregate size (NMAS), and percent compaction at the design and maximum number of gyrations, as well as dust-to-binder ratio or proportion, have fixed criteria. As a reminder, when the gradation passes below the Primary Control Sieve control point, the dust-to-binder ratio criterion can be increase from 0.6 to 1.2 to 0.8 to 1.6, which is stated in Note 9 of M 323-13.

Slide 73


Exercise 1: Dense Graded Asphalt Concrete Mixture Design/Verification Problem
HMEC
• Refer to Module F Lesson 5 Exercise 1 in your Exercises folder on your tablet
• Assign a spokesperson in your group
• Complete the problem your group is assigned
• Be ready to report out how you completed the problem and your answer
Let's break into four groups. Take about 20 minutes to complete the problem assigned to your group.
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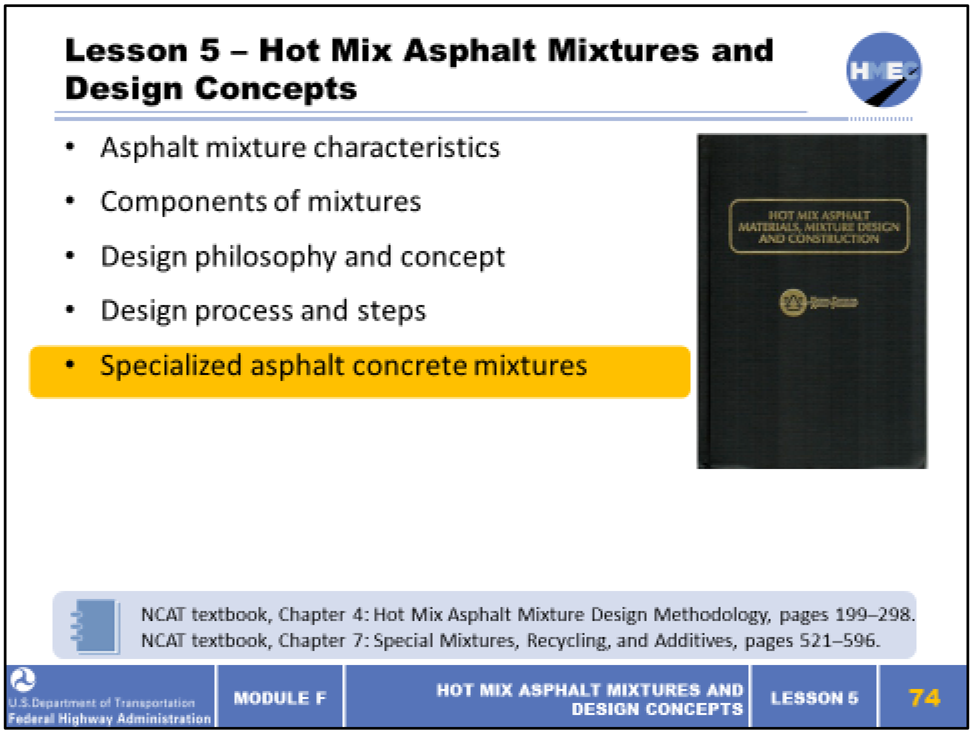
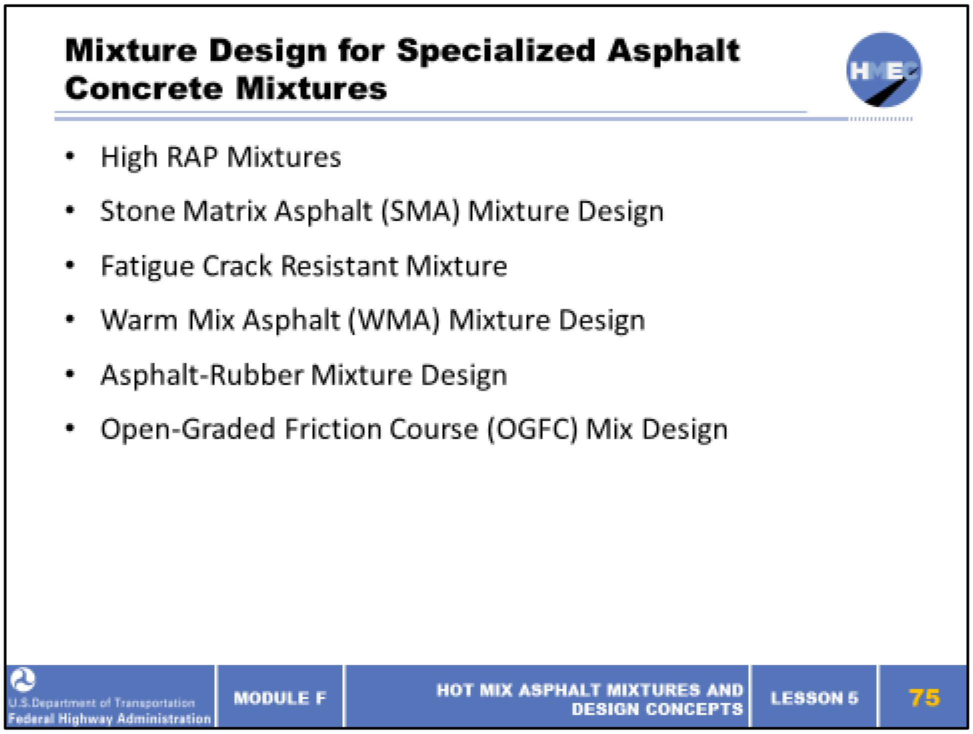
Slide 74



Some of the information covered within this lesson and section comes from many different sources but was generally taken from Chapter 4, Hot Mix Asphalt Mixture Design Methodology, and Chapter 7, Special Mixtures, Recycling, and Additives, of the NCAT textbook and AASHTO test standards and specifications.

Slide 75



The following specialized asphalt concrete mixtures will be briefly discussed in terms of the mixture design process and steps previously listed and discussed within Lesson 5.

Slide 76

High RAP Mixtures

- Guidelines for incorporating RAP are provided in AASHTO M 323, Section 5.3, Table 2

Recommended Virgin Asphalt Binder Grade	RAP Percentage
No change in binder selection	< 15%
Select virgin binder one grade softer than normal (i.e.; PG 58-28 if a PG 64-22 is normally used)	15 to 25%
Follow recommendations from blending charts (AASHTO M 323, Appendix X1)	> 25%

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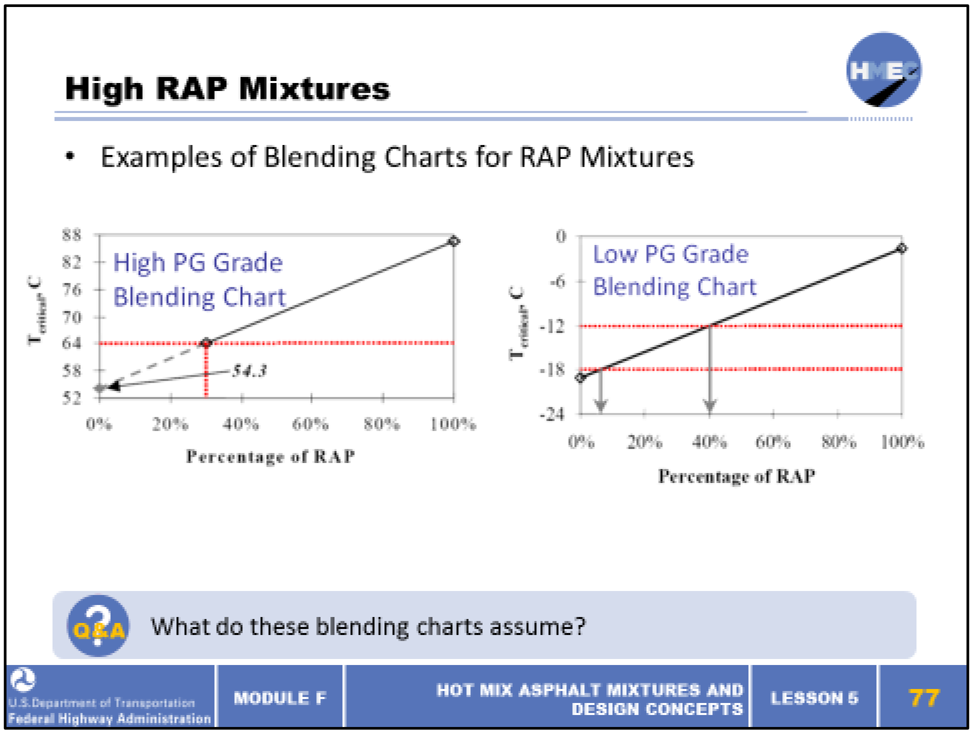
The main goal of M 323 is how to handle the general increase in mixture stiffness and tendency to result in premature cracking at intermediate and low temperatures.

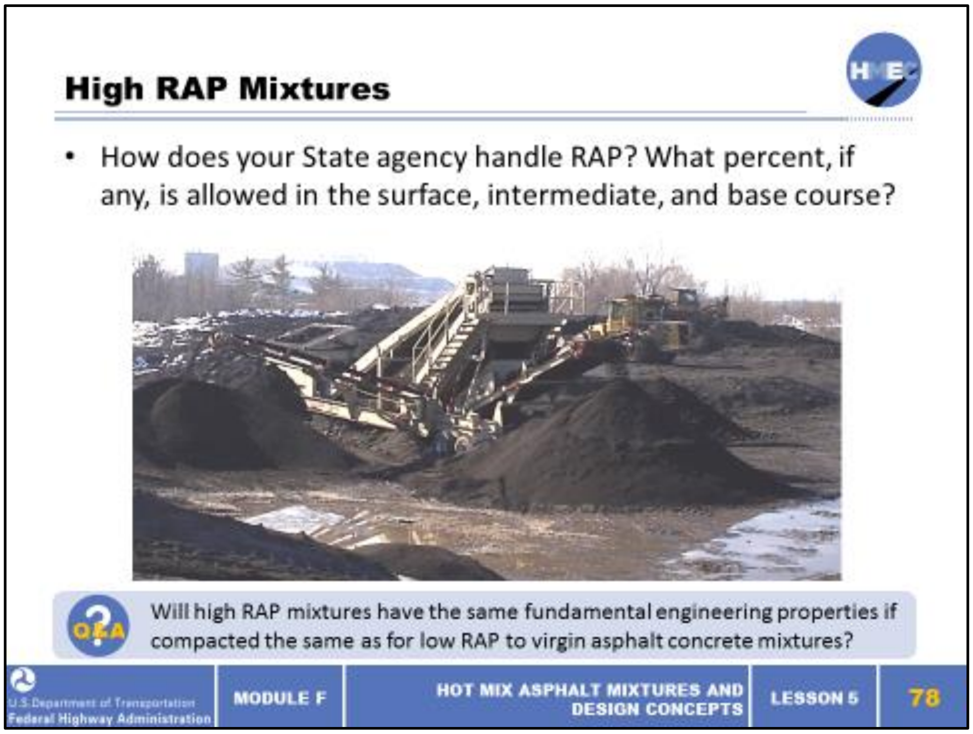
Table 2 in AASHTO M 323 provides guidance on how to modify HMA mixtures with different levels of RAP to limit the potential to cracking.

Slide 77



A blending chart uses the binder grade of the RAP binder and the virgin binder to determine the resultant or final binder content of the final mixture.

Slide 78



The Long Term Pavement Performance (LTPP) program included high RAP mixtures (30% RAP added to the mixtures) within the SPS-5 experiment and is a good source of data for comparing the performance of virgin and RAP mixtures.

Slide 79


High RAP Mixtures
Will high RAP mixtures have the same fundamental engineering properties if compacted the same as for low RAP to virgin asphalt concrete mixtures?
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Slide 80


Stone Matrix Asphalt (SMA) Mixture Design
• Gap-graded, large stone-on-stone contact to provide internal strength; higher VMA aggregate structures
• High binder content (6–7%) provides improved durability
• Polymer-modified asphalt
• Mineral filler or fiber to prevent binder draindown
• In-place air voids < 6%
• Rut-resistant
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Federal Highway Administration
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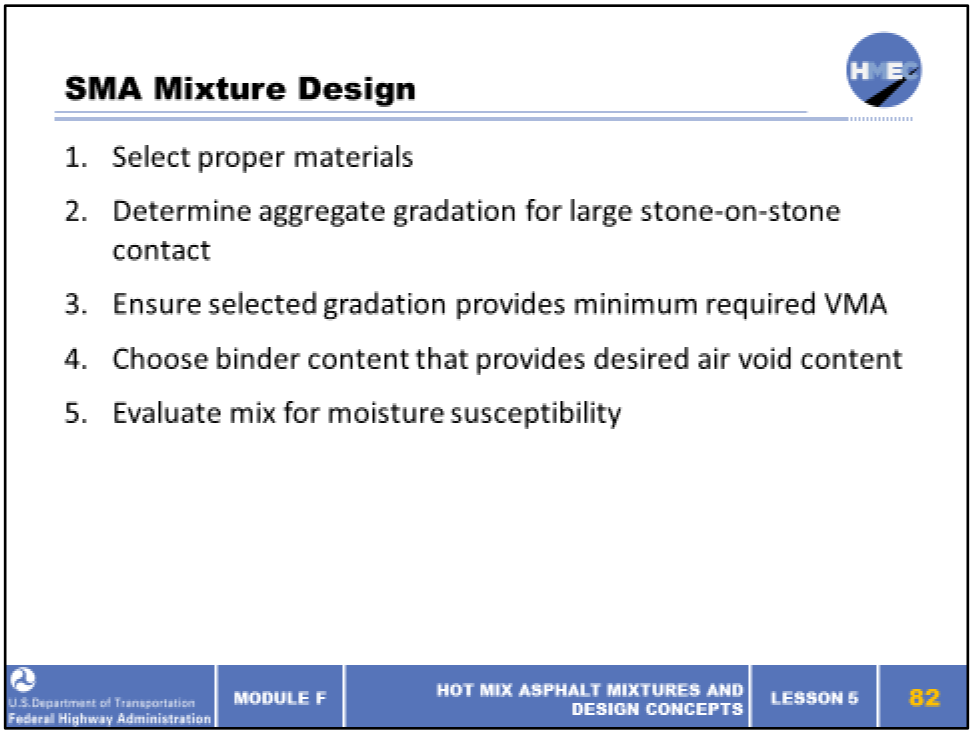
Slide 81

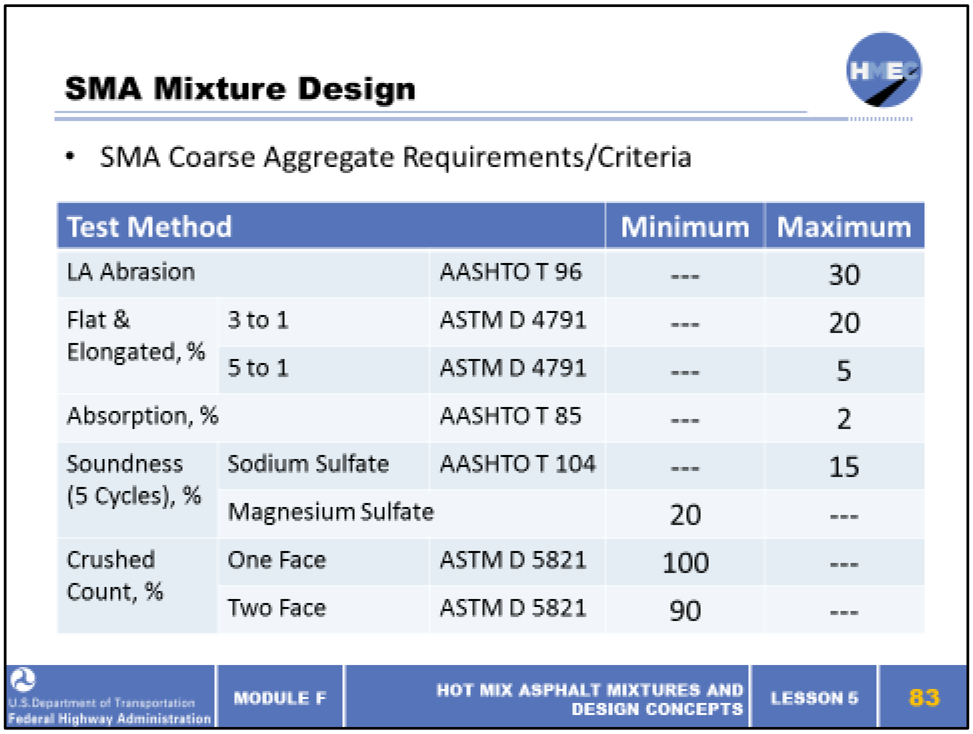
The methodology used for SMA mixture design is provided in AASHTO M 325-08 and AASHTO R 46-08.

Slide 82



The specific steps are listed in this slide.

Slide 83



In general, aggregates used for SMA mixes need to be harder, less prone to degradation or abrasion, and more cubical (reduced percentages of flat and elongated).

The aggregate selected for use in an SMA mixture should meet all of the criteria for the coarse and fine aggregate that are listed in the above table. Most state agencies specify higher levels of angularity. However, recent work on the NCAT Test Track has shown that SMAs constructed with uncrushed gravels resulted in good performance, as related to rutting resistance.

Slide 84

SMA Mixture Design

- SMA Fine Aggregate Requirements/Criteria

Test Method			Minimum	Maximum
Soundness (5 Cycles), %	Sodium Sulfate	AASHTO T 104	---	15
	Magnesium Sulfate		---	20
Liquid Limit, %		AASHTO T 89	---	25
Plasticity Index, %		AASHTO T 90		Non-Plastic

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The requirements shown in the slide are commonly specified by the individual State agency and not required by Superpave for dense graded mixes. However, due to the requirements of hard, angular aggregates, SMA aggregates must meet higher standards.

Slide 85

SMA Mixture Design

- SMA Recommended Gradation; AASHTO M 325

Sieve Size, mm	19 mm	12.5 mm	9.5 mm
25	100		
19	90–100	100	
12.5	50–88	90–100	100
9.5	25–60	50-80	70–95
4.75	20–28	20-35	30–50
2.36	16–24	16-24	20–30
1.18	-	-	- 21
0.60	-	-	- 18
0.30	-	-	- 15
0.075	8.0–11.0	8.0–11.0	8.0–12.0

Percent Passing by Weight

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The percent passing the number 4 sieve (4.75 mm sieve) is important because it relates to the amount of voids between the coarse aggregate particles and to ensure that the aggregate structure or grading selecting will have stone on stone contact. This importance will be presented later within this section.

The percent passing the number 200 sieve (0.075 mm sieve) is normally higher than for well-graded aggregate blends because a higher amount of fines is needed to fill the void structure between the coarse aggregate particles and to stiffen the mixture, making it more resistant to deformation and rutting.

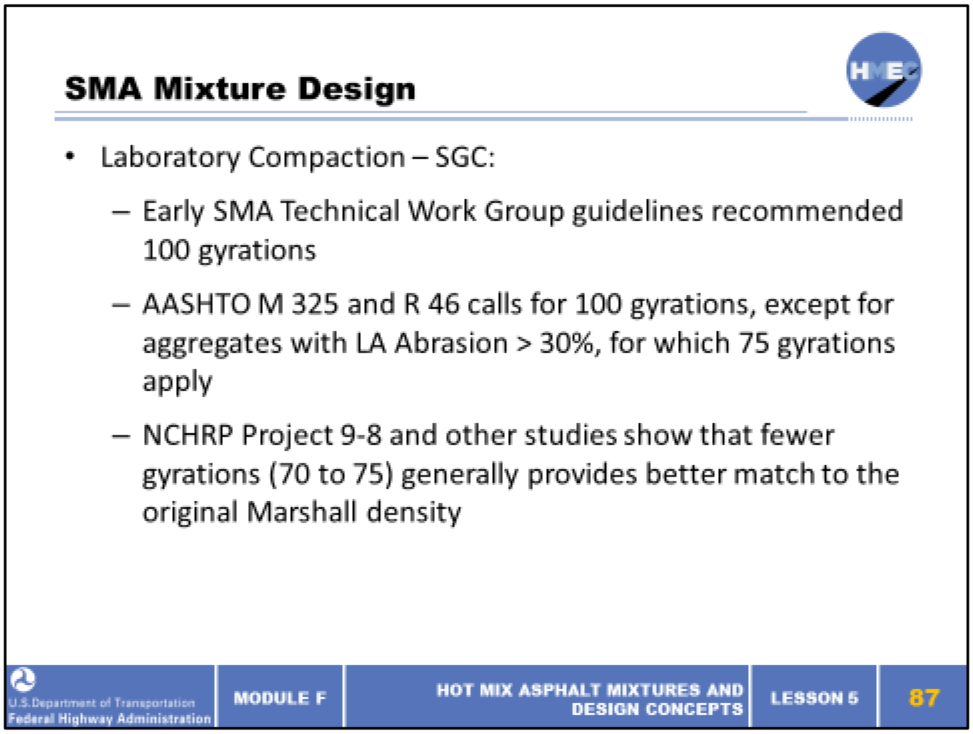
Slide 86

SMA Mixture Design

- AASHTO M 325 – Specifications or mixture design criteria using the SGC:

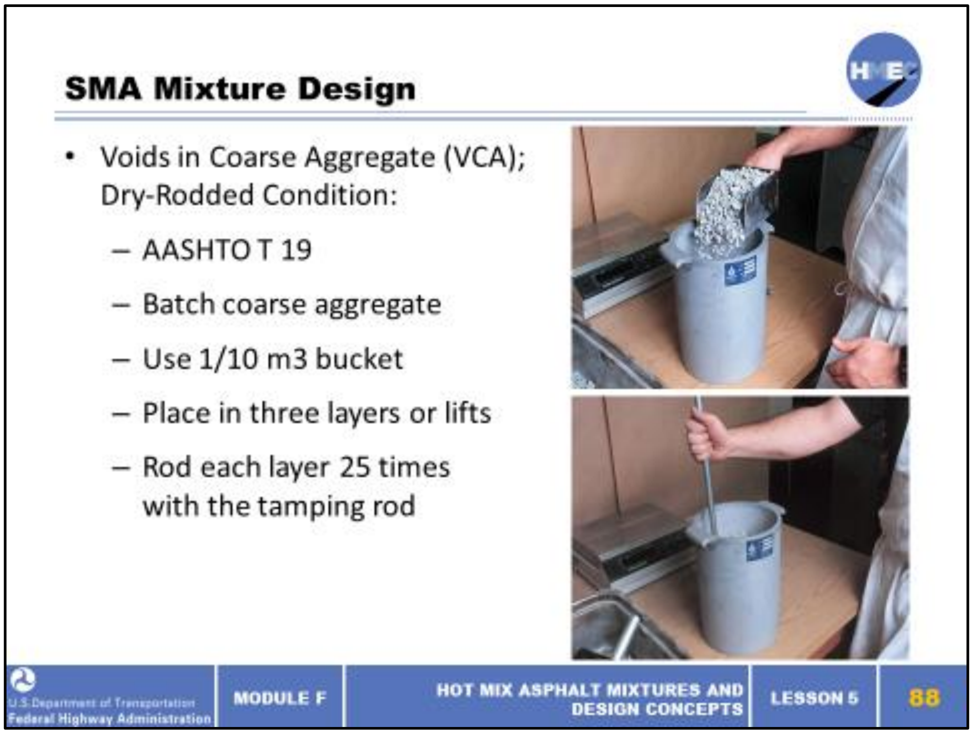
Property	Requirement
Air Voids, %	4.0
VMA, %	17.0 min.
VCA_{Mix}	< VCA_{DRC}
TSR, AASHTO T 283, %	> 80
Draindown at Production Temperature, %	0.3 max.
Binder Content, %	6.0 min.

Slide 87



• AASHTO M 325 and R 46 recommend using 100 gyrations during mix design.
• However, some States that have aggregates that may exhibit high loss in the LA Abrasion (loss > 30%) are recommended to use a lower level of compactive effort (75 gyration) so as not to crush the aggregates during compaction.
• This seems acceptable, since work under NCHRP Project 9-8 have shown that 70 to 75 gyrations have better comparable volumetrics to the original 75 blow Marshall designs.

Slide 88

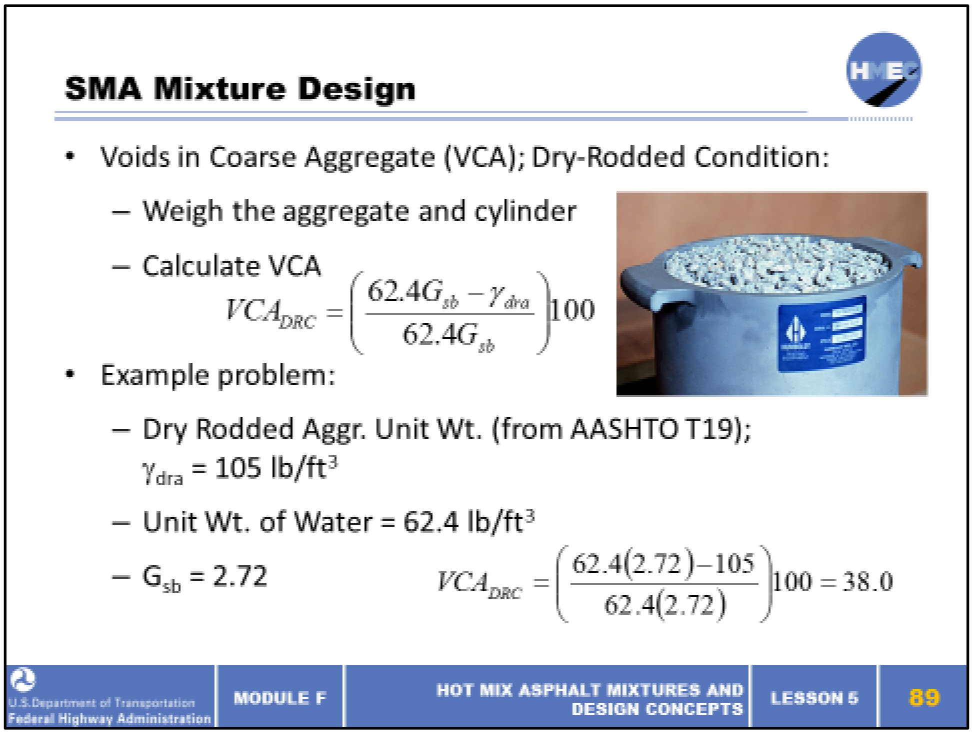


One of the major differences in designing gap-graded mixtures (SMA and OGFC) is to ensure that stone-on-stone contact between the aggregate particles occurs.

This maximizes the loading carrying capacity of mixture, since they generally contain higher asphalt contents than dense-graded mixtures. To accomplish this, the voids in course aggregate (VCA) are determined.

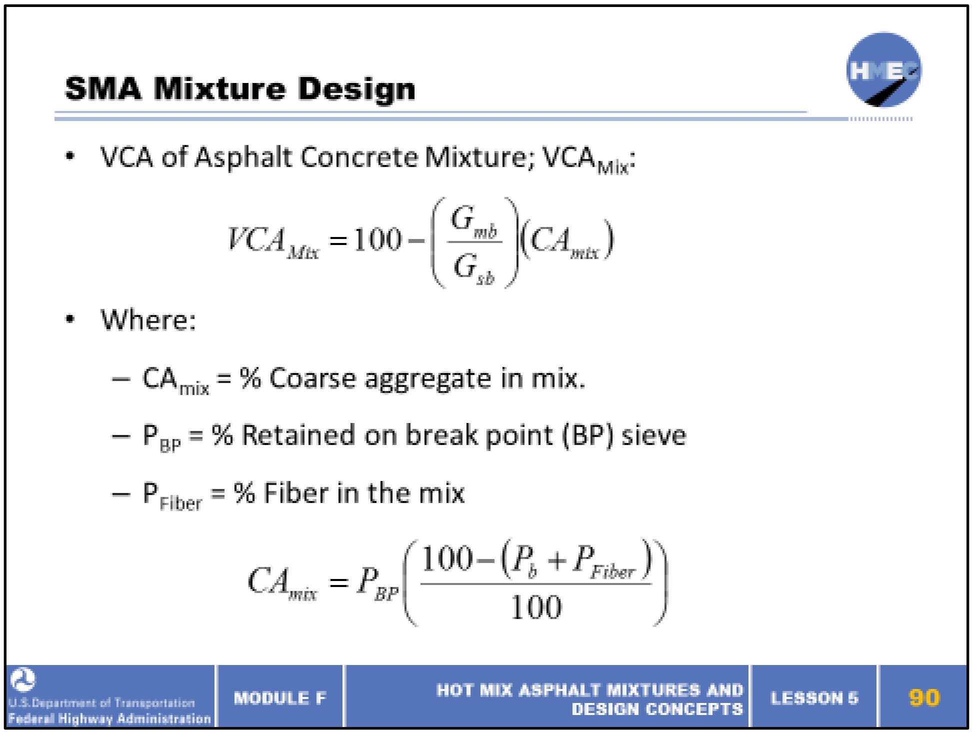
The primary test used to determine VCA is AASHTO T 19. The aggregate is added in three separate lifts and each lift is rodded 25 strokes of the tamping rod.

Slide 89



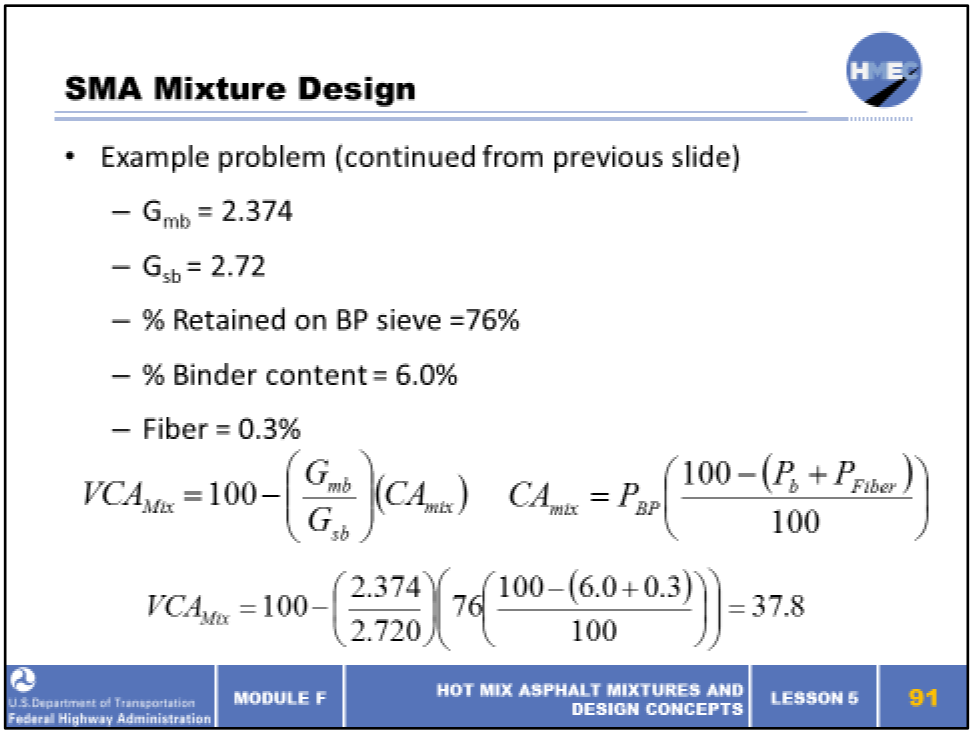
Knowing the volume of the cylinder, the unit weight (mass/volume) can be calculated. This slide also shows how to calculate the VCA_{DRC}.

Slide 90



This slide shows how to calculate the VCA_{mix}.

Slide 91



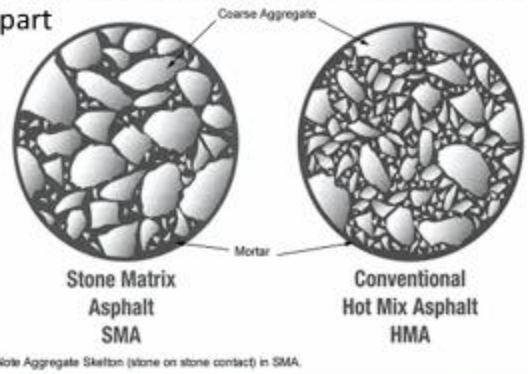
This is proposed to be conducted on the design volumetric design samples compacted to final air void level of 4%.

BP sieve is the #4 (4.75 mm) sieve.

Slide 92

SMA Mixture Design

- VCA of compacted sample (VCAmix) must be less than VCADRC or the intermediate particles may be pushing stone skeleton apart



Q&A Based on the information discussed on the grading, what is the design philosophy for the determining the aggregate gradation?

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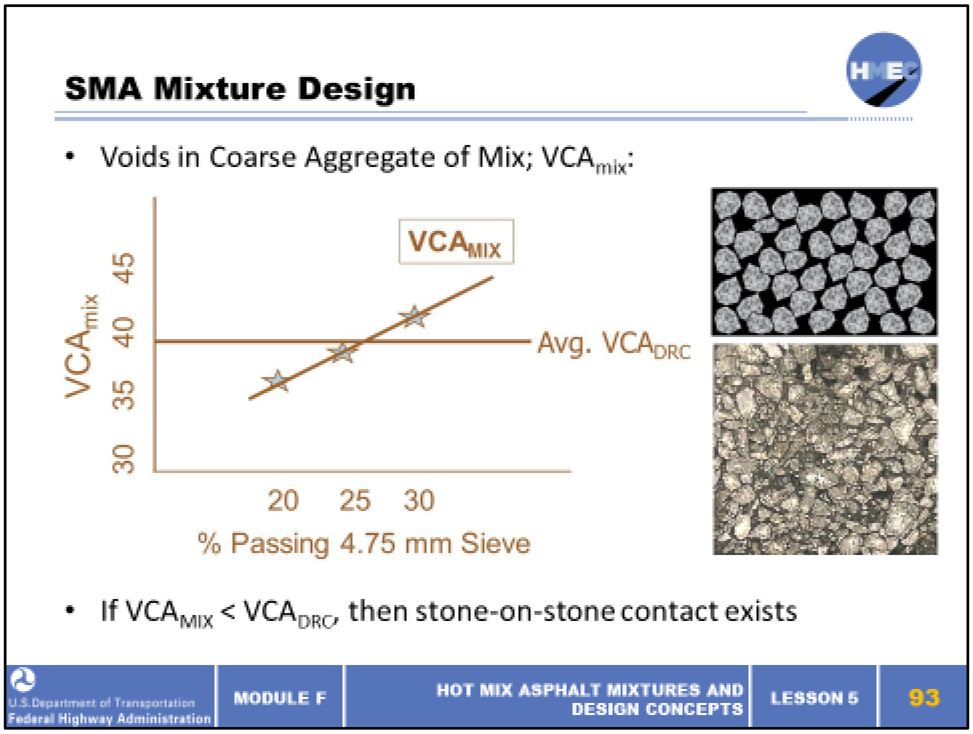
Slide 93

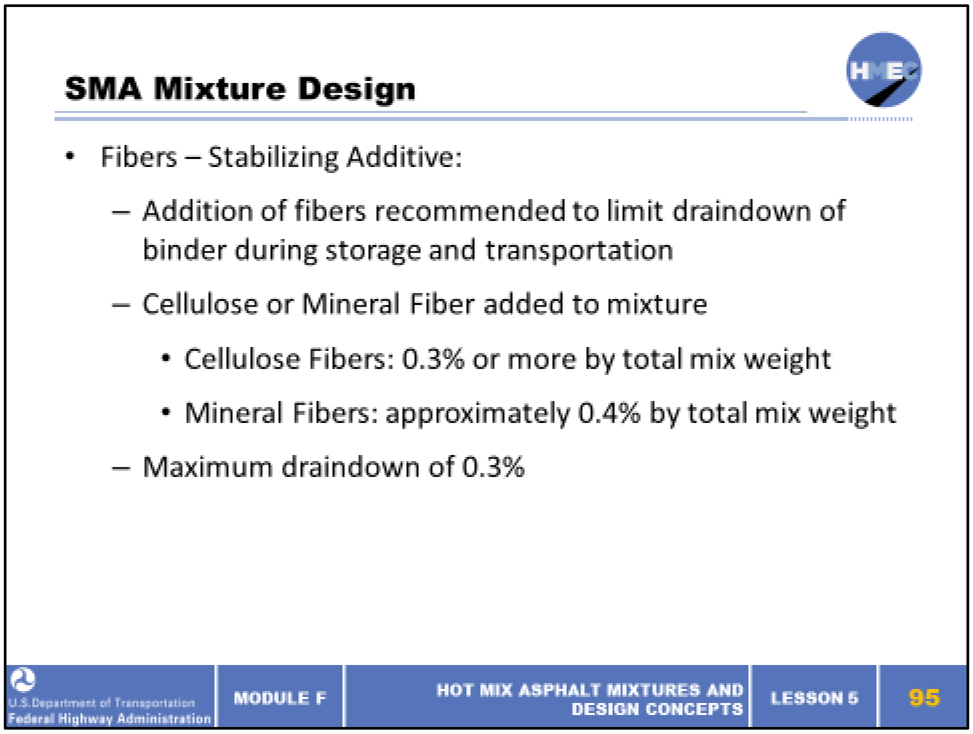


Ultimately, the mix designer wants to have VCA_{mix} to be less than $VCA_{dry\ rodded\ (DRC)}$. When that condition is satisfied: stone-on-stone contact of the larger aggregate particles will exist.

Slide 94

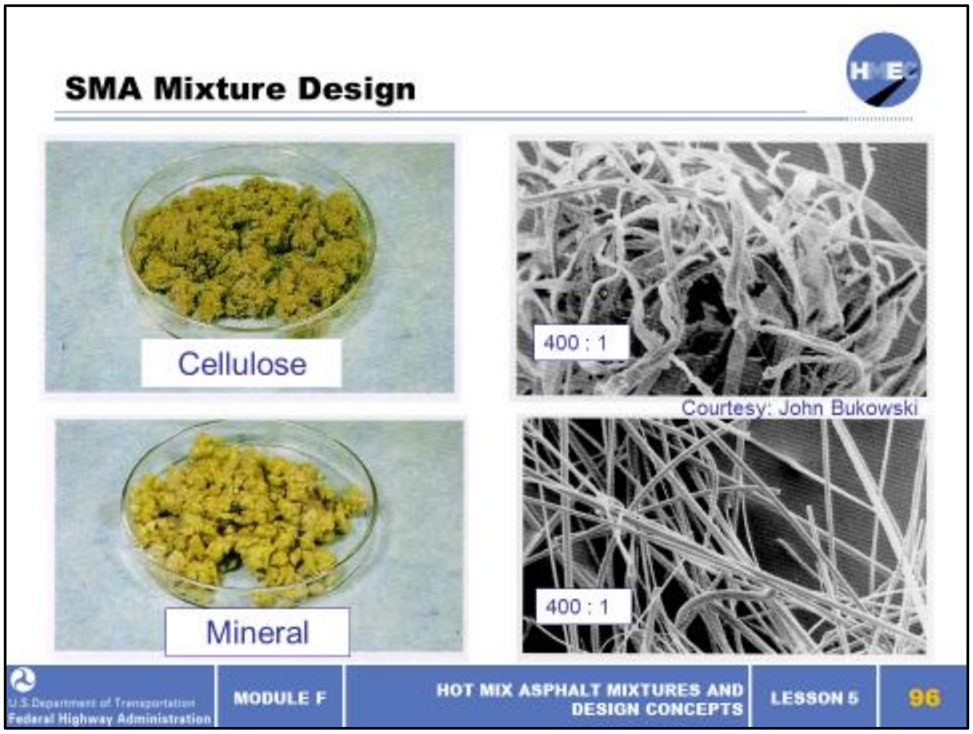
Draindown is when the binder simply drains off the aggregate surface, potentially causing fat spots and areas of binder segregation (some flushing and some areas too dry).

Slide 95



Both mineral and cellulose fibers have been found to be adequate, and selection is primarily a function of cost and availability.

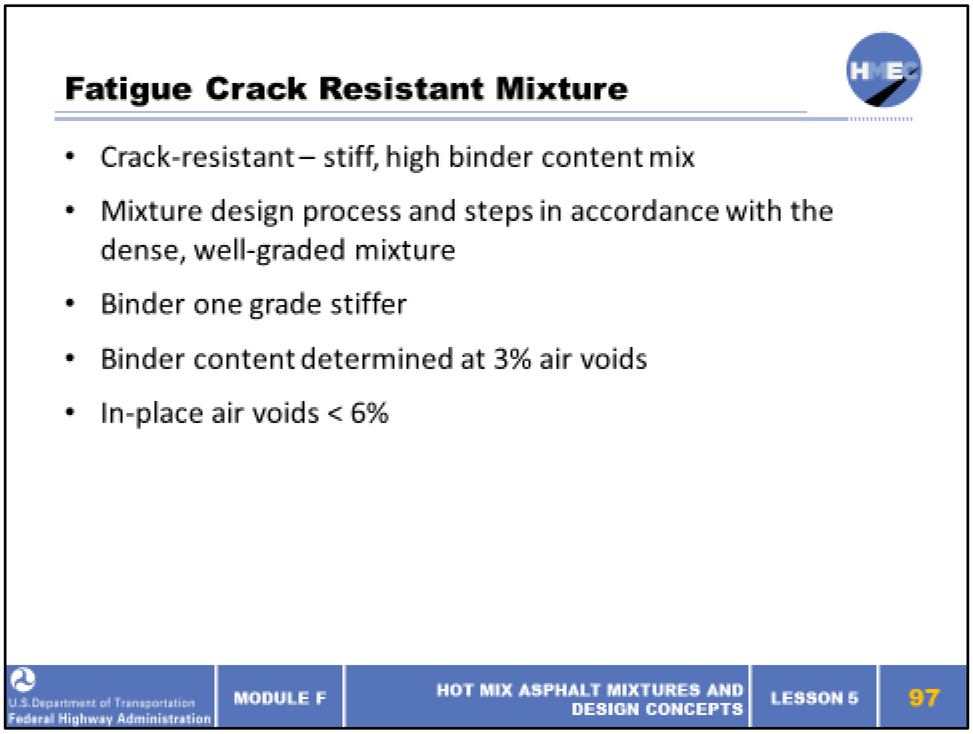
Slide 96



The key point here is there are two different types of fibers that State agencies can specify and can use. Fibers were previously discussed in Lesson #1.

Most of the fibers used in asphalt mixtures today are synthetic. Polypropylene and polyester fibers are typically used as reinforcement in the asphalt mixture. Cellulose fibers are used extensively in SMA mixtures for the purpose of increasing the amount of binder without any draindown. Mineral fibers are used in dense-graded, SMA, and open-graded mixtures. Typically, mineral fibers are manufactured from diabase as a raw material.

Slide 97

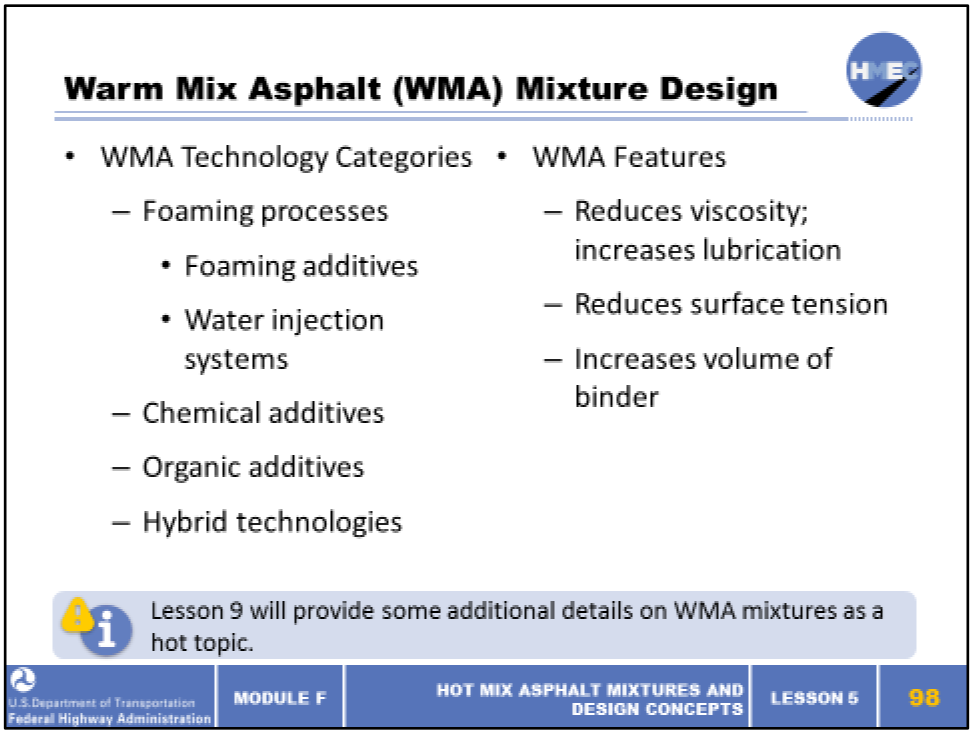


These mixtures are defined as a high stiffness, high binder content mixture to resist deformation from the truck loads. The higher binder content and lower air voids have increased fatigue strength. Typically, the pavement structure is designed so that the tensile strains at the bottom of the asphalt concrete layers are below the endurance limit of the mixture.

The endurance limit of the mixtures is defined as the tensile strain under repeated flexural strains for which no fatigue damage occurs.

These mixtures are designed in accordance with the dense, well-graded mixtures. The difference in that a stiffer asphalt binder is used, about one grade stiffer than required for the project climate and traffic conditions, and the target binder content is determined at a lower design air void level of 3%. The in place air voids after construction are required to be lower than typically used for dense, well-graded mixtures.

Slide 98



Generally, WMA technology categories can be broadly grouped as follows:

1. Organic additives are typically special types of waxes that are added to modify the viscosity of the binder. Organic additives also include hydrocarbon based viscosity modifiers.

2. Chemical additives or surfactants that work to reduce the surface tension of the binder at production and mixing temperatures, which improves mixing and compaction at lower temperatures, but does not affect the properties of the binder.

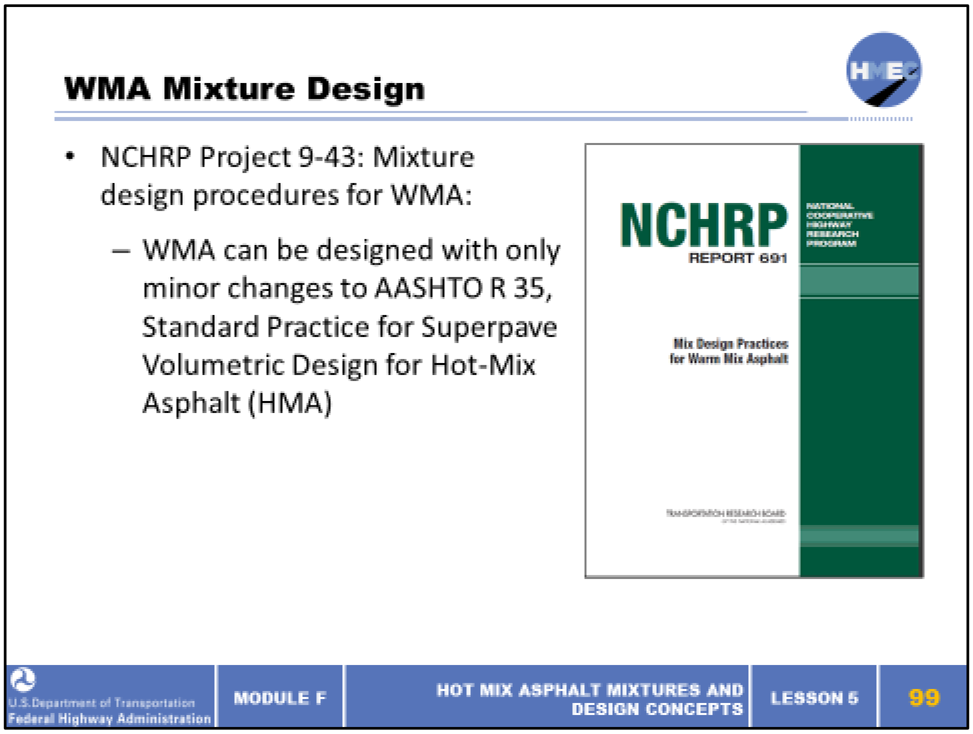
3. Processes that produce foamed asphalt. This category includes two processes:

• Foaming additives that release water when added with the mix at high temperatures to cause foaming action. This can come in the form of highly controlled aggregates or zeolites.

• Mechanical foaming devices or water injection systems inject small amounts of water into the hot asphalt binder to cause foaming action.

4. Sometimes a combination of the above mentioned technologies is also used. For example, a water injection technology is used in conjunction with a low dosage of chemical additives.

Slide 99

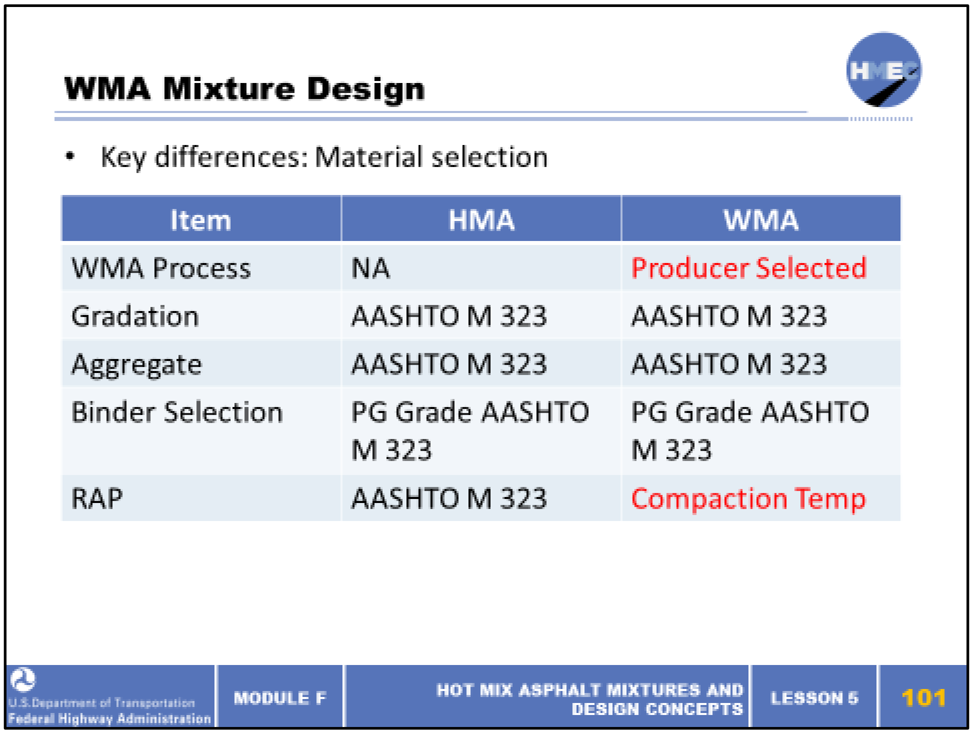


NCHRP Project 9-43 was the first in a series of national research projects addressing WMA. The primary objective of NCHRP Project 9-43 was to adapt a laboratory mixture design procedure to WMA. The primary product of the project was an Appendix to AASHTO R3 5 titled *Special Mixture Design Considerations and Practices for Warm Mix Asphalt (WMA)*. This appendix was balloted by AASHTO and published in the 2012 edition of the AASHTO Standard Specifications for Transportation Materials and Methods of Sampling and Testing.

Slide 100

The next three slides will compare WMA and HMA for each element.

Slide 101



This chart presents a summary of the differences between HMA and WMA for the materials selection portion of the mixture design process. It shows the key item, how it is addressed in current HMA design, and the similarities and differences for WMA design. Go through each of the five items as follows:

1. WMA Process. For WMA design, the producer must select the WMA process and the planned field mixing and compaction temperatures because the fabrication of WMA specimens in the laboratory is process specific, simulating in an approximate manner, the production of the mixture in the field. WMA process selection is best made by the producer in consultation with the specifying agency and WMA process suppliers.

2. Gradation. Same as HMA.

3. Aggregate. Same as HMA.

4. Binder Grade. Same as HMA.

5. RAP. When a recycled binder is used in WMA, there is a limit on the stiffness of the recycled binder to ensure adequate mixing of the new and recycled materials. This limit will generally not affect RAP but will limit the use of recycled asphalt shingles in many WMA mixtures.

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WMA Mixture Design

- Key Differences: Volumetric Design

Item	HMA	WMA
Mixing and Compaction Temperatures	Viscosity	Coating Compactability
Specimen Preparation	Standard	Process specific
Optimum Binder Content	AASHTO M 323 Volumetrics	AASHTO M323 Volumetrics

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This chart presents a summary of the differences between HMA and WMA for the volumetric design portion of the mixture design process. It shows the key item, how it is addressed in current HMA design, and the similarities and differences for WMA design.

1. Mixing and Compaction Temperature. Viscosity based mixing and compaction temperatures cannot be used to control coating, workability, and compactibility of WMA mixtures. For WMA, coating and compactibility are evaluated at the planned field production and compaction temperatures.

2. Specimen Preparation. Standard specimen fabrication procedures are used in HMA mixture design. For WMA, the specimen fabrication procedures are process specific using one or more of the generic categories previously described: 1. additive added to the binder, 2. additive added to the mixture, 3. wet aggregate mixtures, and 4. foamed asphalt.

3. Optimum Binder Content. Same as HMA, using the same gyration level, same design air void content, same VMA criteria, and same limits on dust to effective asphalt ratio.

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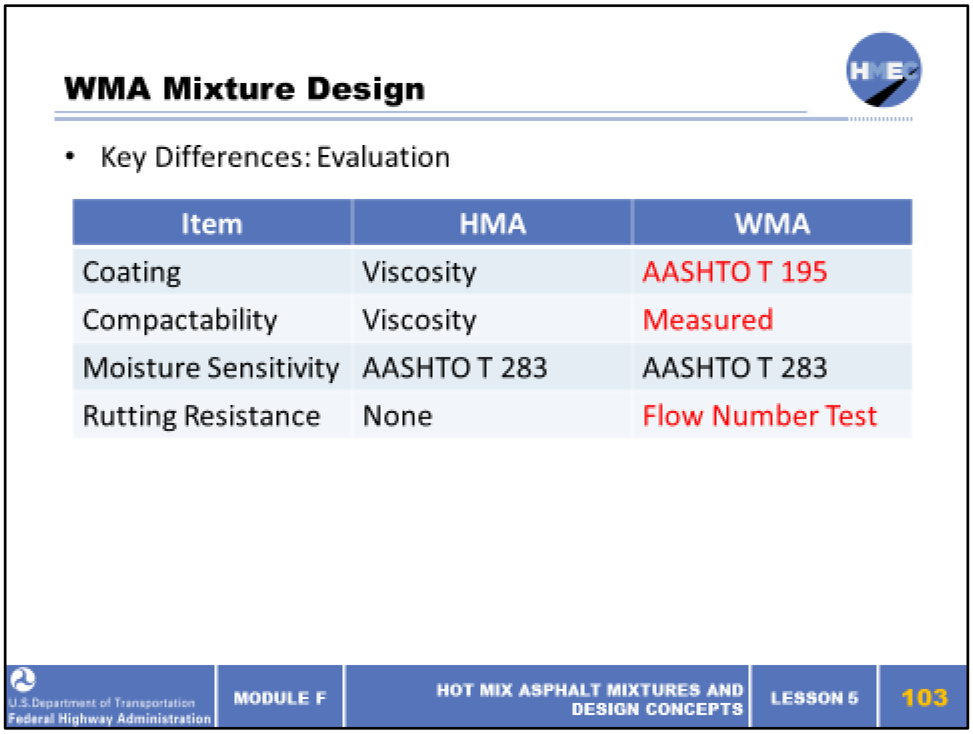
Slide 103



This chart presents a summary of the differences between HMA and WMA for the mixture evaluation portion of the mixture design process. It shows the key item, how it is addressed in current HMA design, and the similarities and differences for WMA design.

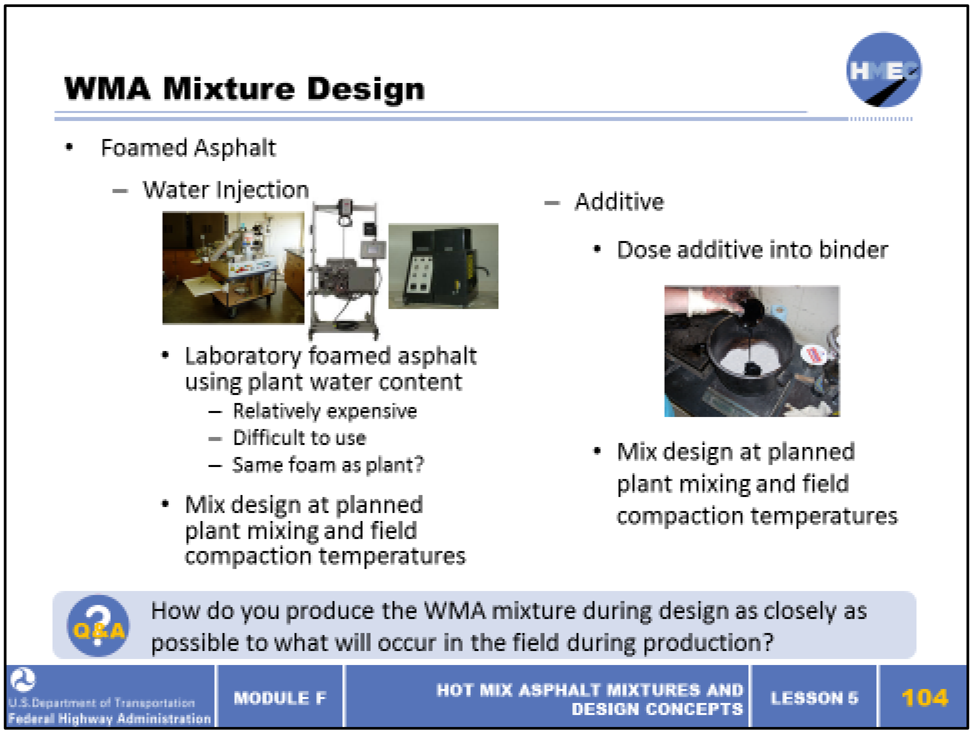
1. Coating. The viscosity based mixing temperatures used in HMA design to ensure adequate coating cannot be used with the wider range of WMA processed currently available. AASHTO T 195, Standard Method of Test for Determining Degree of Particle Coating of Bituminous-Aggregate Mixtures, is used with WMA to evaluate particle coating the planned field production temperature.

2. Compactibility. The viscosity based compaction temperatures used in HMA design cannot be used to control workability and compactibility for the wide range of WMA processes currently available. For WMA design, compactibility is measured with the gyratory compactor by evaluating the number of gyrations to reach 8% air voids at the planned field compaction temperature and 30 °C below the planned field compaction temperature.

3. Moisture Sensitivity. The same evaluation, AASHTO T 283, is recommended for WMA and HMA. The criteria for WMA are the same as HMA.

4. Rutting Resistance. Since the short-term aging of WMA mixtures will be less due to the lower temperatures, it is important to evaluate the rutting resistance of WMA mixtures. The Appendix to AASHTO R35 uses the flow number test for this evaluation. The flow number will likely be added in the future to HMA mixture design. States that currently use a wheel tracking device should also use that device for WMA mixtures.

Slide 104

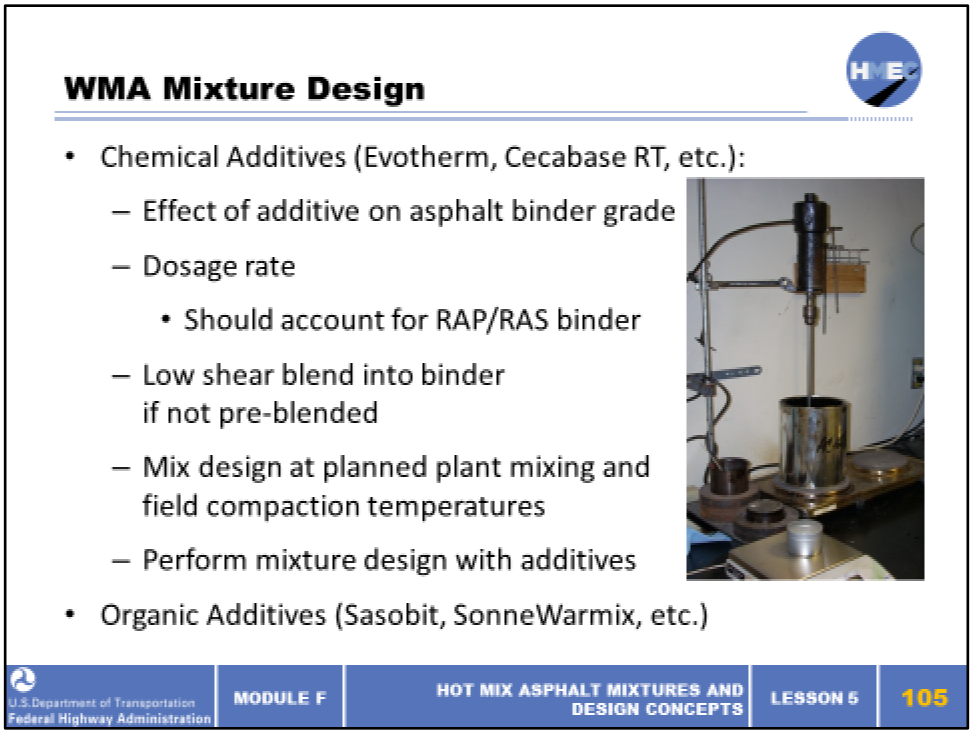


The mix design considerations designing mixtures with foamed asphalt depend on the way the foamed asphalt is produced. The mix design procedures for WMA recommend reproducing the WMA process as closely as possible.

• Water injection systems. For water injection systems, the mix design should be performed using foamed asphalt produced at the water content used in the plant. Currently, there are three laboratory foaming devices available, but there are concerns with these devices. First they are relatively expensive ($15,000 to $50,000). Second, they are somewhat difficult to use. It is difficult to control the amount of foamed asphalt produced and there are issues with clogging. Third, and probably most importantly, is whether they reasonably reproduce the foam produced by the plant systems. For these reasons, laboratory mix designs are not commonly done with foamed asphalt when water injection systems are used. A normal HMA design is used and the mix is produced with water injection systems at lower temperatures.

• Foaming additives. It is easy to add foaming additives during laboratory mix design. Use the photo to explain that after the binder is added in a pool in the mix, the foaming additive as added to the pool. The mix design them proceeds as usual at the lower temperatures for the WMA that will be produced.

Slide 105



The mix design considerations designing mixtures with chemical WMA additives are the same as those for organic WMA additives.

• Effect of the additive on the asphalt binder grade. Make sure the WMA additive (chemical or organic) does not change the grade of the asphalt binder at the dosage rate being used. You may need to change the asphalt binder to meet the specified grade for the project; especially, if you are expecting the WMA additive to improve the high temperature grade.

• Dosage rate. What dosage rate will you be using. If you are adding RAP, the dosage in the virgin asphalt binder needs to be increased to account for the RAP binder.

• Adding the additive to the mix. Normally for laboratory designs, the additive (chemical or organic) is blended into the binder using a paddle type mixer.

• Mix and compact lab specimens at planned production and compaction temperatures. After adding the additive to the asphalt binder, the mix design proceeds using the planned plant mixing and field compaction temperatures.

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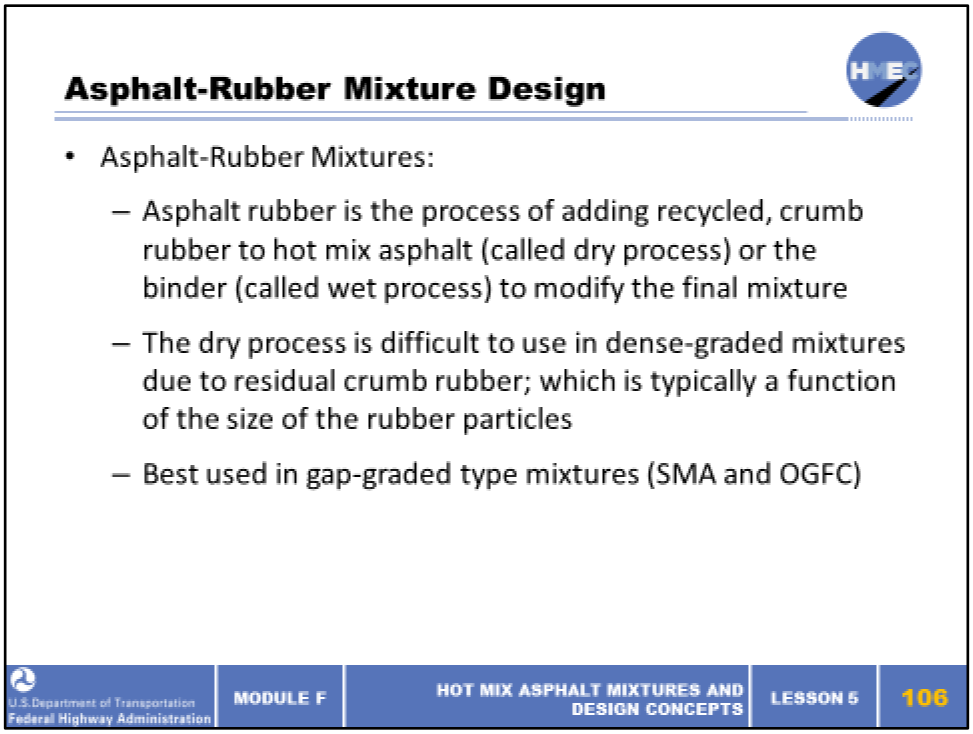
Slide 106



Asphalt rubber has been used for years in certain regional areas, but its use in OGFC (called AR-OGFC) has grown in recent years due to some research showing AR-OGFC generate lower noise levels than any other pavement surface material.

In the dry process for well-graded (not gap-graded) mixtures, those agencies that use 30-mesh particle rubber size have not observed any major problems with performance.

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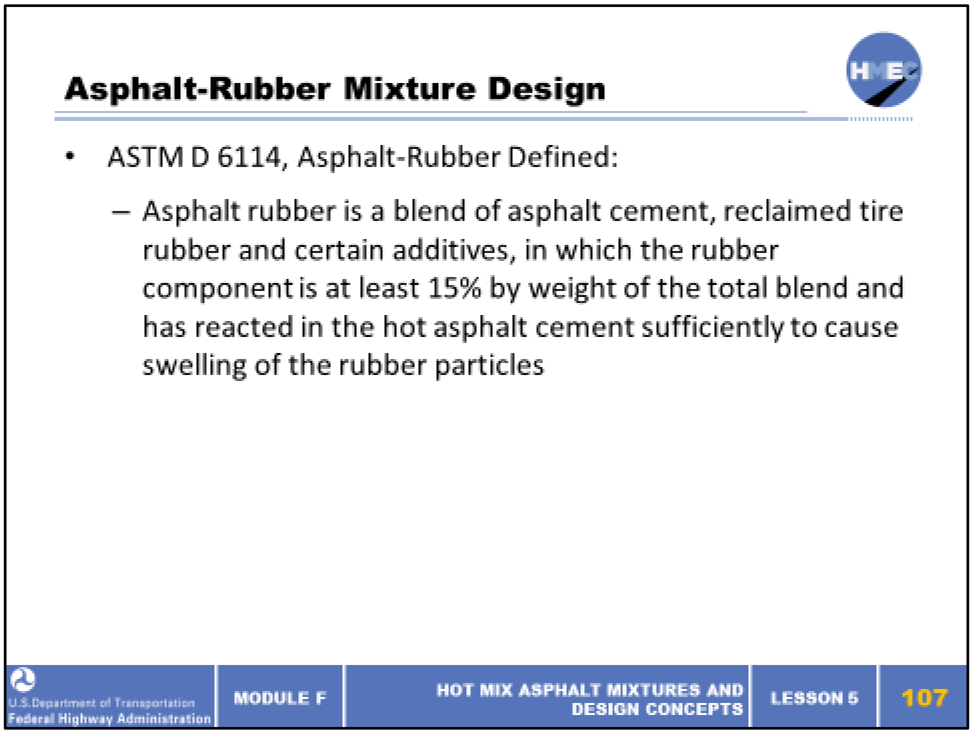

Asphalt rubber is defined in ASTM D 6114 as "a blend of asphalt cement, reclaimed tire rubber and certain additives, in which the rubber component is at least 15% by weight of the total blend and has reacted in the hot asphalt cement sufficiently to cause swelling of the rubber particles." However, this is an empirical definition based on using more tires and not an engineering based definition. In some cases, mixes require much less rubber and using the quantity identified in the ASTM definition can result in inferior mixture properties that actually reduce performance or result in mixture deterioration much sooner than for conventional dense-graded unmodified mixtures or mixture modified by other techniques.

More importantly, a number of states have successfully used asphalt rubber mixtures that do not meet this definition. Simply having less than 15 percent by weight of the total blend of the rubber component does not mean the mix is not an asphalt rubber mix.

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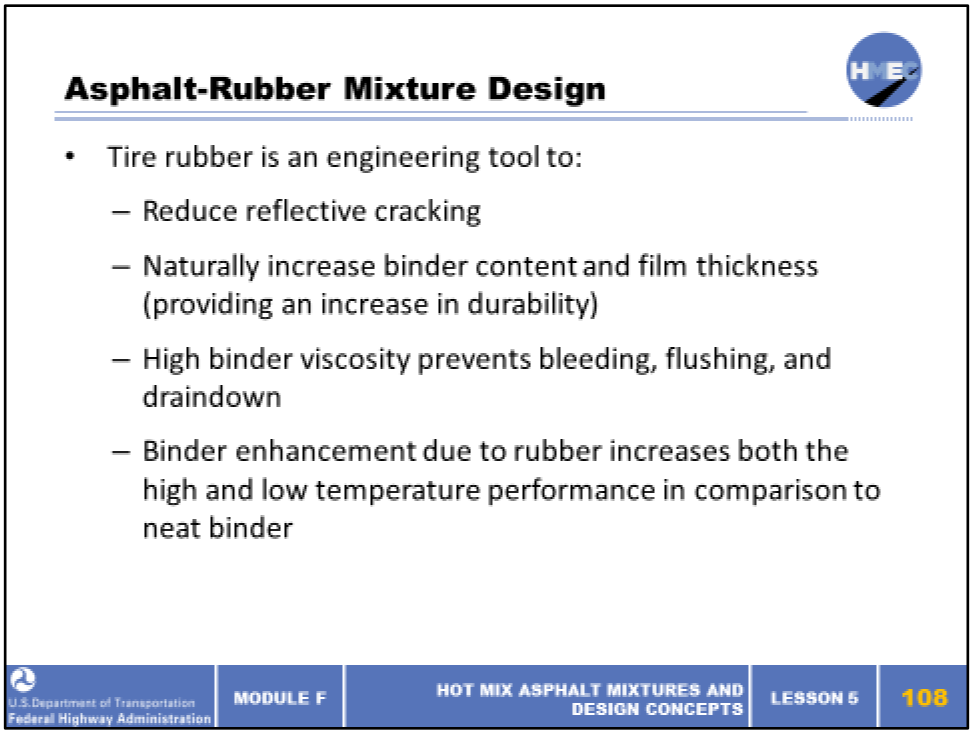
Slide 108



In general the benefits from the use of asphalt rubber are reported as the 3Es: engineering, environmental, and economic benefits. Some of the engineering benefits are listed in the slide and should be discussed in greater detail, because the reason for using asphalt rubber mixtures is mostly related to the engineering benefits.

The second E in the 3Es is the environmental benefit; the reduction of scrap tires in landfills. Typically AR-OGFC mixtures, placed approximately 1-inch thick, use over 1,000 tires per lane mile.

The third E in the 3Es is the economic benefit or savings through the use of asphalt rubber mixtures. The economic benefit is related to a cost savings in multiple areas, including: a possible reduction in the amount of binder, extended service life through more durable mixtures and reduce cracking thereby reducing the life cycle costs of a pavement structure.

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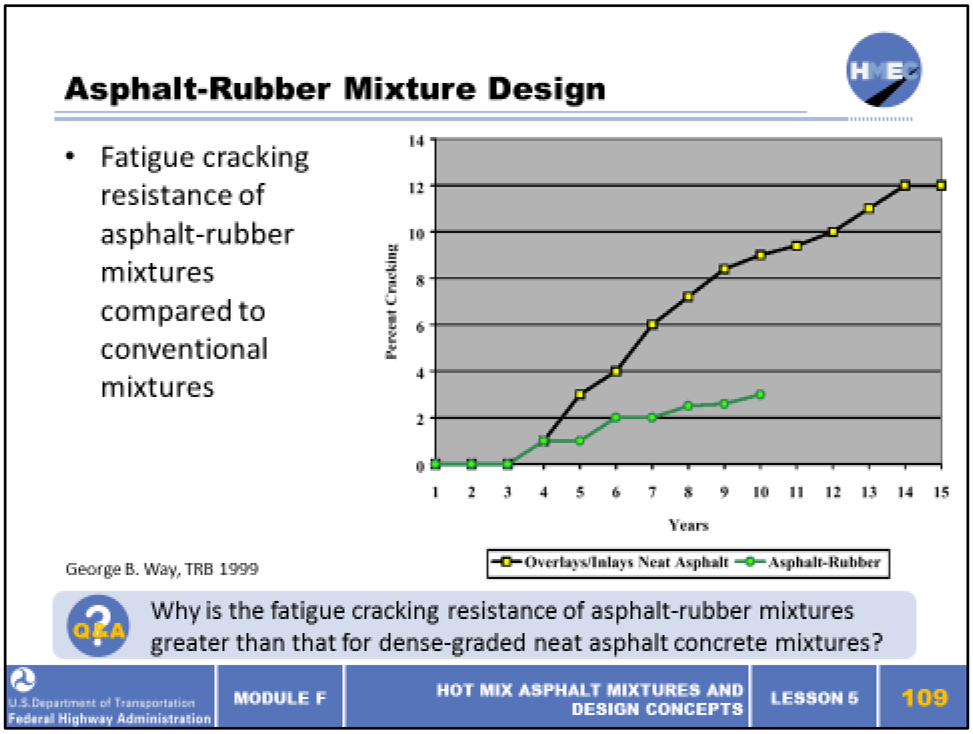
Slide 109



This chart only compares neat unmodified mixture to an asphalt rubber mixture. The asphalt rubber mixture did exhibit lower amounts of cracking. However, the neat mixture can be modified with other materials and also exhibit a reduction in cracking. The point is that many different materials can be used to improve on the properties of bituminous mixtures to achieve better performance or lower levels of distress. The key is the cost of the mixture in comparison to the neat unmodified mixture and extended service life from the modified mixture. Lesson #6 includes test procedures that can be used to measure the engineering properties of a mixture to determine or estimate the difference between two mixtures without field performance data.

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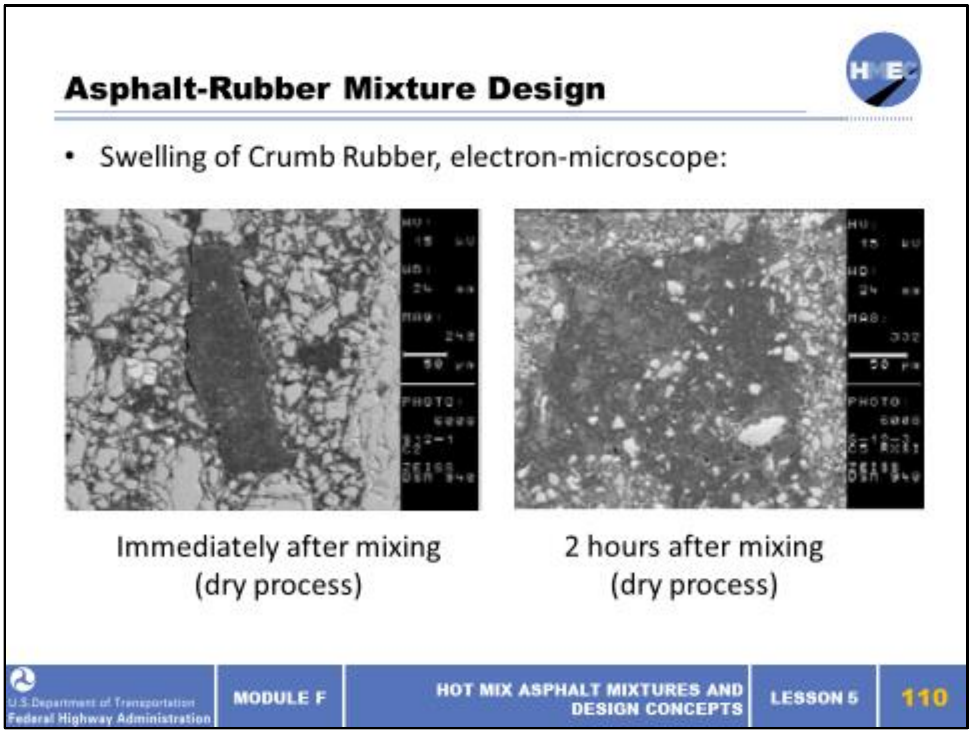
Slide 110


Asphalt-Rubber Mixture Design
• Swelling of Crumb Rubber, electron-microscope:
Immediately after mixing
(dry process)
2 hours after mixing
(dry process)
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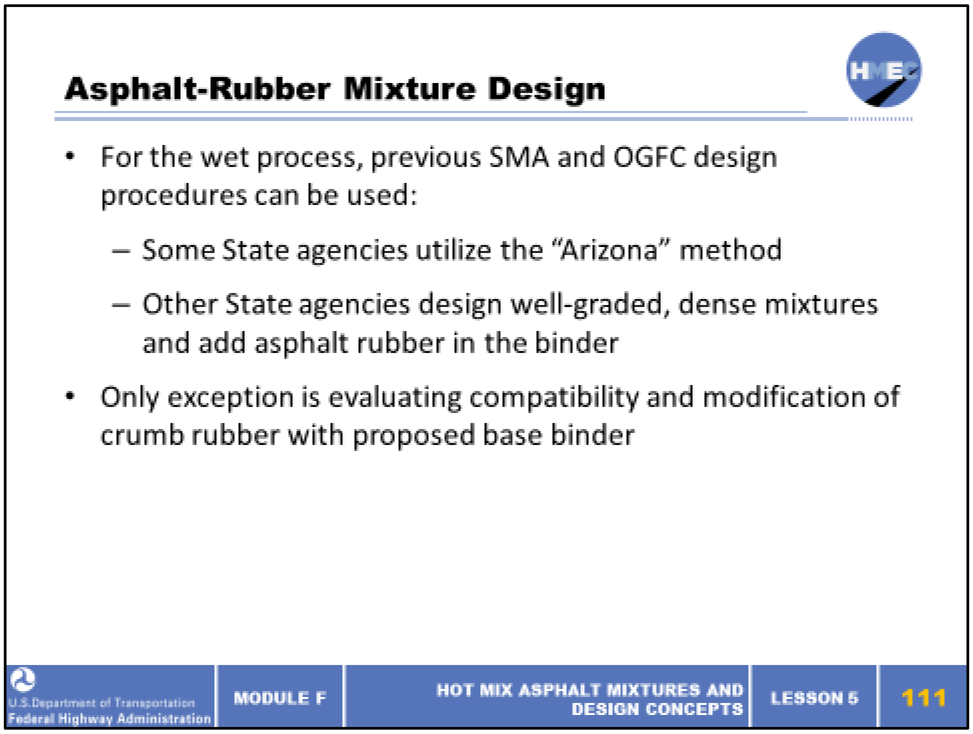

Some state agencies utilize the "Arizona" method, but it is not necessary to treat the mix like an SMA or OGFC. Some agencies have used asphalt rubber in well-graded, dense mixes for a long time and use typical Superpave design procedures previously discussed in Lesson #5. Florida currently uses rubber to modify binders to a PG 76-22 and over the past decade. The process has been found to work well.

The only exception is evaluating compatibility and modification of crumb rubber with proposed base binder. This needs to be conducted prior to the final mixture design.

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The most common of these is the viscosity test using the Haake model shown in the slide. This portable device is also used in the field during quality control.

However, grading the asphalt rubber binder in accordance with the PG system is becoming much more common. The dynamic shear rheometer (DSR), as discussed in Lessons #2 and #3, is being used to characterize binders with rubber. Some of these details in using the DSR have yet to be adopted by AASHTO, but are expected to be adopted in the near future as the particle size relative to the gap size of the DSR get worked and there is a better understanding on how the particles influence the test results.

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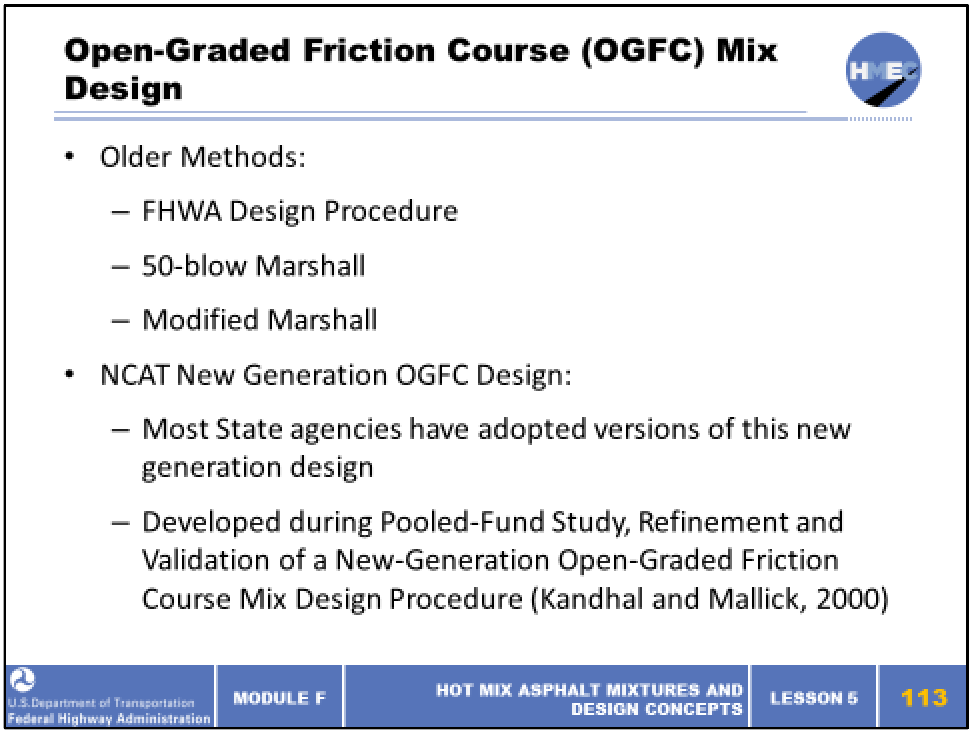
Open-Graded Friction Course (OGFC) Mix Design
HMEC
• Older Methods:
– FHWA Design Procedure
– 50-blow Marshall
– Modified Marshall
• NCAT New Generation OGFC Design:
– Most State agencies have adopted versions of this new generation design
– Developed during Pooled-Fund Study, Refinement and Validation of a New-Generation Open-Graded Friction Course Mix Design Procedure (Kandhal and Mallick, 2000)
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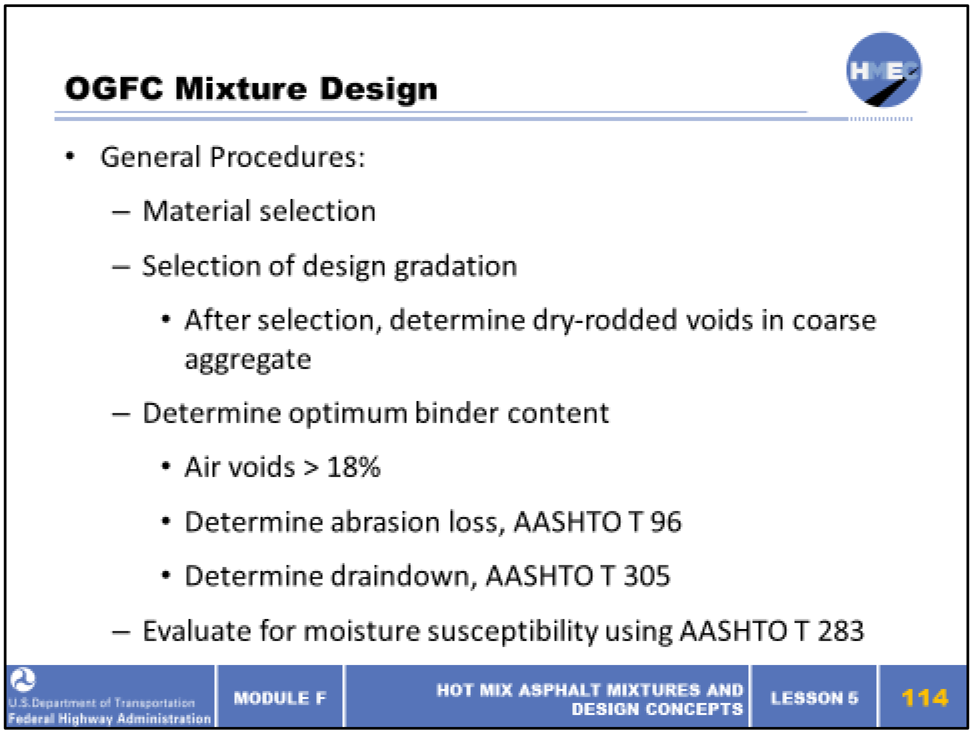
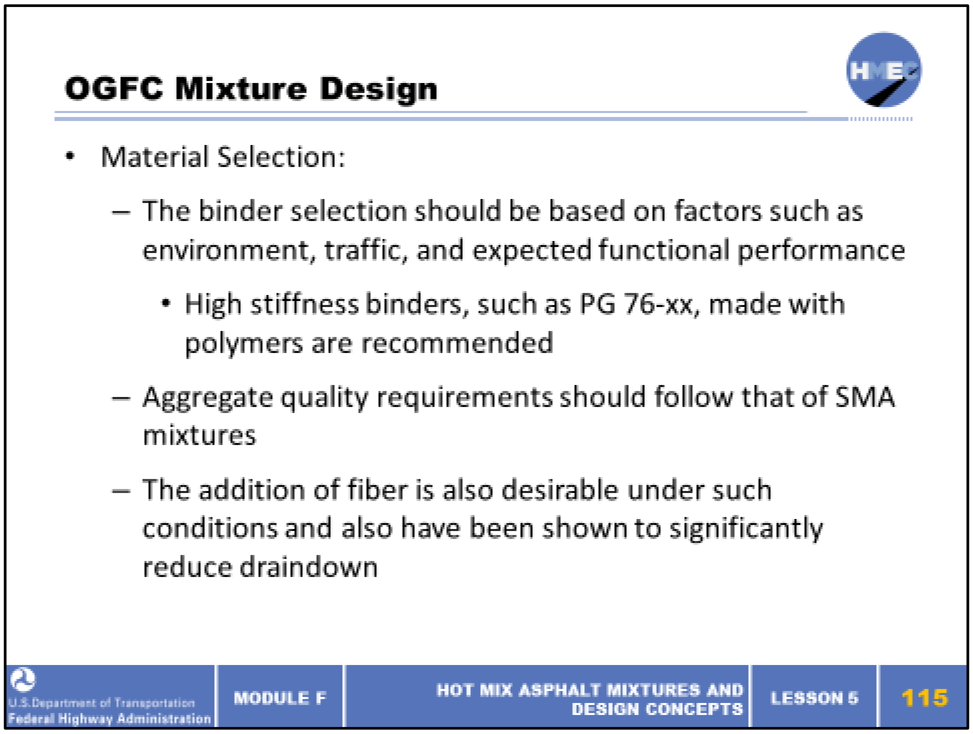


The NCAT New Generation OGFC mixture design comprises four main components.

Slide 115



These include good quality and cubical aggregates, polymer-modified binders, and stabilizing fibers.

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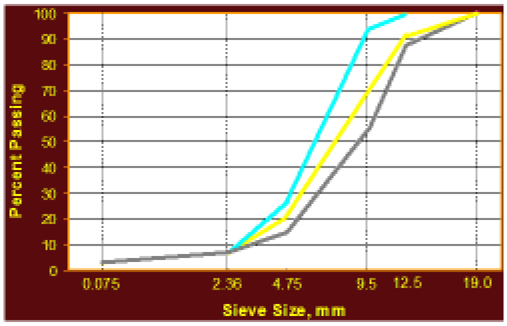
OGFC Mixture Design

- Recommended Gradation Range:



Sieve	Percent Passing
19 mm	100
12.5 mm	85-100
9.5 mm	55-75
4.75 mm	10-25
2.36 mm	5-10
0.075 mm	2-4

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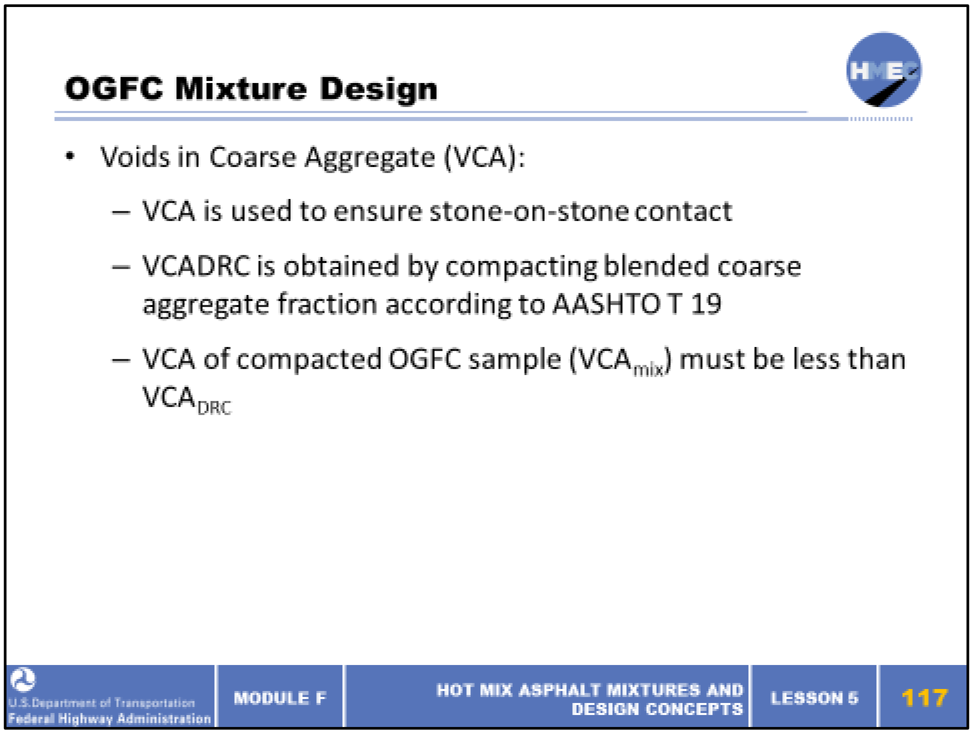

OGFC Mixture Design
HMEC
• Voids in Coarse Aggregate (VCA):
– VCA is used to ensure stone-on-stone contact
– VCADRC is obtained by compacting blended coarse aggregate fraction according to AASHTO T 19
– VCA of compacted OGFC sample (VCA_{mix}) must be less than VCA_{DRC}
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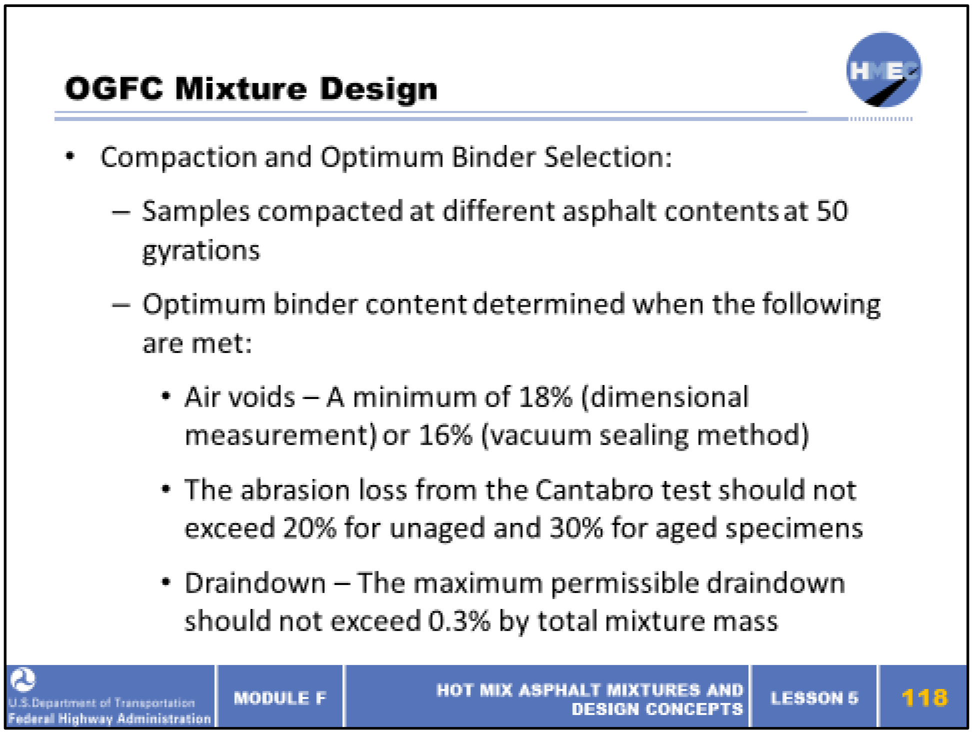
OGFC Mixture Design
• Compaction and Optimum Binder Selection:
– Samples compacted at different asphalt contents at 50 gyrations
– Optimum binder content determined when the following are met:
• Air voids – A minimum of 18% (dimensional measurement) or 16% (vacuum sealing method)
• The abrasion loss from the Cantabro test should not exceed 20% for unaged and 30% for aged specimens
• Draindown – The maximum permissible draindown should not exceed 0.3% by total mixture mass
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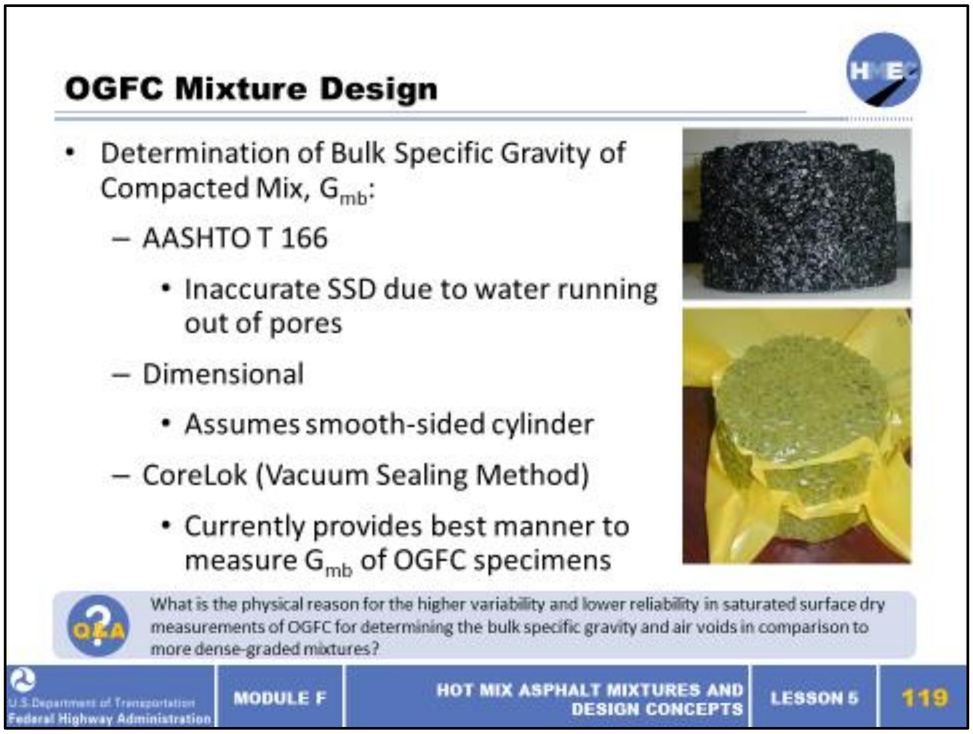

Due to the higher air void levels, water will run out of the pores, resulting in final calculations that under-determine the air void level. It is recommended to use procedures like vacuum sealing, paraffin wax sealing, or dimensional analysis to determine the bulk specific gravity of OGFC specimens.

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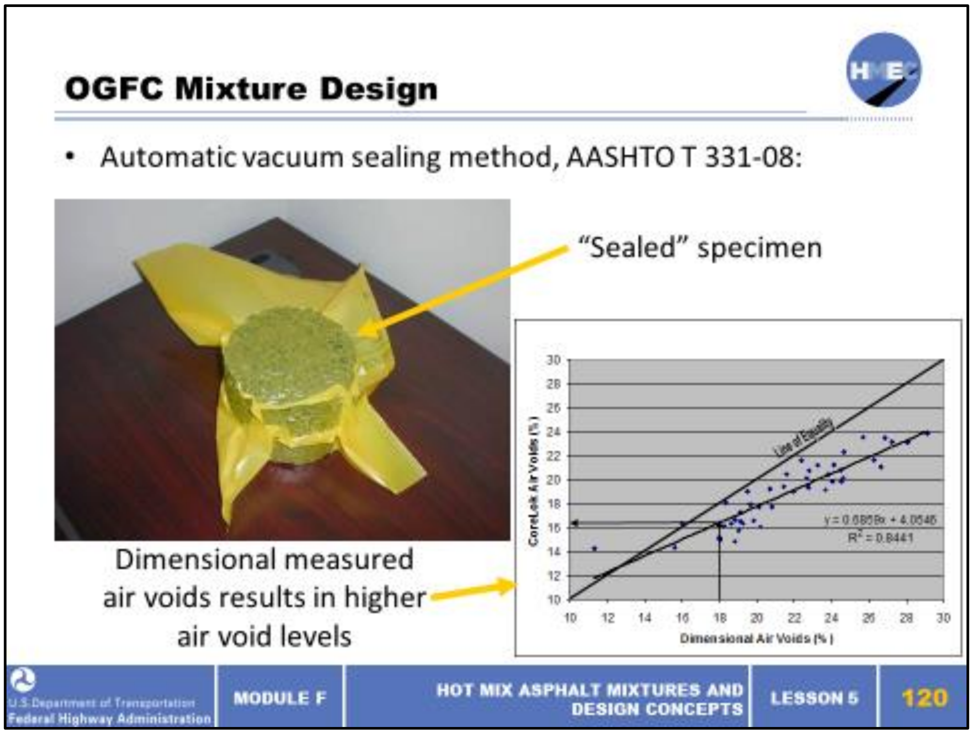
OGFC Mixture Design
• Automatic vacuum sealing method, AASHTO T 331-08:
"Sealed" specimen
Dimensional measured air voids results in higher air void levels
Line of Equality
y = 0.6859x + 4.0546
R² = 0.8441
CoreLok Air Voids (%)
Dimensional Air Voids (%)
10 12 14 16 18 20 22 24 26 28 30
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OGFC Mixture Design
HMEC
• Verification of abrasion and impact in accordance with Cantabro Testing:
– 20% maximum loss unaged
Before
After
(Take largest remaining)
U.S. Department of Transportation Federal Highway Administration
MODULE F
HOT MIX ASPHALT MIXTURES AND DESIGN CONCEPTS
LESSON 5
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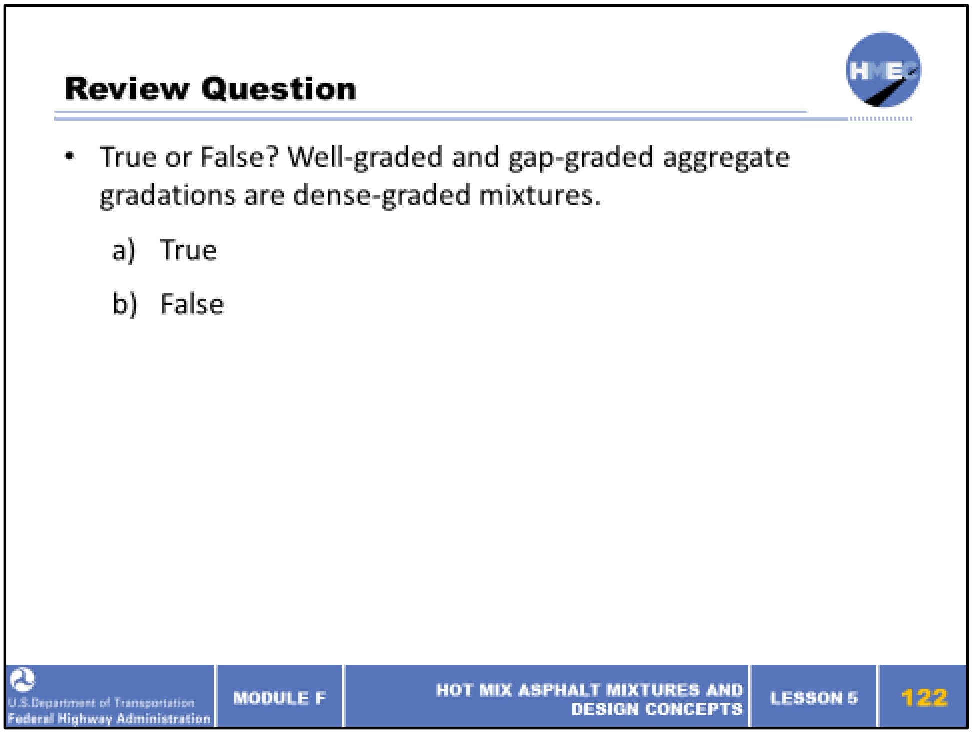
Slide 122


Review Question
True or False? Well-graded and gap-graded aggregate gradations are dense-graded mixtures.
a) True
b) False
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MODULE F
HOT MIX ASPHALT MIXTURES AND DESIGN CONCEPTS
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Slide 123

Review Question

- What does the FHWA 0.45 power gradation charts represent?
  - a) The maximum theoretical specific gravity of the aggregate blend
  - b) The theoretical maximum density of the aggregate blend
  - c) An aggregate blend that requires a higher amount of binder

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Review Question

- For dense, well-graded mixtures, what air void level is used to select the target binder content?
  a) 10.5%
  b) 7%
  c) 4%
  d) 2%
  e) None of the above

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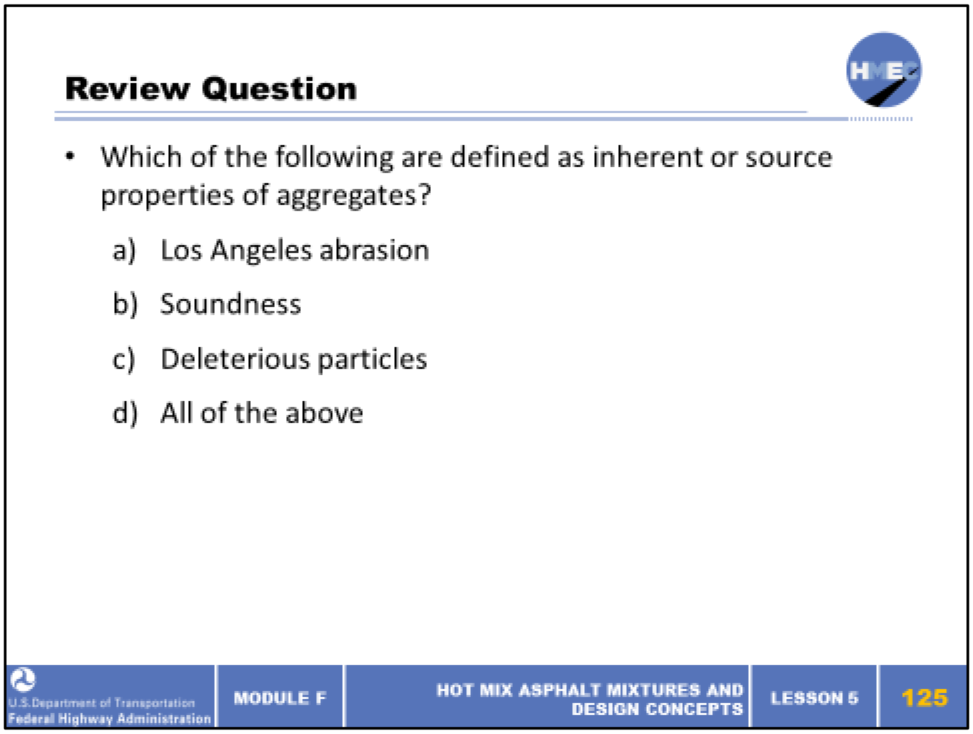
Slide 125


Review Question
HMEC
• Which of the following are defined as inherent or source properties of aggregates?
a) Los Angeles abrasion
b) Soundness
c) Deleterious particles
d) All of the above
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MODULE F
HOT MIX ASPHALT MIXTURES AND DESIGN CONCEPTS
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Review Question

- True or False? $N_{initial}$ is known as the tenderness check on the compaction curve, while N_{max} is known as the rutting check.
  - a) True
  - b) False

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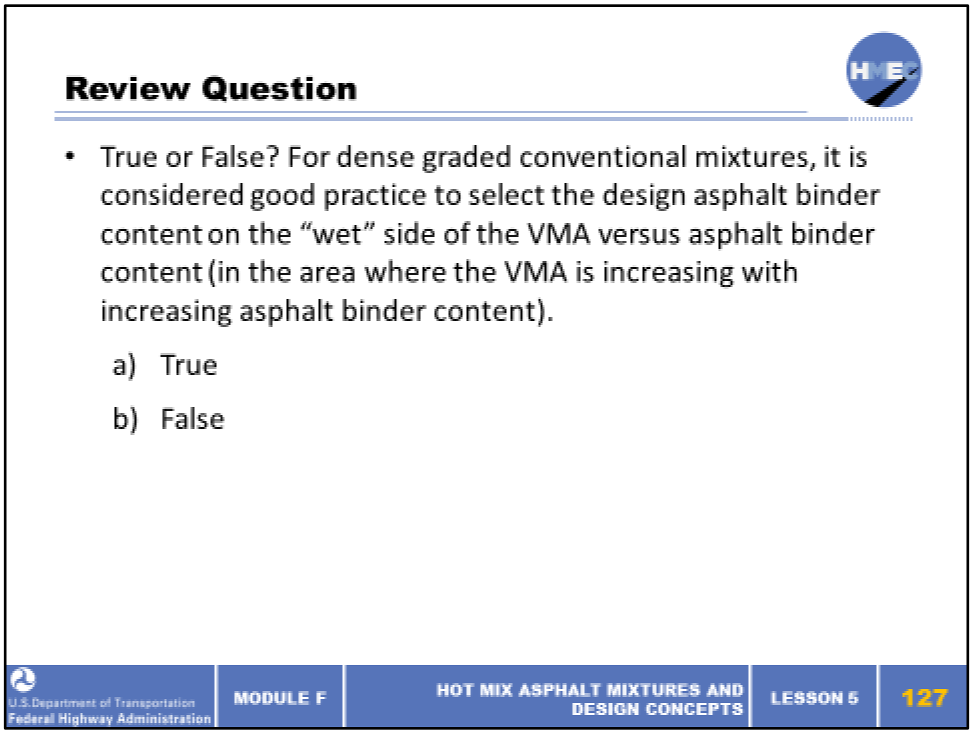
Review Question
HMEC
• True or False? For dense graded conventional mixtures, it is considered good practice to select the design asphalt binder content on the “wet” side of the VMA versus asphalt binder content (in the area where the VMA is increasing with increasing asphalt binder content).
a) True
b) False
U.S. Department of Transportation
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HOT MIX ASPHALT MIXTURES AND DESIGN CONCEPTS
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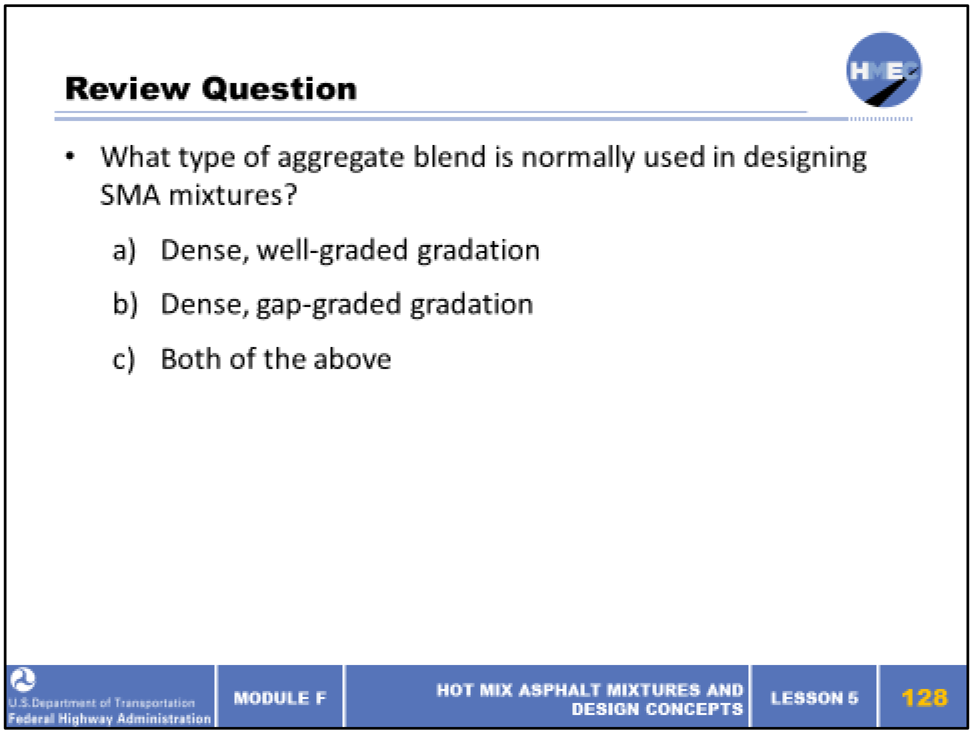
Slide 128


Review Question
HMEC
• What type of aggregate blend is normally used in designing SMA mixtures?
a) Dense, well-graded gradation
b) Dense, gap-graded gradation
c) Both of the above
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LESSON 5
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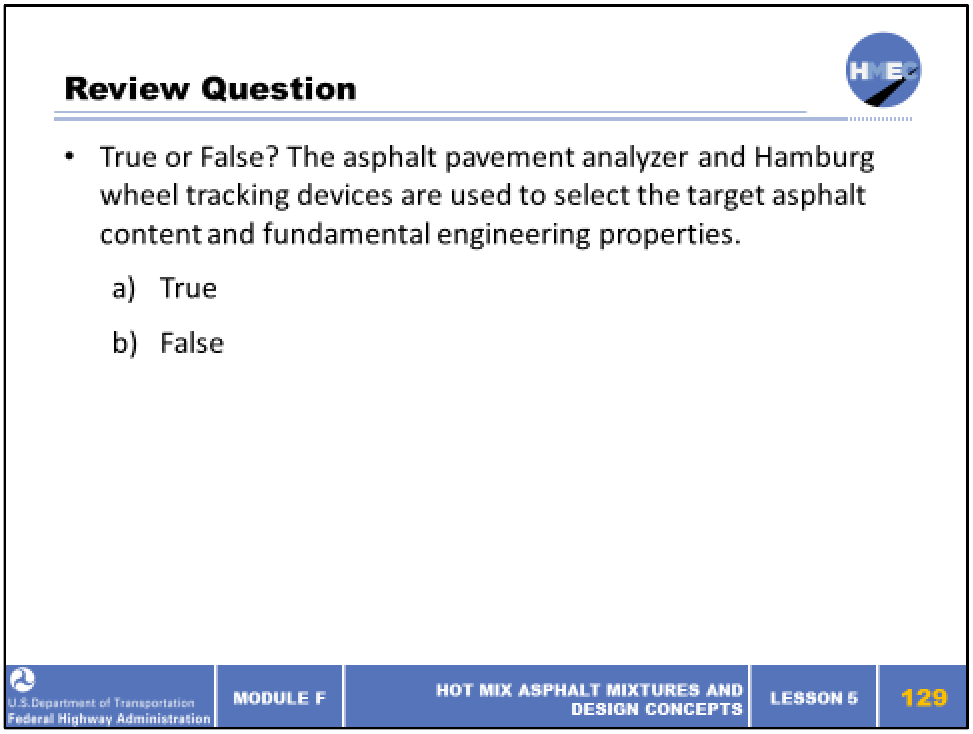
Slide 129


Review Question
HMEC
• True or False? The asphalt pavement analyzer and Hamburg wheel tracking devices are used to select the target asphalt content and fundamental engineering properties.
a) True
b) False
U.S. Department of Transportation
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HOT MIX ASPHALT MIXTURES AND DESIGN CONCEPTS
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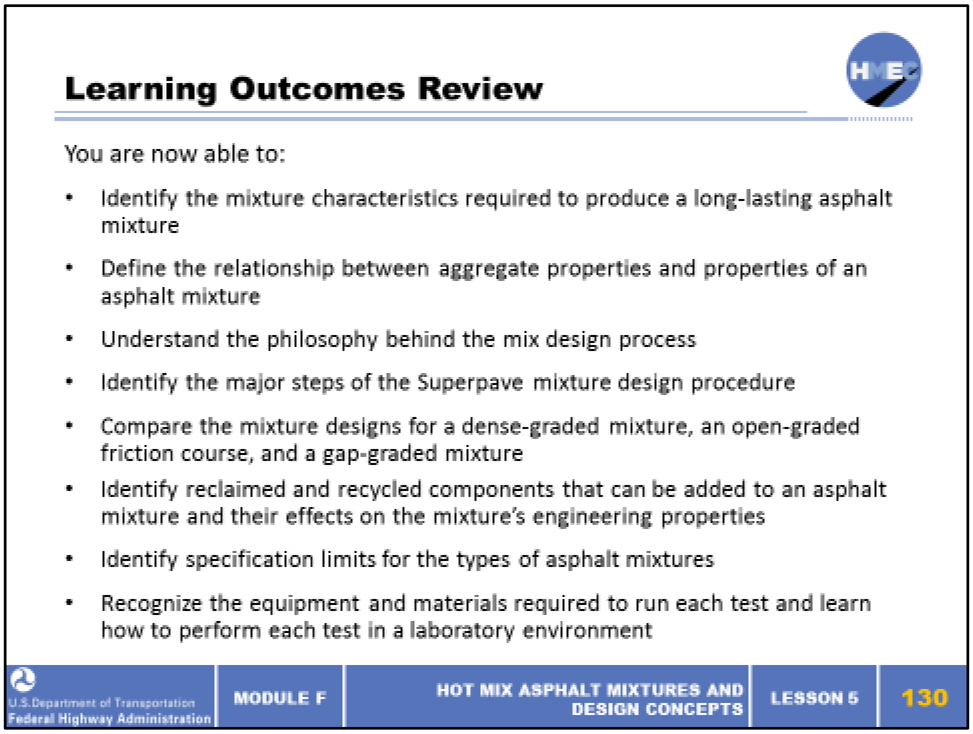
Slide 130


Learning Outcomes Review
HMEC
You are now able to:
• Identify the mixture characteristics required to produce a long-lasting asphalt mixture
• Define the relationship between aggregate properties and properties of an asphalt mixture
• Understand the philosophy behind the mix design process
• Identify the major steps of the Superpave mixture design procedure
• Compare the mixture designs for a dense-graded mixture, an open-graded friction course, and a gap-graded mixture
• Identify reclaimed and recycled components that can be added to an asphalt mixture and their effects on the mixture's engineering properties
• Identify specification limits for the types of asphalt mixtures
• Recognize the equipment and materials required to run each test and learn how to perform each test in a laboratory environment
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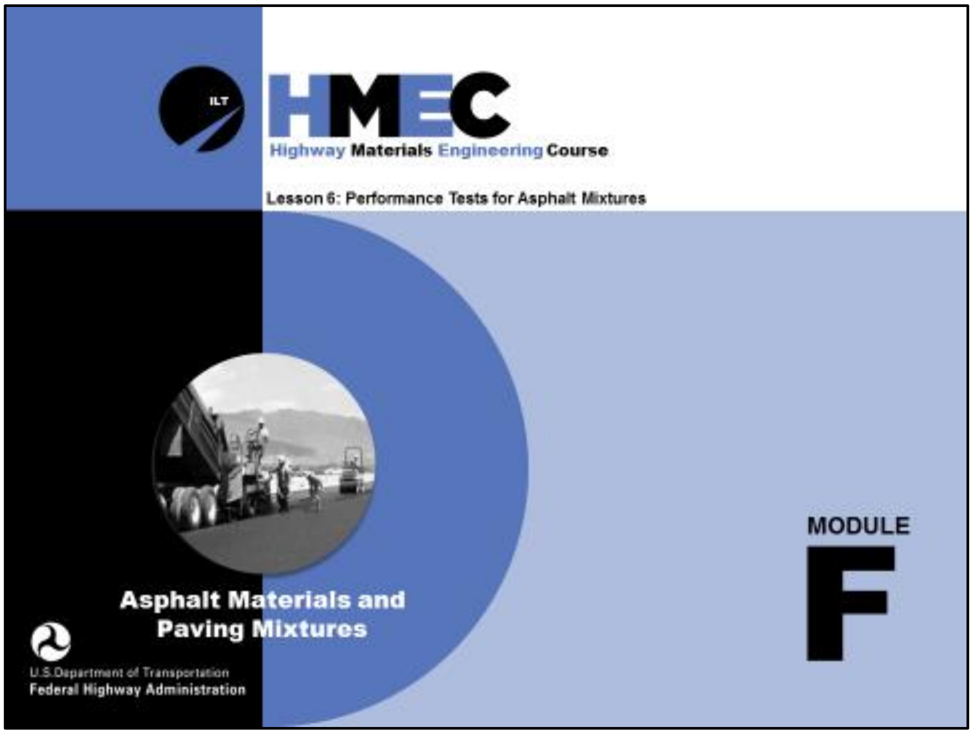
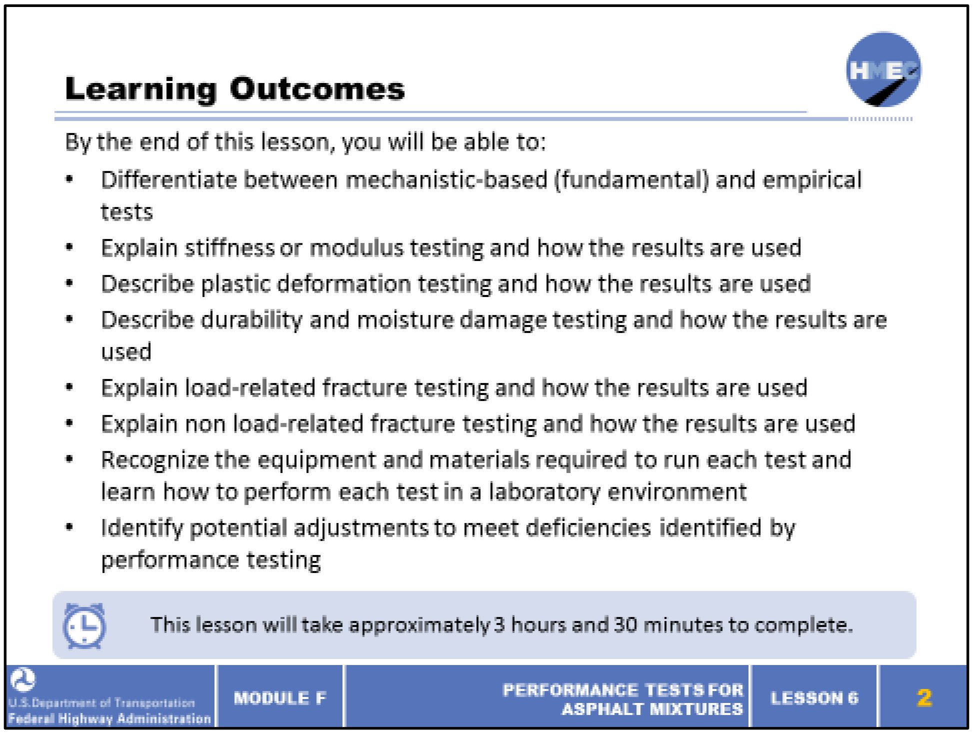
Slide 1


ILT
HMEC
Highway Materials Engineering Course
Lesson 6: Performance Tests for Asphalt Mixtures
Asphalt Materials and Paving Mixtures
U.S.Department of Transportation
Federal Highway Administration
MODULE
F

Slide 2


Learning Outcomes
HMEC
By the end of this lesson, you will be able to:
• Differentiate between mechanistic-based (fundamental) and empirical tests
• Explain stiffness or modulus testing and how the results are used
• Describe plastic deformation testing and how the results are used
• Describe durability and moisture damage testing and how the results are used
• Explain load-related fracture testing and how the results are used
• Explain non load-related fracture testing and how the results are used
• Recognize the equipment and materials required to run each test and learn how to perform each test in a laboratory environment
• Identify potential adjustments to meet deficiencies identified by performance testing
This lesson will take approximately 3 hours and 30 minutes to complete.
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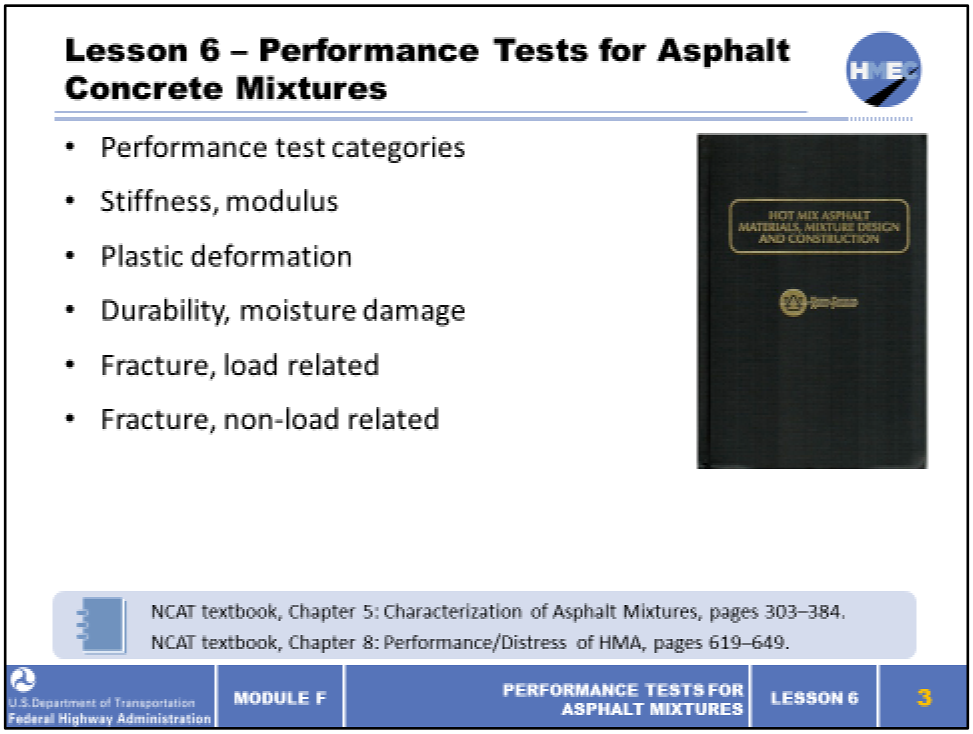
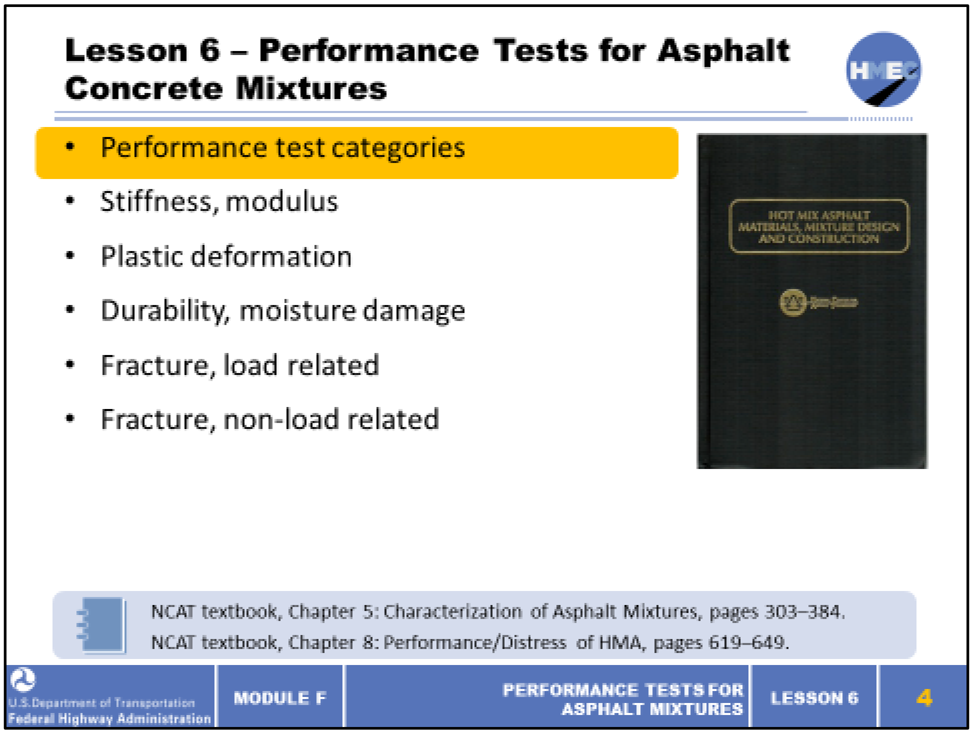
MODULE F
PERFORMANCE TESTS FOR ASPHALT MIXTURES
LESSON 6
2

Slide 3

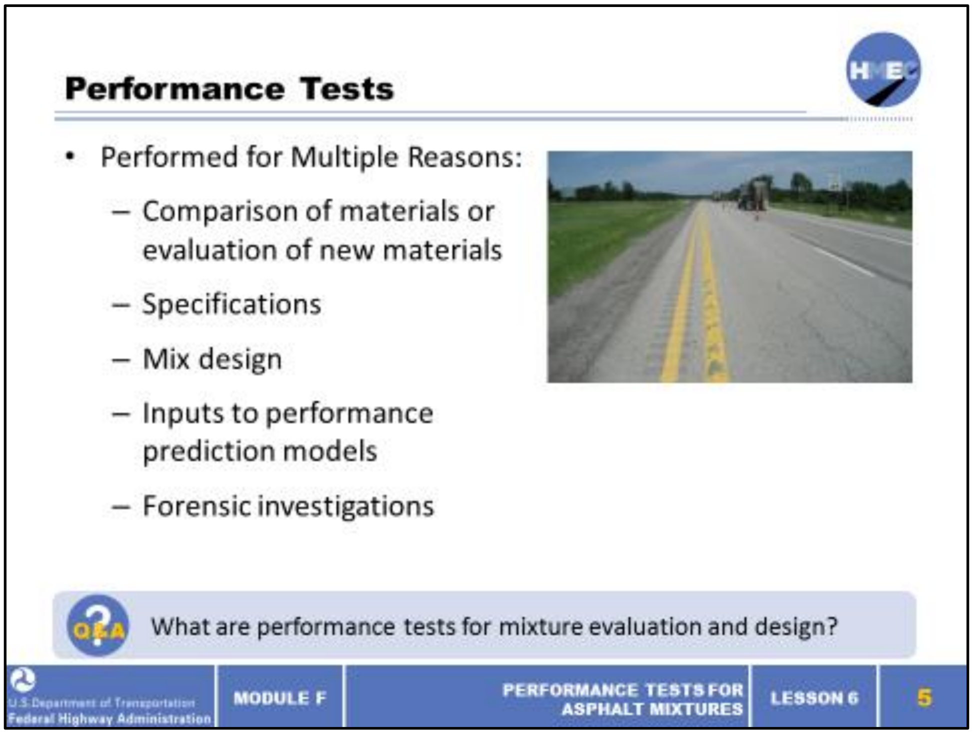

Lesson 6 – Performance Tests for Asphalt Concrete Mixtures
HMEC
• Performance test categories
• Stiffness, modulus
• Plastic deformation
• Durability, moisture damage
• Fracture, load related
• Fracture, non-load related
HOT MIX ASPHALT MATERIALS, MIXTURE DESIGN AND CONSTRUCTION
NCAT textbook, Chapter 5: Characterization of Asphalt Mixtures, pages 303–384.
NCAT textbook, Chapter 8: Performance/Distress of HMA, pages 619–649.
U.S. Department of Transportation
Federal Highway Administration
MODULE F
PERFORMANCE TESTS FOR ASPHALT MIXTURES
LESSON 6
3

Slide 4



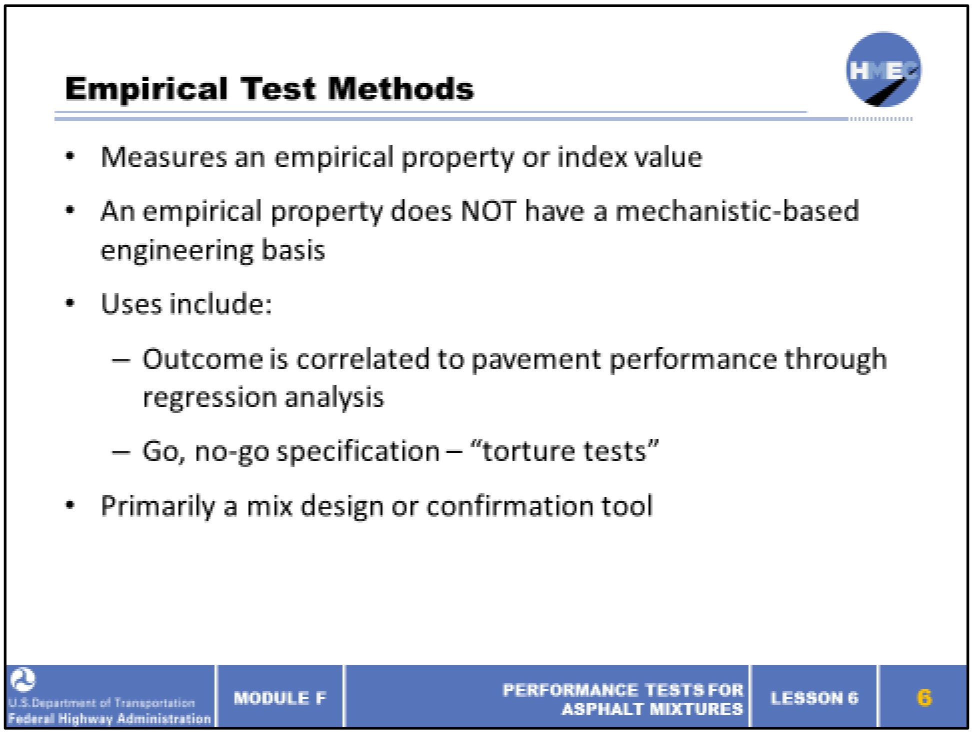
Although the need for materials characterization is not new, there has been little agreement or consensus throughout industry on which tests should be adopted and used. As such, there are many fundamental tests for measuring fundamental properties of asphalt concrete mixtures for use in evaluating performance and predicting distress. This lesson identifies the ones that are more commonly used, as well as some of the newer tests. It is not a complete listing of all fundamental tests that are documented in the literature.

Slide 5



Mixture characterization tests to predict performance are generally used for pavement design and analysis. For these processes, pavement response to load in the field may be predicted using multilayer elastic theory or finite element analysis techniques. Fundamental material properties are required as inputs to these procedures that provide stresses and strains in a pavement structure for a given set of material properties and loading conditions. Conversely, mixture characterization tests can also be used for confirming a mixture design and for comparing different mixtures under different conditions. Empirical material properties or test outcomes can be used for this purpose.

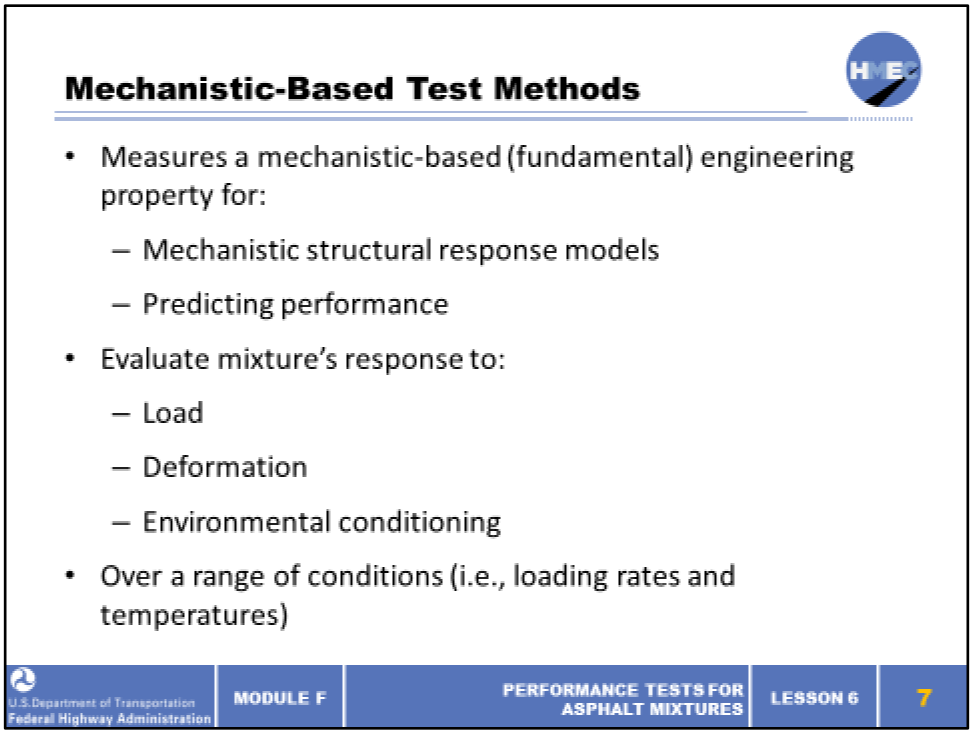
Slide 6



The measured property or outcome of an empirical test is essentially an index. An example of an empirical property would be Marshall stability. This is simply an indicator of the stability of a mixture, not a mechanistic-based engineering (fundamental) property of the mixture.

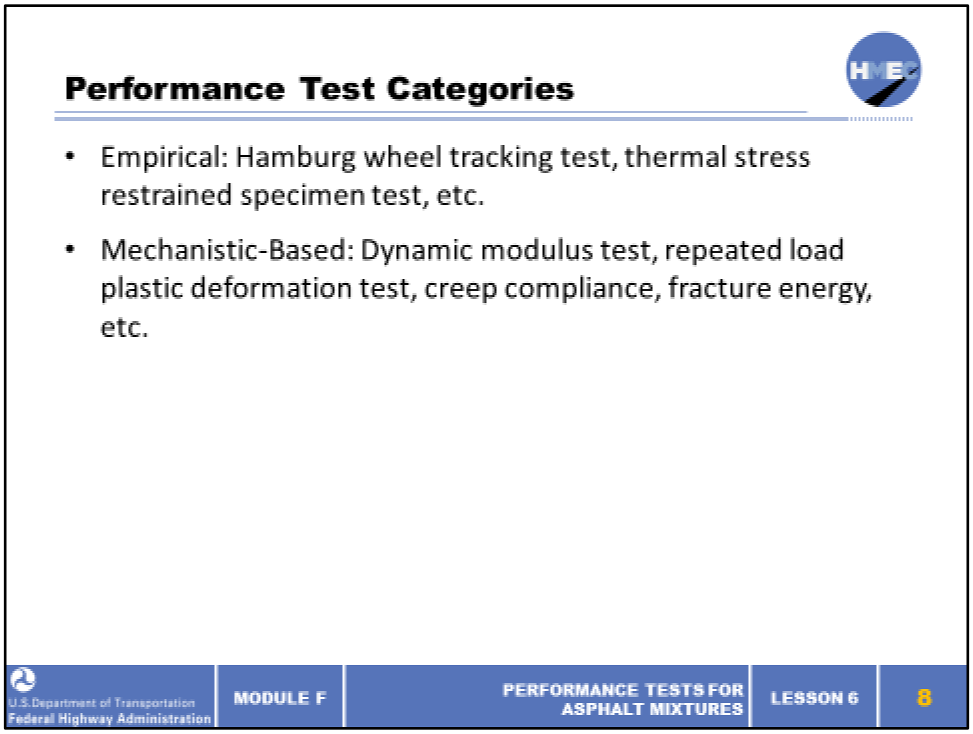
The use of empirical test results is limited by the fact that the properties measured with them do not have a mechanistic-based engineering basis. They are typically used as pass/fail or go/no-go tests in specifications. They may also be correlated to performance using linear regression techniques. They cannot, however, be used to predict performance history of a mixture.

Slide 7



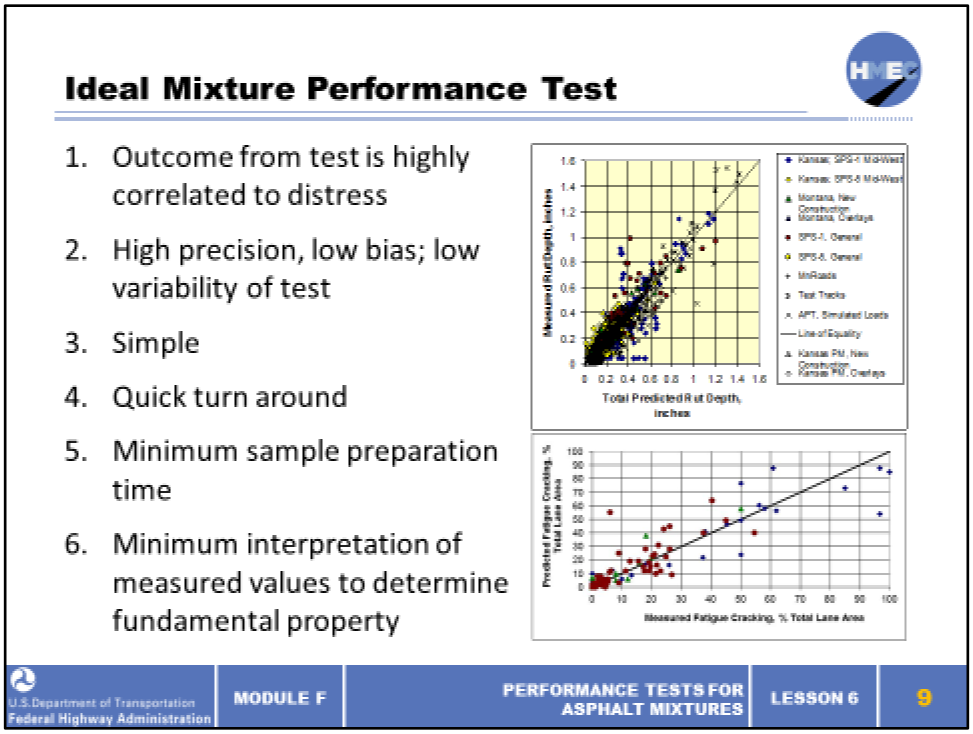
The use of mechanistic-based test results is not limited like empirical test results are. They may be used as pass/fail or go/no-go tests in specifications, and they may also be correlated to performance using linear regression techniques just as empirical test can. More importantly, however, they can also be used to predict performance history using mechanics of materials principles. Thus, mechanistic-based tests can be used for mechanistic-empirical pavement design purposes.

Slide 8



Empirical tests have been more commonly used but are less reliable and the results should be used with caution in extrapolating the results outside the infraspace from which the test and its criterion were developed. However, the use of mechanistic-based engineering properties and the tests for measuring those properties is increasing. A reason for this increase in use of the mechanistic-based properties is related to the development and use of mechanistic and empirical-mechanistic pavement design procedures. One example is the Mechanistic-Empirical Pavement Design Guide (MEPDG), which will be discussed in detail in Module E.

Slide 9


Ideal Mixture Performance Test
1. Outcome from test is highly correlated to distress
2. High precision, low bias; low variability of test
3. Simple
4. Quick turn around
5. Minimum sample preparation time
6. Minimum interpretation of measured values to determine fundamental property
Measured Rut Depth, inches
Total Predicted Rut Depth, inches
Measured Fatigue Cracking, % Total Lane Area
U.S. Department of Transportation
Federal Highway Administration
MODULE F
PERFORMANCE TESTS FOR ASPHALT MIXTURES
LESSON 6
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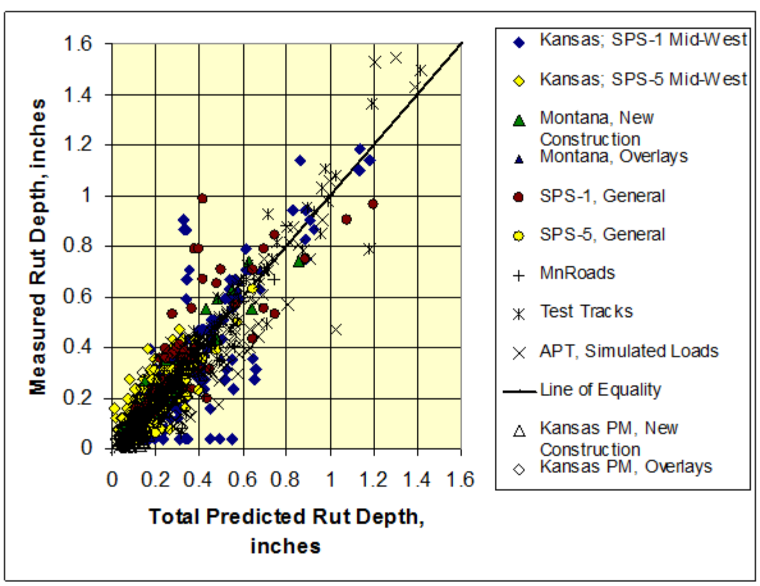
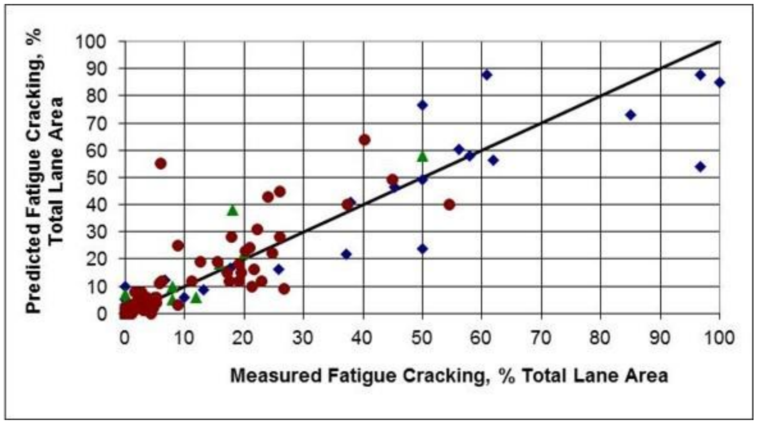

Measured Rut Depth, inches
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0 0.2 0.4 0.6 0.8 1 1.2 1.4 1.6
Total Predicted Rut Depth, inches
Kansas; SPS-1 Mid-West
Kansas; SPS-5 Mid-West
Montana, New Construction
Montana, Overlays
SPS-1, General
SPS-5, General
MnRoads
Test Tracks
APT, Simulated Loads
Line of Equality
Kansas PM, New Construction
Kansas PM, Overlays



The sensitivity of the test method to material variability as well as the sensitivity of performance predictions made using the test results must be considered. If a test is highly sensitive to material variability and it cannot be controlled well in preparing test specimens, then the test probably should not be used. Or if used, the results should be viewed with caution.

It is also important to consider the sensitivity of performance predictions to the inherent variability associated with a performance test method prior to using it. The repeatability and reproducibility of a test can be established and used in performance prediction simulations to assess the sensitivity of the predictions to the test method variability. Repeatability and reproducibility were defined and discussed during Module A.

Examples of critical specimen preparation include compaction and aging methods. Examples of critical testing conditions that should be considered include use of loads/stresses to simulate 80 kN axle loads and testing at the maximum or minimum temperatures that a pavement is expected to be exposed to.

The ideal mixture characterization test would provide mixture response to the wide range of loading and environmental conditions a mixture would be exposed to in-service.

• Because asphalt concrete is a viscous-elastic material, the ideal test would have to provide an indication of elastic, viscous, viscoelastic, and plastic behavior of the mixture.
• The test would give the load-deformation and strength properties over the range of expected field conditions.
• This is an extremely complicated request, and the reality is that a single mixture characterization test capable of providing all this does not exist today. It is certain, however, that the test would have to provide fundamental material properties.

The practicality of a performance test should also be considered before initiating a testing program. Specimen preparation requirements and test duration should both be given ample consideration. For less important projects there are indirect methods of estimating material properties that may be more appropriate than performing extensive, expensive performance testing. This is particularly important relative to the size of the project. If a mixture is being used as a functional overlay on a county road as opposed to for a new lane on an uphill grade on an interstate expected to be exposed to 100 million equivalent single-axle loads (ESALs) over its design life, then different levels of mixture characterization testing can be justified.

Slide 10

Performance Indicators

Structural Distresses	Predicted by the MEPDG	Functional Distresses	Predicted by the MEPDG
Rutting	Yes	IRI	Yes
Bottom-Up Fatigue Cracks	Yes	Low Temperature Cracks	Yes
Top-Down Fatigue Cracks	No	Reflection Cracks	Yes
Slippage Cracks	No	Block Cracks	No
Edge Cracks	No	Raveling	No
Long. Joint Deterioration	No	Bleeding and Flushing	No
Potholes	No	Shoving	No

The Mechanistic-Empirical Pavement Design Guide (MEPDG) is discussed in Module E.

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Flexible pavements exhibit a range of distresses or performance indicators, which have been grouped into structural and functional distresses. Some of the structural and functional distresses are predicted by the MEPDG. The models or transfer functions used to predict these distresses define the fundamental mixture properties needed for predicting distress over time.

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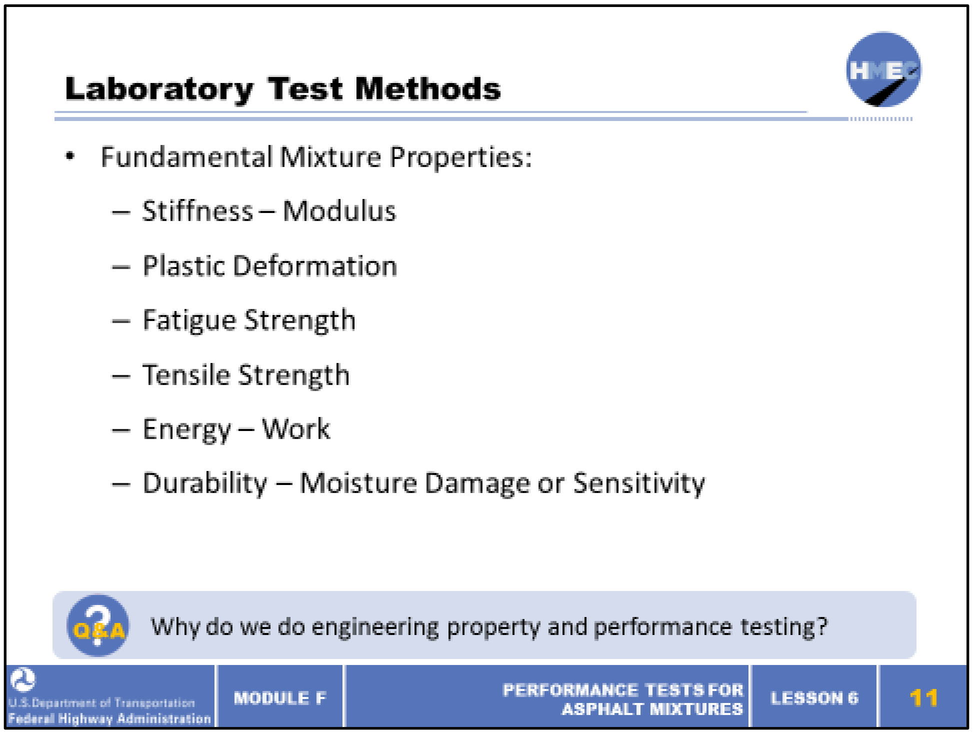

Mixture characterization or performance tests are grouped by the fundamental or empirical property they measure, and the definition of fundamental and empirical were previously defined and explained within this lesson. In some cases, there is a gray area between the fundamental and empirical tests. In other words, empirical tests can have a fundamental basis of development but their results do not provide a fundamental mix property for predicting a distress or performance indicator over time. The important point is that the correlation to performance needs to be defined and confirmed for both the fundamental and empirical tests. Empirical tests to estimate fundamental properties or performance through correlations can certainly be used but are generally confined to the conditions from which the correlations and criteria were developed. Extrapolations beyond the development conditions should be used with caution.

Slide 12

Complete the Chart:
Laboratory Test Methods by Distress

Test Designation	Performance Indicator			
	Rutting	Load Cracks	Temp. Cracks	Durability
Dynamic Modulus				
Resilient Modulus				
Flow Number				
RLPD				
High Temp. IDT				
APA				
Hamburg				
IDT – Moisture Damage				
Cantabro				
Intermediate Temp. IDT				
Fatigue Strength				
DC(T), Fracture Energy				
Overlay Tester				
IDT Creep Compliance				

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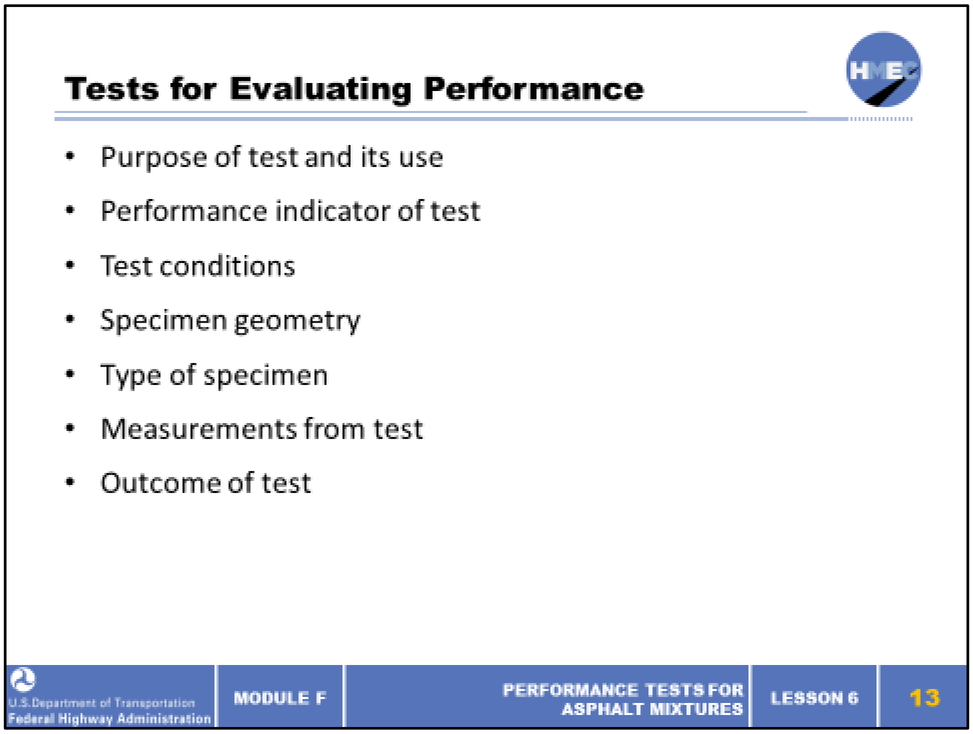
Complete this chart as the lesson progresses. Put an "M" in the appropriate columns for each type of test that is mechanistic-based and an "E" in the columns that are empirical-based.

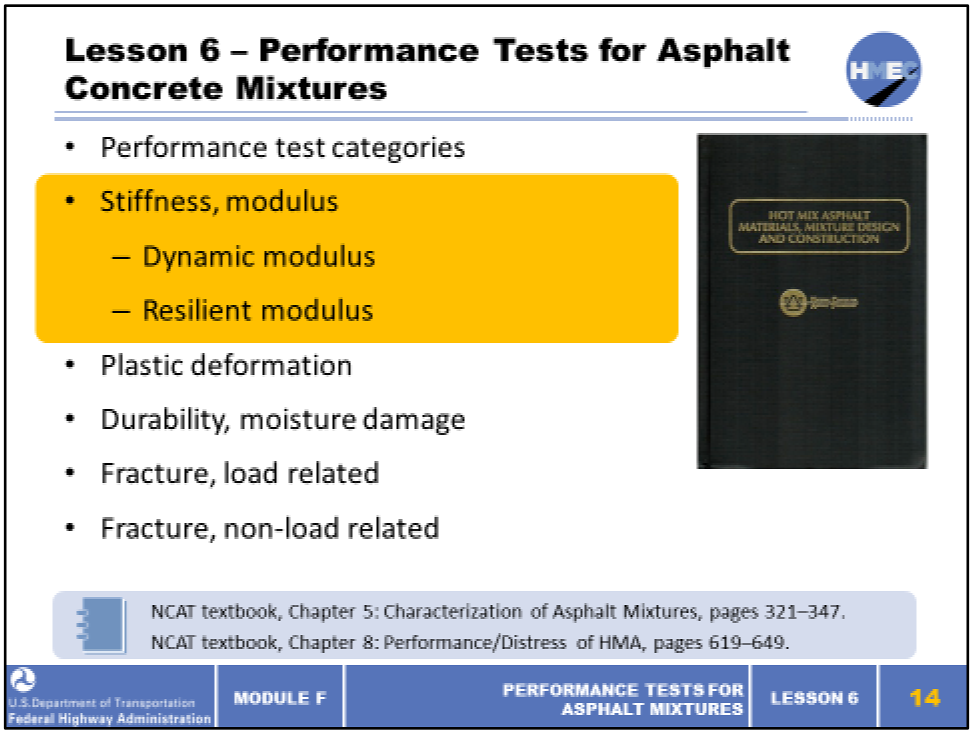
Mechanistic-based engineering (fundamental) and empirical tests are included within this table and discussed within this lesson because both groups or categories are being used for mixture design. It is important for participants to understand the advantages and limitations of each, and how the test results or outcomes are being used.

Slide 13



Seven factors are typically used in explaining and discussing different test methods and protocols, as noted above. Obviously, many other factors are important and needed to understand the test and the outcomes from those tests, but these provide the basic information for presenting each test method in a limited time frame.

Slide 14



Material stiffness is a mechanistic property that can provide valuable information when comparing different mixtures. For example, asphalt concrete mixtures that obtain higher stiffness at higher temperatures are more resistant to rutting. Conversely, asphalt concrete mixtures that obtain lower stiffness at lower temperatures can be more resistant to fatigue-type cracks at lower temperatures.

Slide 15

Tests for Evaluating Performance

Dynamic Modulus Test	
Test Designation	AASHTO T 342/TP 79 and ASTM D3497
Purpose of Test and Its Use	Measure of stiffness; provides inputs to the MEPDG and other structural response models; mix design and acceptance
Performance Indicator	Rutting and fatigue cracking
Test Conditions	Stress controlled; uniaxial, unconfined; range of temperatures and loading frequencies
Specimen Geometry	Cylindrical; height to diameter ratio requirement
Type of Specimen	Laboratory gyratory compacted and short-term aged
Measurement	Axial load and vertical displacement with time and load cycles
Outcome of Test	Dynamic modulus and phase angle as a function of temperature & loading frequency

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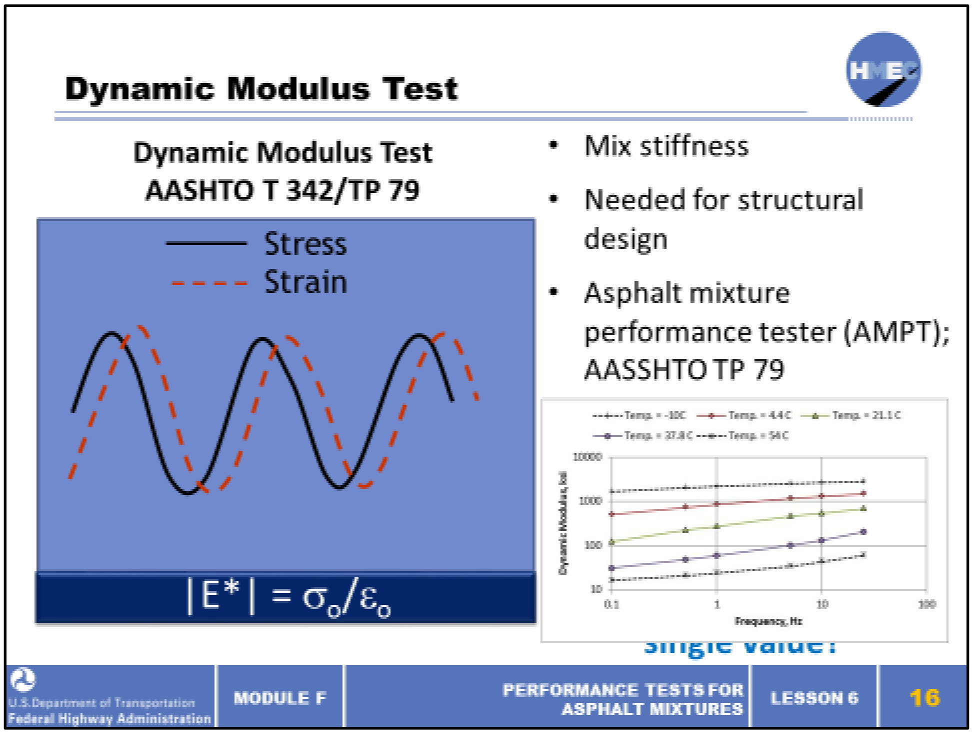
The more common test procedure to determine the mixture stiffness of asphalt concrete mixtures is the dynamic modulus test, specified in AASHTO T 342/TP 79.

- The dynamic modulus test consists of evaluating the mixture stiffness at various test temperatures and loading frequencies to simulate potential field conditions.
- Because of this, the dynamic modulus values are the input parameters in the MEPDG.
- The testing is conducted within the materials linear elastic range so that minimal to no permanent strain occurs.

Slide 16


Dynamic Modulus Test
HMEC
Dynamic Modulus Test
AASHTO T 342/TP 79
Stress
Strain
|E*| = σo/εo
• Mix stiffness
• Needed for structural design
• Asphalt mixture performance tester (AMPT); AASSHTO TP 79
MODULE F
PERFORMANCE TESTS FOR ASPHALT MIXTURES
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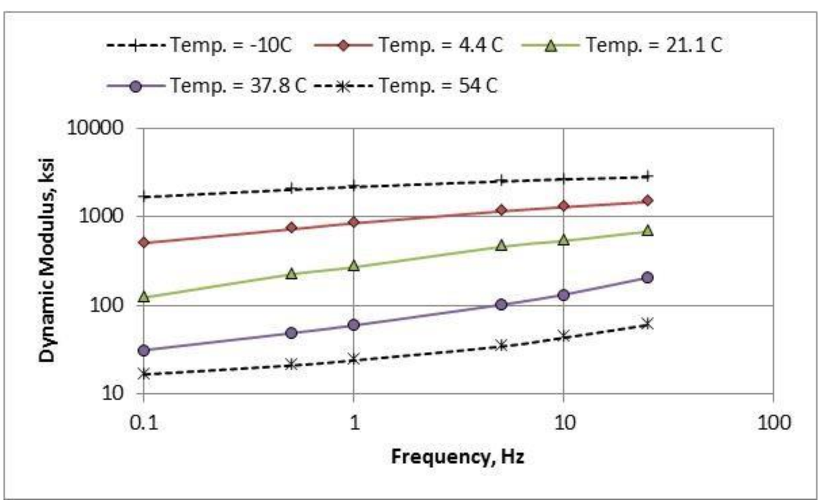
16
U.S. Department of Transportation
Federal Highway Administration

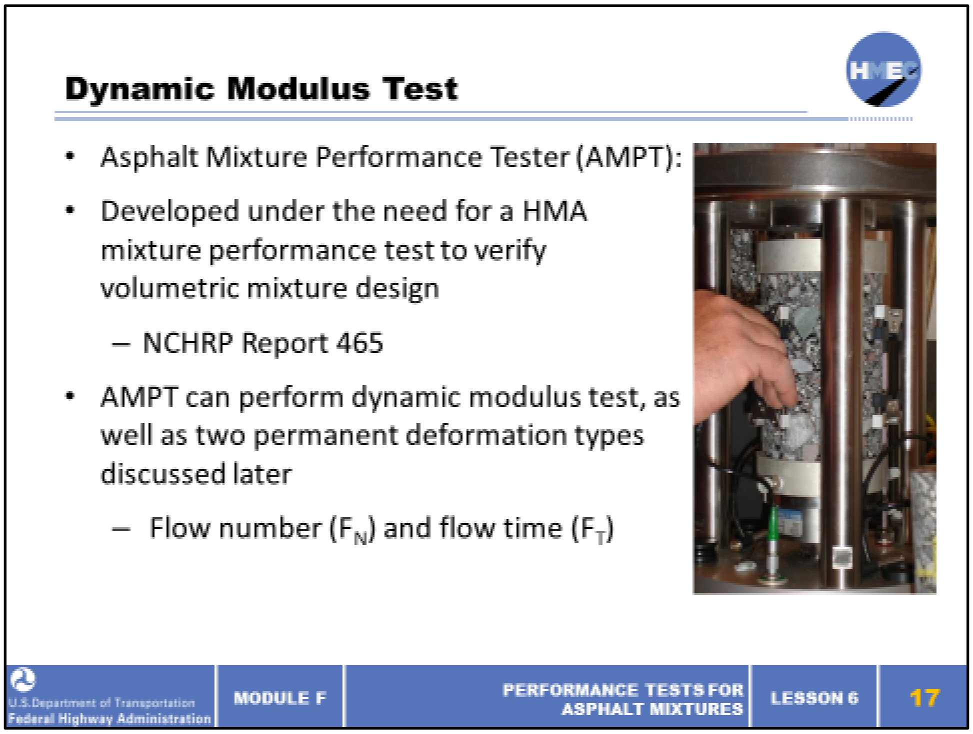

Temp. = -10C
Temp. = 4.4 C
Temp. = 21.1 C
Temp. = 37.8 C
Temp. = 54 C
Dynamic Modulus, ksi
10000
1000
100
10
0.1
1
10
100
Frequency, Hz

The dynamic modulus test is a stress-controlled, uniaxial (unconfined) test. The deviator or uniaxial stress is applied using a sinusoidal waveform, where both the applied stress and resultant vertical deformations are measured. The peak applied stress and the peak vertical deformation are used to calculate the dynamic modulus. The phase angle of the mixture is also measured during testing.

The phase angle is described as the time lag difference between the applied stress and the measured deformation. The phase angle provides information regarding the general elasticity of the mixture.

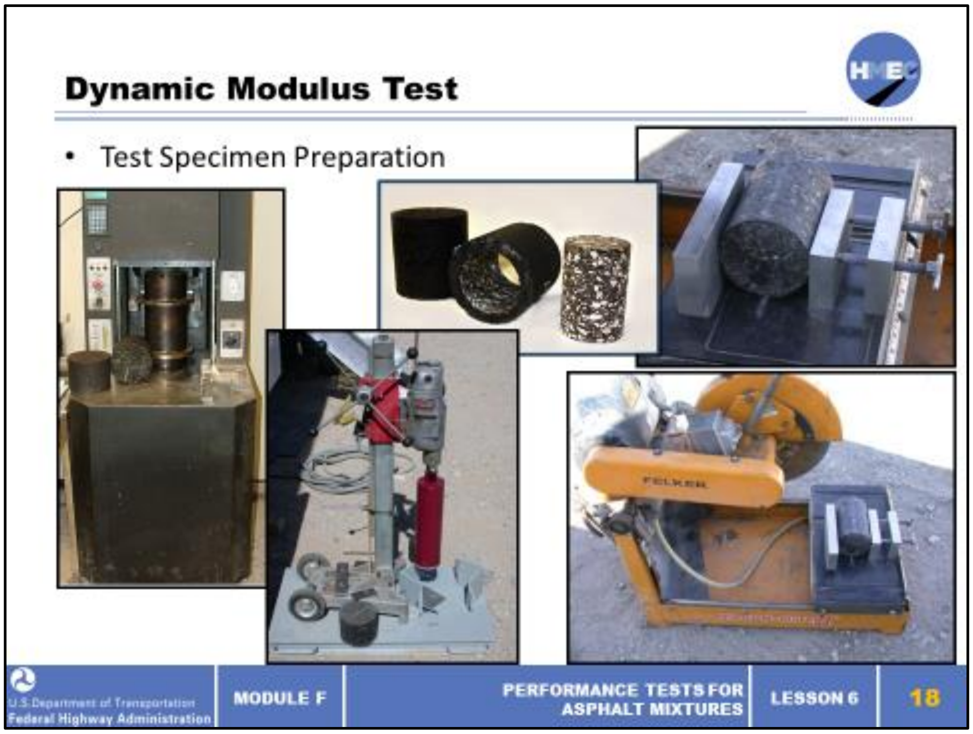
At low phase angle values, the material behaves more like a linear, elastic material. At higher phase angle values, the material begins to behave as a viscous material. A perfectly elastic material has a phase angle of zero, while a viscous material has a phase angle of 90 degrees.

Slide 17



The AMPT was developed under NCHRP Project 9-29, specifically to provide high-quality test data for hot mix asphalt (HMA) mix design performance verification. NCHRP Project 9-29 encompassed the development of an asphalt mixture testing device that could characterize the asphalt mixture (i.e., mixture stiffness and rutting performance). Future modifications are being considered to include a fatigue type test in the AMPT.

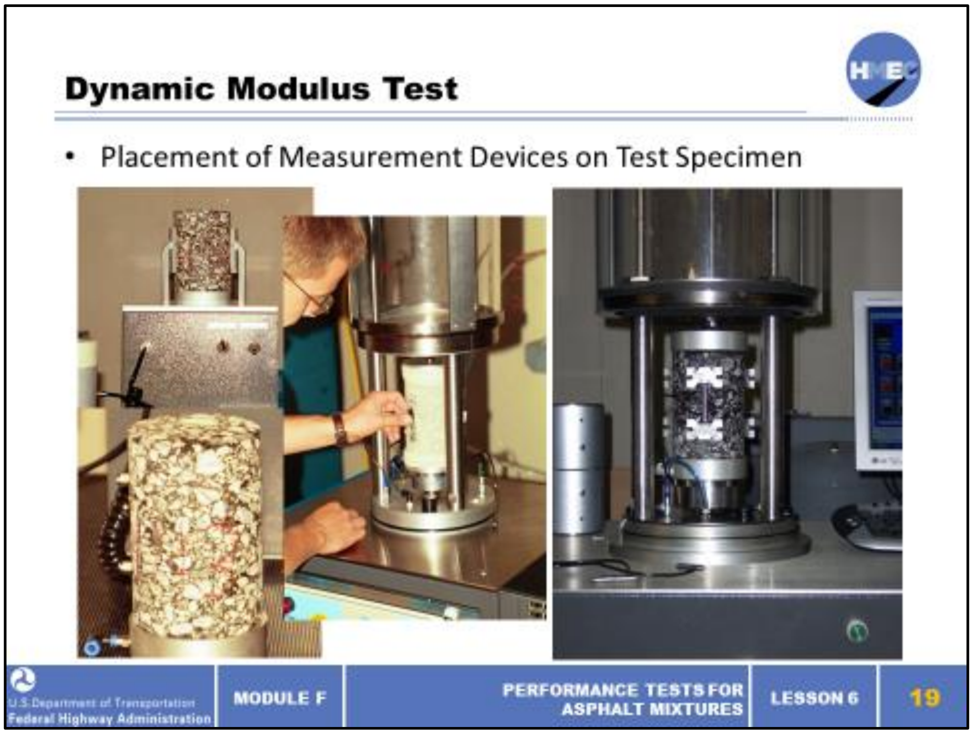
Slide 18



Dynamic modulus test specimens are produced by compacting tall gyratory specimens (at least 170 mm tall and 150 mm in diameter) and then coring out the inner 100 mm diameter core. The ends of the cored specimen are then trimmed to a final height of 150 mm.

This is to achieve uniform air void distribution (density) throughout the sample. Previous research studies have shown that the gyratory compactor produces specimens with unequal air void distributions throughout the specimen—air void levels increase towards the outside of the gyratory cylinder. More importantly, the allowable variation in air voids from the target value is plus or minus 0.5% for dynamic modulus testing. However, industry is looking into increasing that value to plus and minus 1%.

Slide 19



The dynamic modulus test is usually performed without any confinement, but confinement can be applied. In this case, membranes are placed over the specimen and the LVDTs are attached to the specimen through the membrane and sealed, as shown in the illustration to the left. The illustration on the right is an unconfined, uniaxial test specimen.

There are devices available to assist in attaching the LVDTs to the specimen to ensure the uniaxial deformation devices are placed at the correct location and properly aligned on the specimen.

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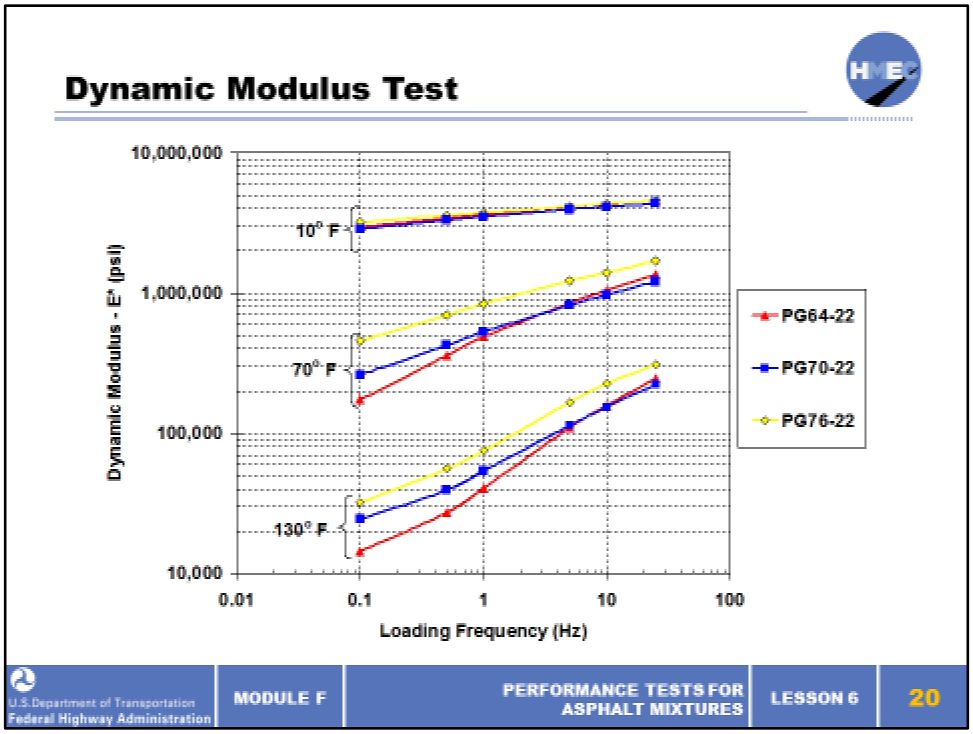
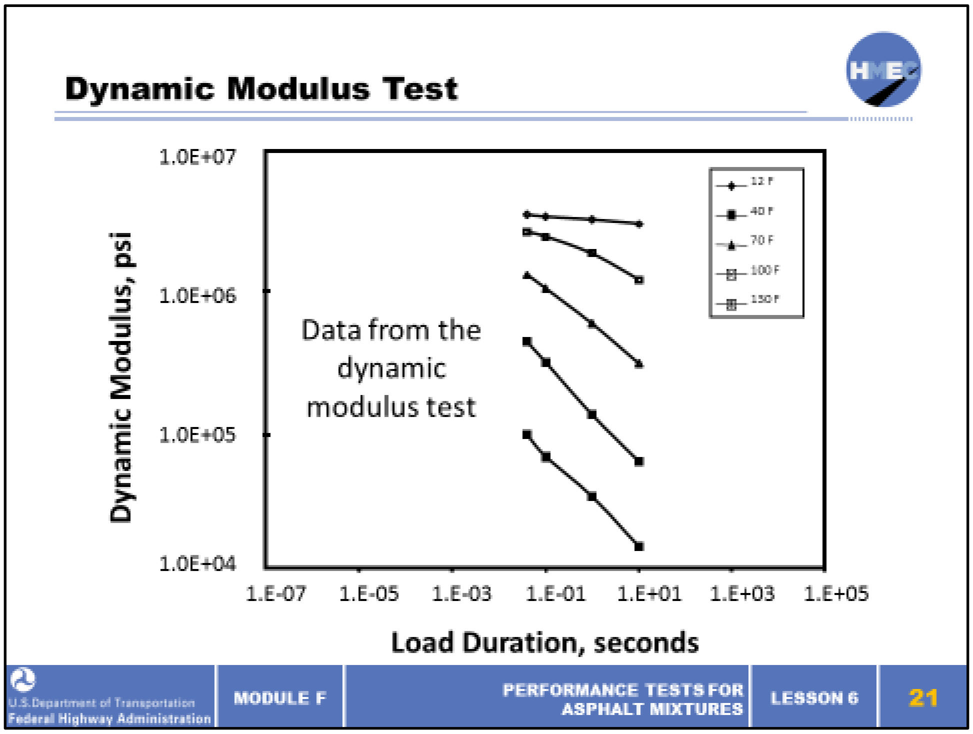
Slide 20



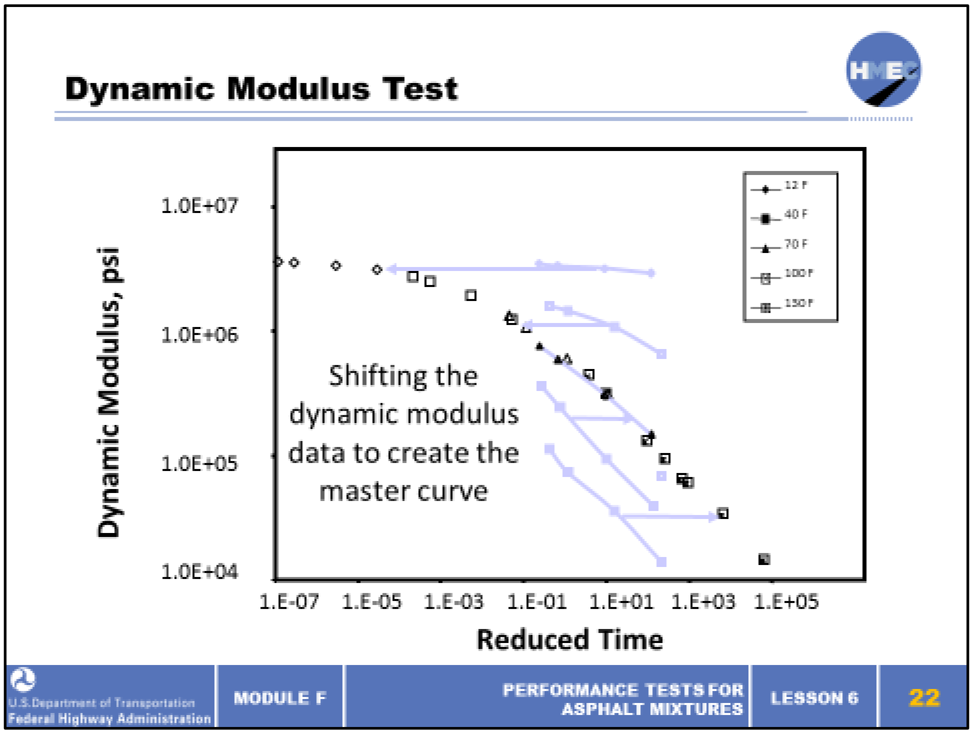
The dynamic modulus test results show that little to no difference in modulus/stiffness is witnessed at the low test temperature. This is expected as all three asphalt binders have a low temperature PG of -22 °C. However, at the intermediate, and especially at the high test temperature, there is a clear difference between the three binder grades.

Slide 21



The test temperatures and loading frequencies included in this illustration are the ones required as minimum inputs to the MEPDG. These dynamic modulus values, however, are entered in the MEPDG software in a tabular format, and do not require any further analysis except for reviewing and checking the reasonableness of the data.

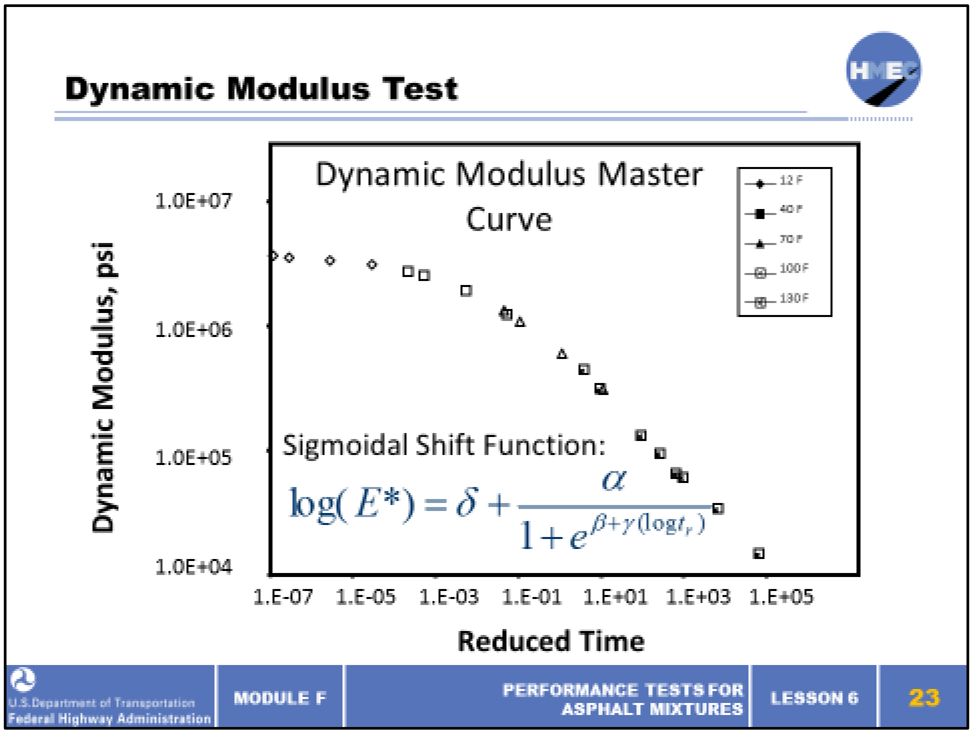
Slide 22



The master curves should be developed in accordance with AASHTO R 62. However, most, if not all, data acquisition systems that come with the AMPT have a built in procedure for shifting the dynamic modulus data for creating or developing the master curve. These data acquisition systems report the actual dynamic modulus values as well as the coefficients of the master curves as outcomes from the test.

As noted in the previous slide, the coefficients of the master curve are not entered into the MEPDG software or other pavement response calculation programs based on elastic layered theory, only a tabular listing of the dynamic modulus values are entered.

Slide 23



The master curve is calculated internally within the MEPDG software and used to determine the dynamic modulus values as a function of pavement depth, traffic speed, and mix or layer temperature.

For comparing the dynamic modulus results from different mixtures, most users typically use the actual dynamic modulus values themselves, rather than comparing the coefficients from the master curve. The coefficients generated from the master curve exhibit less difference between significantly different mixtures than the actual dynamic modulus values because the coefficients are determined over a wide range of temperatures and loading frequencies.

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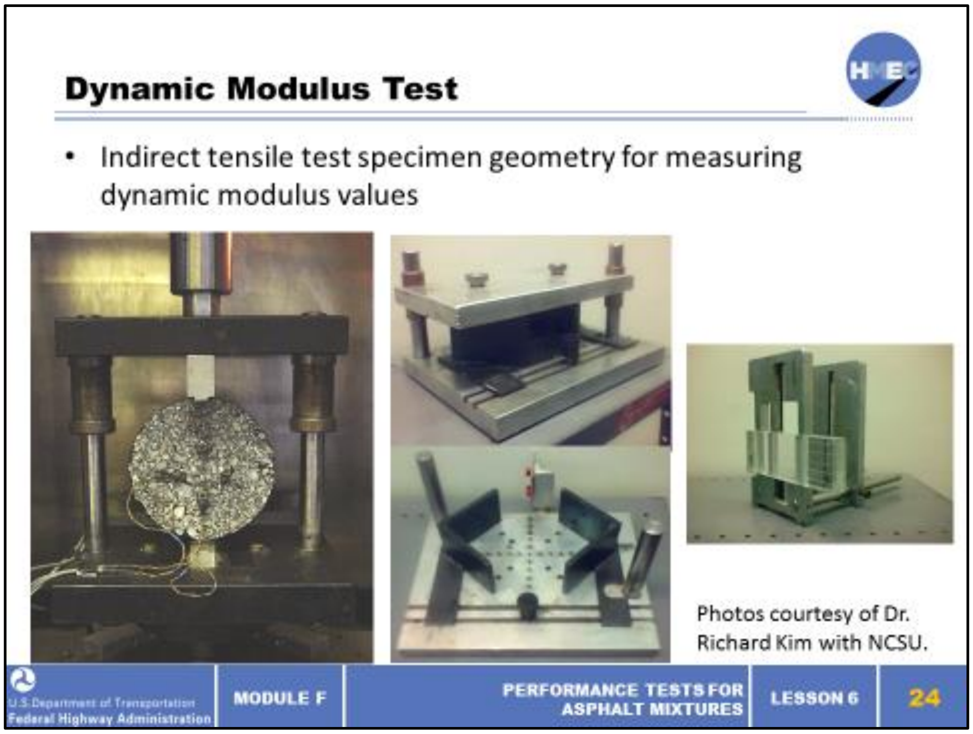
Slide 24



The IDT specimen geometry is being looked into for measuring dynamic modulus because the IDT is applicable in testing field cores. North Carolina State University (NCSU) under the leadership of Dr. Richard Kim. Specifically, roughness tests have been completed and reviewed by the FHWA Expert Task Group for using the IDT test specimen geometry in measuring dynamic modulus.

Slide 25

Tests for Evaluating Performance

Resilient Modulus Test	
Test Designation	ASTM D7369
Purpose of Test & Its Use	Measure of stiffness; provides inputs to structural response models
Performance Indicator	Rutting and fatigue cracking
Test Conditions	Stress controlled; uniaxial or confined; range of temperatures
Specimen Geometry	IDT or cylindrical with height to diameter ratio requirement
Type of Specimen	Core or laboratory gyratory compacted (short-term aged)
Measurement	Axial load and displacements (vertical and/or horizontal) with time & load cycles; recoverable displacements
Outcome of Test	Resilient modulus as a function of temperature

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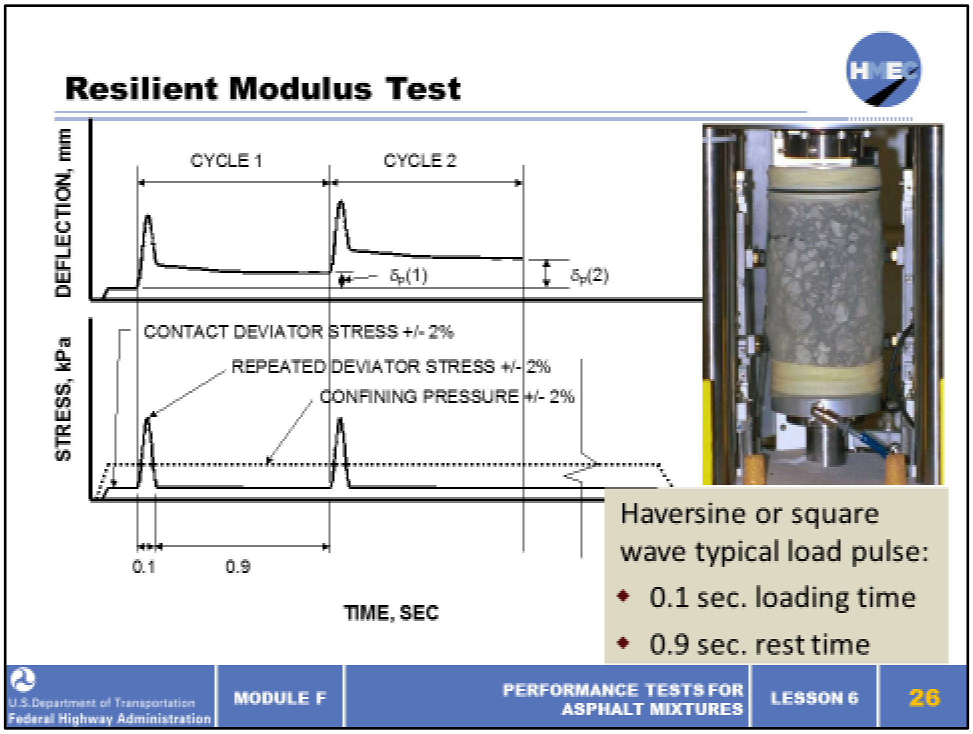
A less common test procedure to determine the mixture stiffness of asphalt concrete mixtures is the resilient modulus test, specified in ASTM D7369.

• The resilient modulus test consists of evaluating the mixture stiffness at various test temperatures.
• The testing is conducted within the materials linear elastic range so that minimal to no permanent strain occurs.

Similar to the dynamic modulus test, the resilient modulus values can be measured on uniaxial or confined compression type specimens geometries, as well as indirect tensile specimen geometry.

Slide 26



The resilient modulus test is a stress-controlled test, which was used within the Long-Term Pavement Performance (LTPP) program for measuring the modulus of the HMA mixture for the special project studies (SPS) experiments. Historically, the resilient modulus test has been used extensively to characterize the stiffness property of asphalt concrete mixtures. More recently, however, it is being used much less because the dynamic modulus test is much easier to perform with the development and adoption of the AMPT.

The deviator or uniaxial stress is applied using a sinusoidal waveform, where both the applied stress and resultant vertical deformations are measured. The peak applied stress and recovered or elastic deformations are used to calculate the resilient modulus. The type of test specimen geometries applicable to this test method include: uniaixial or unconfined and triaxial or confined compression specimens, and indirect tensile specimens. The resilient modulus test is usually performed over a range of temperatures similar to the dynamic modulus test.

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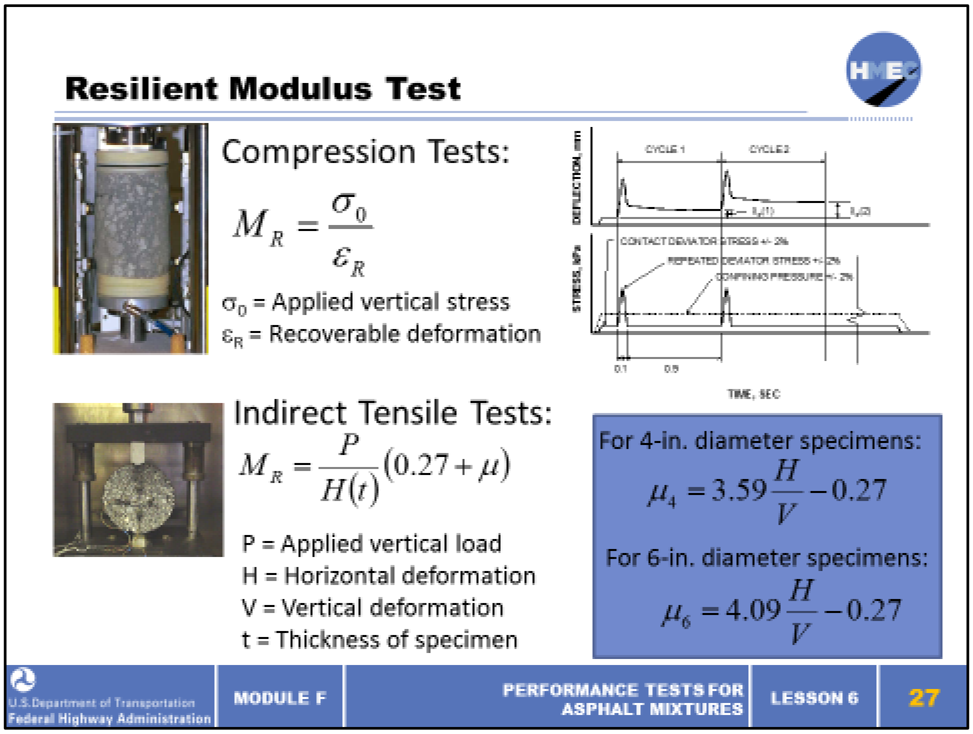
Slide 27



The equations for calculating resilient modulus for the indirect tensile specimens are dependent on the diameter of the test specimen. Calculating Poisson's ratio from the horizontal and vertical measured displacements can result in values significantly greater than 0.5, especially for higher test temperatures. When the values start to exceed 0.5, this implies the assumptions of elastic layer theory that were used to develop these equations to compute resilient modulus no longer apply. At this point, the test specimen is starting to behave or respond like a compression-type specimen and/or cracks may be starting to occur around where the loading platens contact the specimen. It is advisable to always compute Poisson's ratio so that condition can be identified from the test results.

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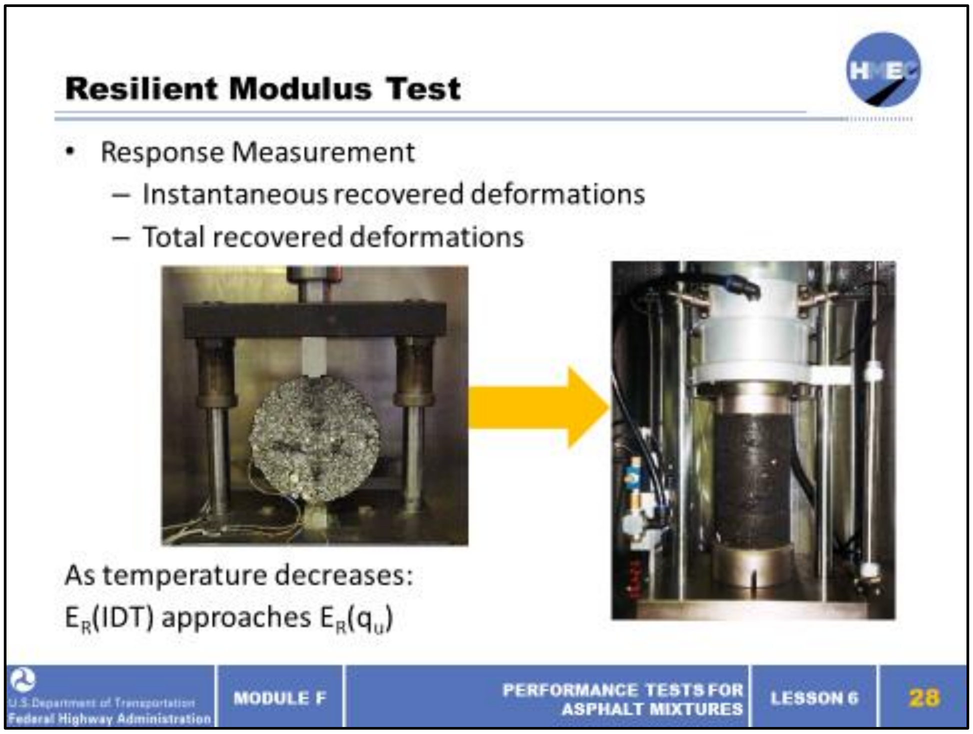
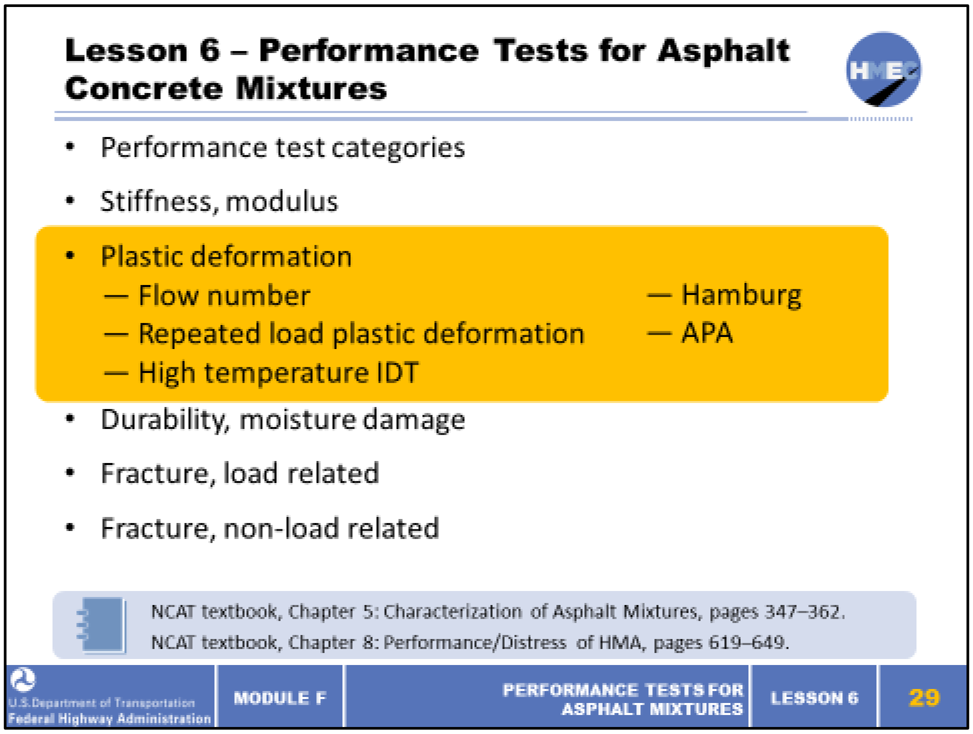
Slide 28



The total resilient modulus has been used in most pavement performance studies. Both resilient modulus values are reported in the LTPP database for those mixtures tested in the laboratory.

Slide 29



Plastic deformation by itself is not a mechanistic property of the mixture. However, the plastic deformation coefficients of parameters of the rut depth prediction model is considered a mechanistic properties because the parameters are used to predict rut depths over time in the MEPDG and other programs. The tests that provide the plastic deformation characteristics or parameters of rut depth prediction models include: repeated load plastic deformation test and the repeated shear constant height test. The tests that provide an index on how susceptible the asphalt concrete mixture is to rutting or distortion include flow number, asphalt pavement analyzer wheel tracking device, Hamburg wheel tracking device, and the high temperature indirect tensile test.

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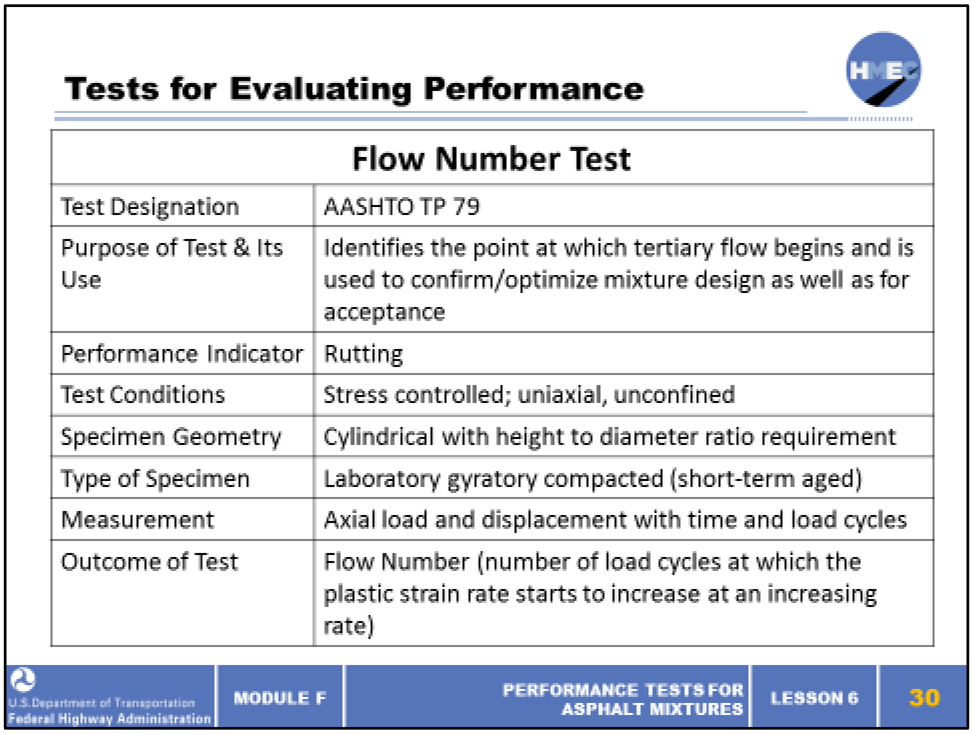
Slide 30

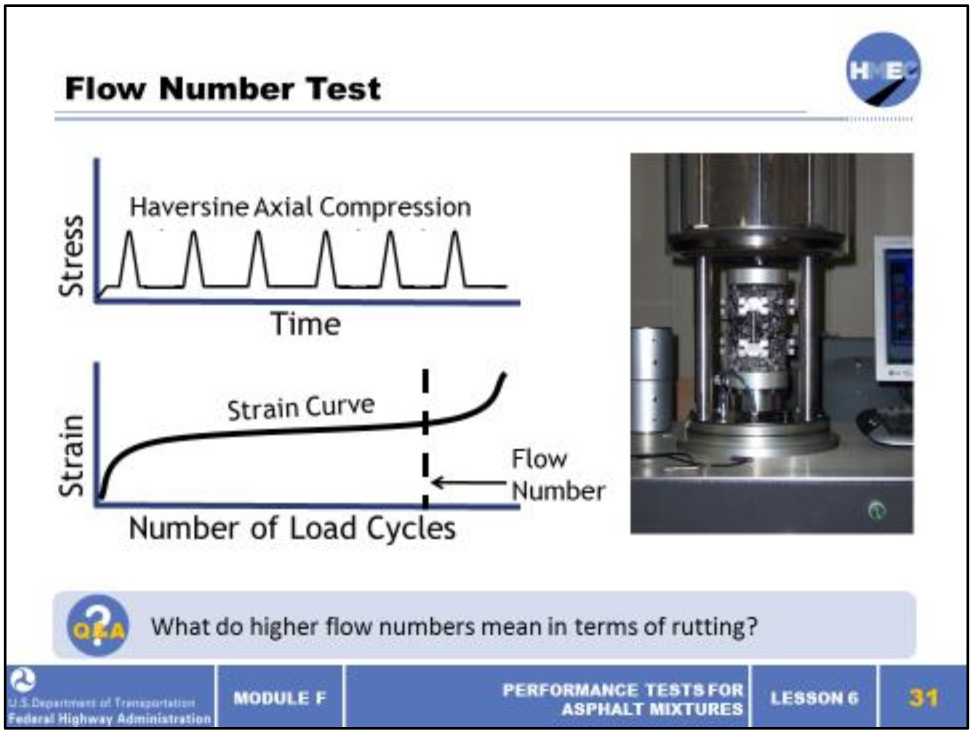


Tests for Evaluating Performance

Flow Number Test	
Test Designation	AASHTO TP 79
Purpose of Test & Its Use	Identifies the point at which tertiary flow begins and is used to confirm/optimize mixture design as well as for acceptance
Performance Indicator	Rutting
Test Conditions	Stress controlled; uniaxial, unconfined
Specimen Geometry	Cylindrical with height to diameter ratio requirement
Type of Specimen	Laboratory gyratory compacted (short-term aged)
Measurement	Axial load and displacement with time and load cycles
Outcome of Test	Flow Number (number of load cycles at which the plastic strain rate starts to increase at an increasing rate)

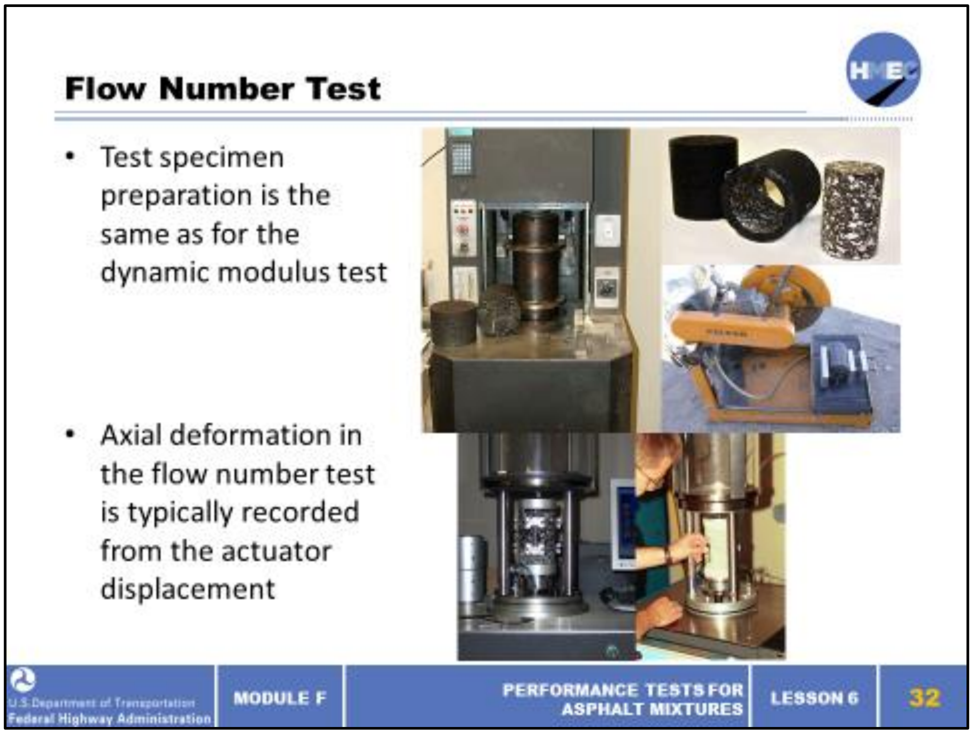
Flow number was developed under NCHRP project 9-19 and 9-29. In addition, the AMPT can be used to perform the flow number test. The test defines the number of load cycles at which tertiary flow begins to occur. The test is a repeated load test can be used as part of the mixture design process to select the upper limit of asphalt content to maximize durability without scarifying the mixture's resistance to distortion and prevent unacceptable levels of rutting.

Slide 31



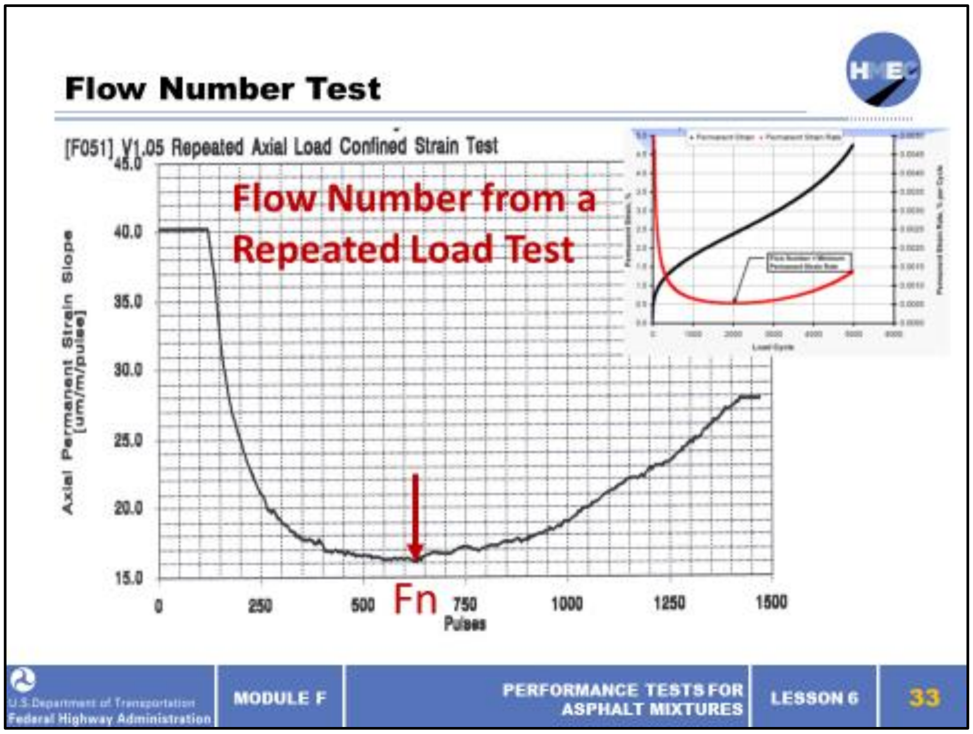
The flow number is defined as the point where the permanent strain increases uncontrolled. This is referred as tertiary flow, as stated in the previous slide. Similar to the dynamic modulus testing, the flow number test is conducted in accordance with the AASHTO standard TP 79 and the AMPT.

Slide 32



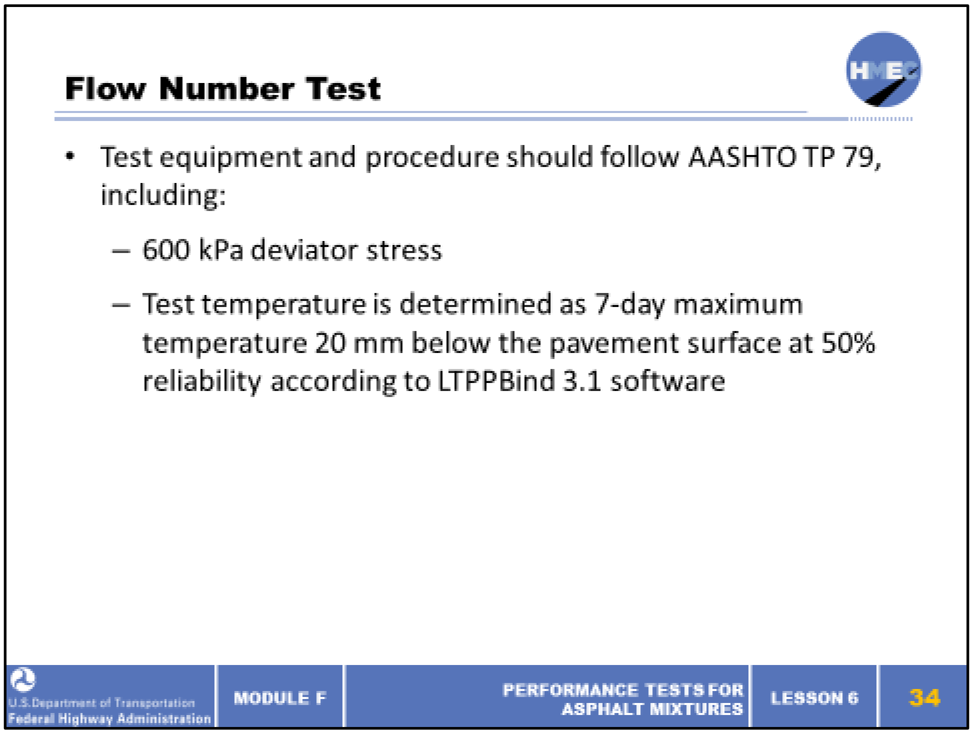
The flow number test specimens are prepared the same as for the dynamic modulus test specimens, which have been already discussed. In summary, the test specimens are produced by compacting tall gyratory specimens (at least 170 mm tall and 150 mm diameter) and then coring out the inner 100 mm diameter core. The ends of the cored specimen are then trimmed to a final height of 150 mm. It is more important for the flow number specimens than for the dynamic modulus specimens that both ends of the specimen be parallel and perpendicular to the vertical axis of the specimen. If the ends are not parallel to each other, it can cause the specimen to "walk" during the repeated load test. The test results from this condition will be inaccurate flow number values.

Slide 33



For very stiff mixtures that are highly resistant to plastic deformation, no flow number will be measured.

Slide 34



Recent work conducted under NCHRP 9-33 includes the flow number test as a performance verification during HMA mixture design. Preliminary requirements are provided for mixtures designed for certain traffic levels. As traffic levels increase (higher ESALs), the minimum flow number increases requiring a mixture with greater resistance to rutting. The research for this test is included and described in NCHRP report #513.

Slide 35

Flow Number Test

- Flow Number Criteria:

Traffic Level Million ESALS	WMA	HMA
< 3	NA	NA
3 to < 10	30	50
10 to < 30	105	190
≥ 30	415	740

Q&A Why are the criteria so different between WMA and HMA?

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Flow number criteria that have been developed are mixture dependent. The criteria used for HMA and WMA are different and will likely change over time. The criteria may also be found to be dependent on the type of additives to be used for WMA.

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Tests for Evaluating Performance

Repeated Load Plastic Deformation Test	
Test Designation	NCHRP Report 719 (Project 9-30A)
Purpose of Test & Its Use	Measure of resistance to rutting/distortion; inputs for the plastic strain coefficients of the rut depth transfer function in the MEPDG
Performance Indicator	Rutting
Test Conditions	Stress controlled; 10 psi confinement; one to three test temperatures
Specimen Geometry	Cylindrical with height to diameter ratio requirement
Type of Specimen	Laboratory gyratory compacted (short-term aged)
Measurement	Axial load and plastic displacements with time and load cycles
Outcome of Test	Plastic strain coefficients and total plastic strain

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The repeated load plastic deformation test is not a new test. This test was used to provide the input values to the older VESYS program sponsored and developed by the FHWA in the early 1980s. It was also used to evaluate a mixture's resistance to rutting and distortion in the Asphalt-Aggregate Mixture Analysis System (AAMAS) that laid the foundation for the SHRP Asphalt Research Program. The AMPT can be used to perform repeated load plastic deformation test. The difference between this test and the flow number is how the data are analyzed and confinement. The test is a repeated load test can be used as part of the mixture design process to select the upper limit of asphalt content to maximize durability without scarifying the mixture's resistance to distortion and prevent unacceptable levels of rutting.

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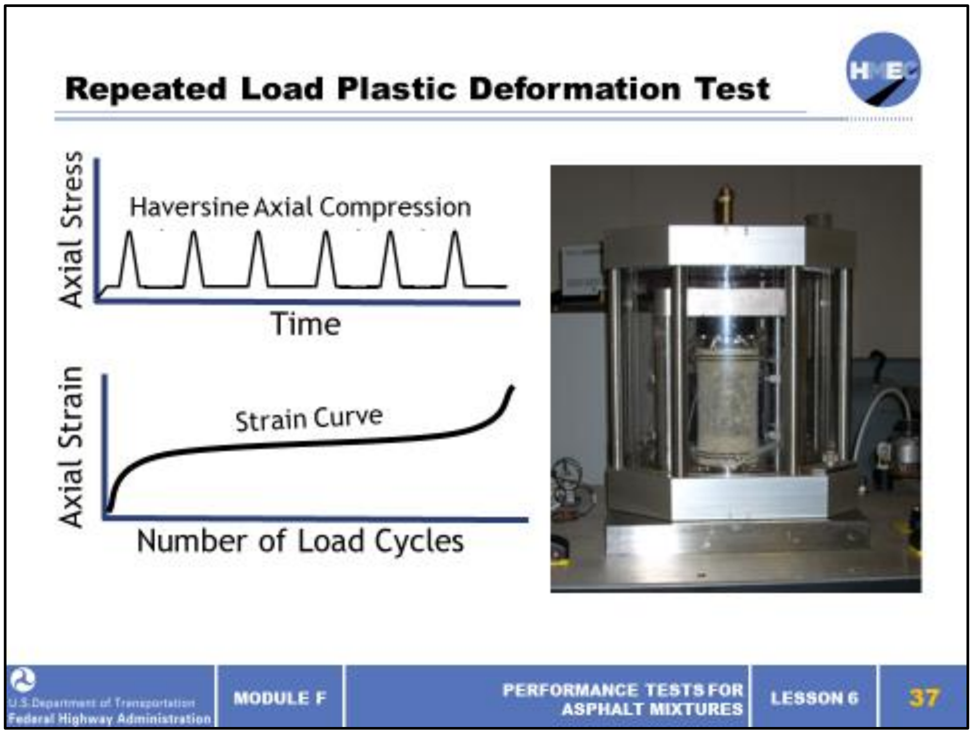
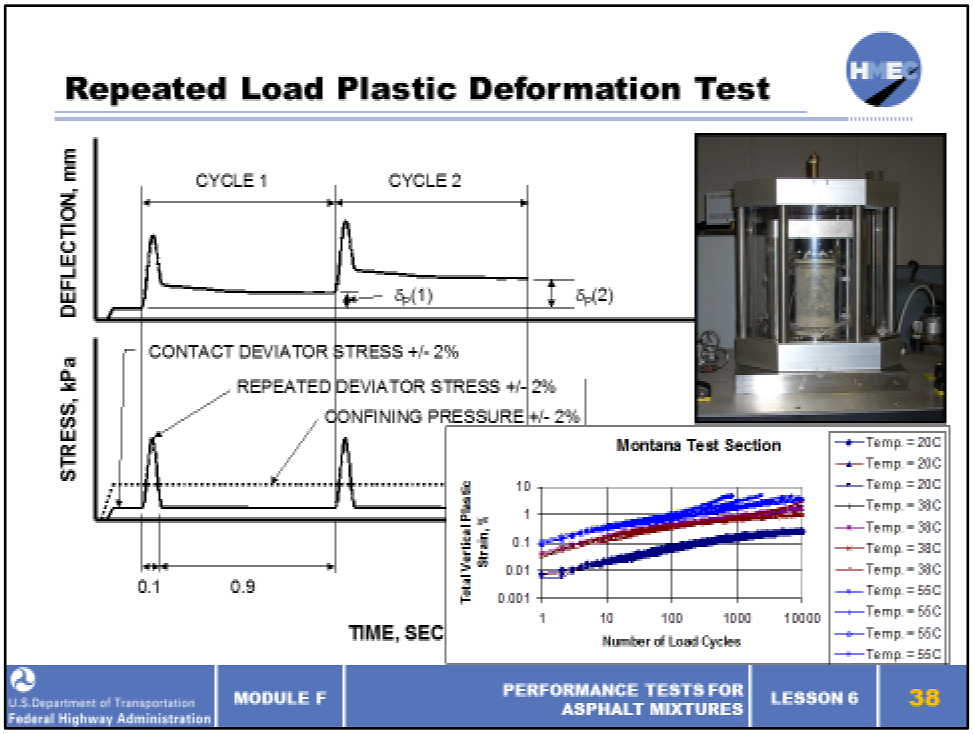
Slide 37



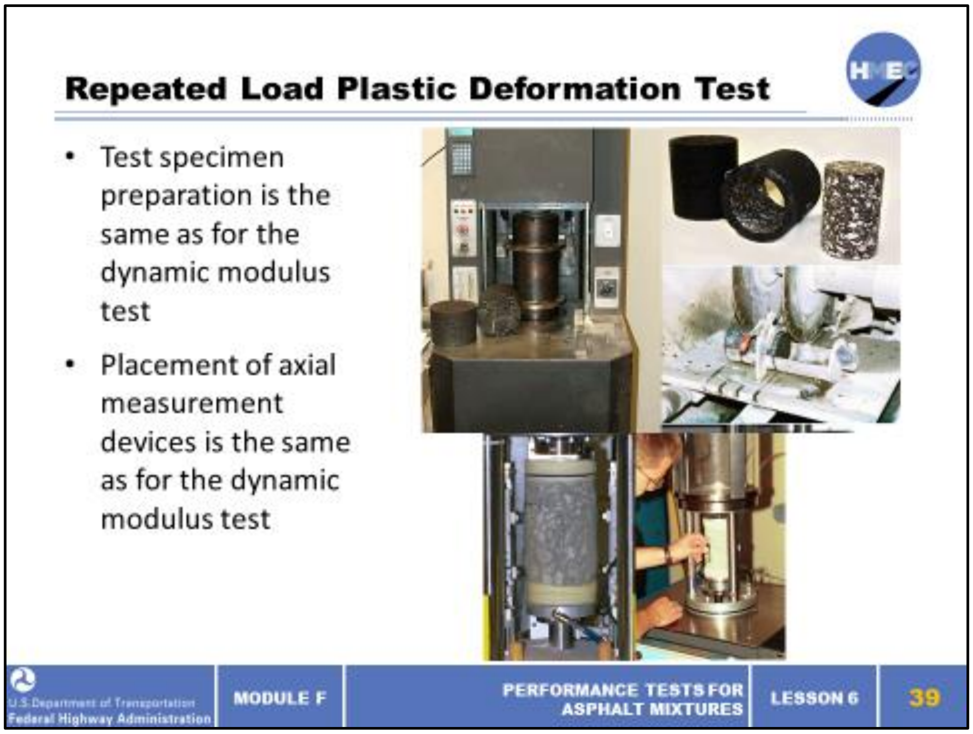
The repeated load plastic deformation test provides the accumulated total and plastic strain. Unlike the flow number test, the plastic deformation parameters can be determined even if the specimen does not exhibit tertiary flow, as shown in this slide. Similar to the dynamic modulus and flow number tests, the repeated load plastic deformation test can be performed using the AMPT.

Slide 38



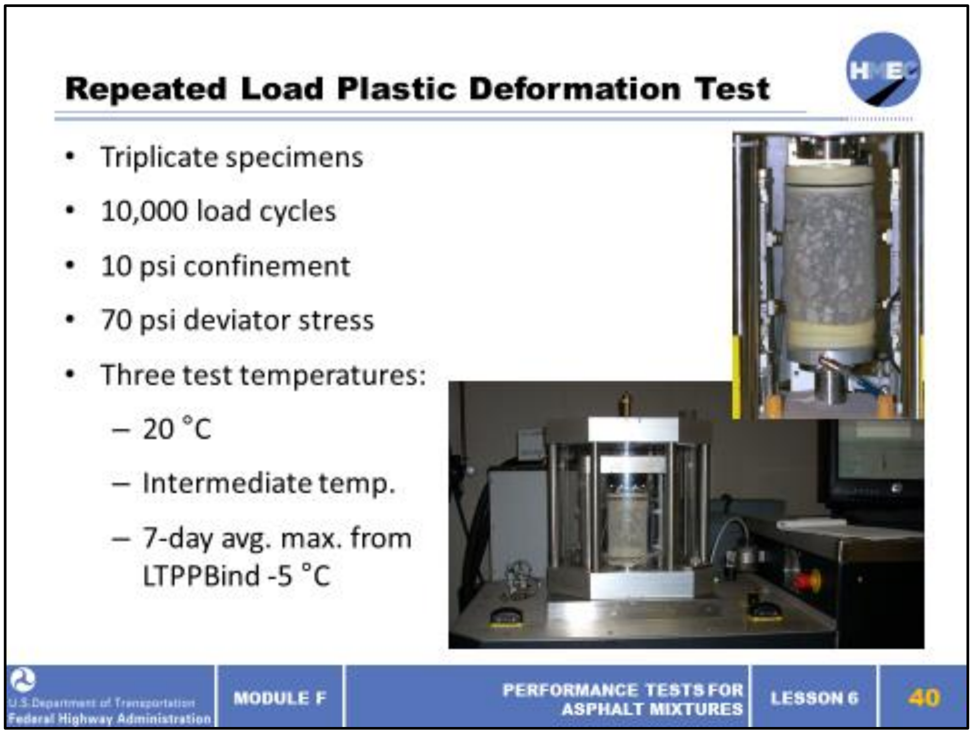
The plastic vertical or axial strains and total strains are measured and accumulated with the number of load cycles. The result is a relationship between number of load cycles and accumulated plastic strain. The outcome from the test is defined as the slope and intercept of this relationship within the steady state or secondary range of accumulated plastic strain versus number of load cycles, as shown by the illustration in this slide.

Slide 39



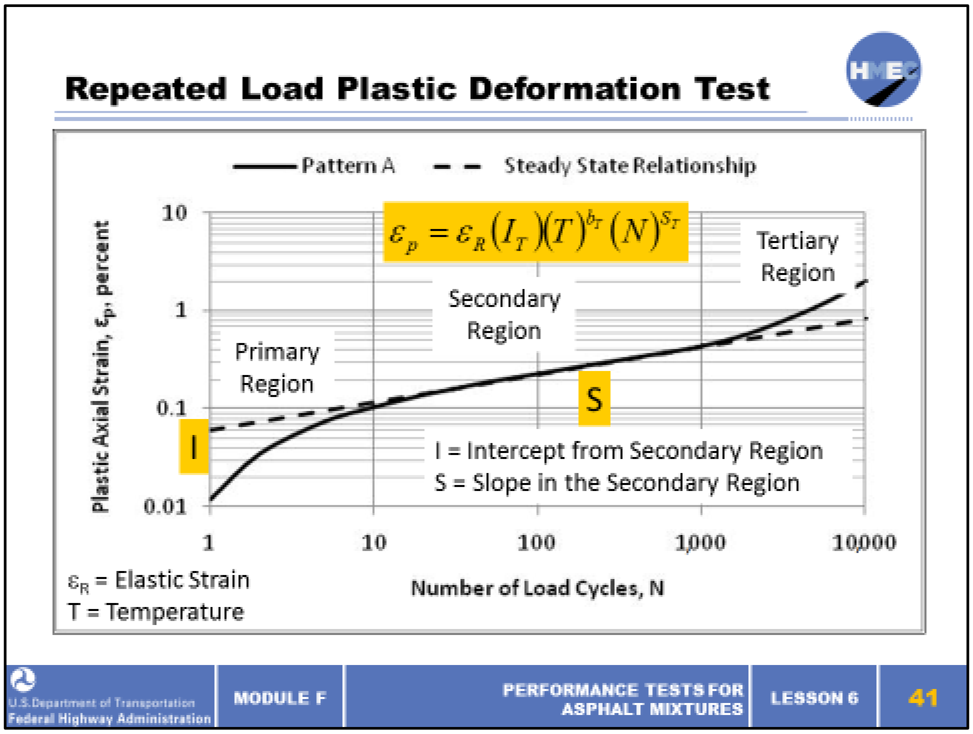
The repeated load plastic deformation test specimens are prepared the same as for the flow number and dynamic modulus test specimens, which have been already discussed. In summary, the test specimens are produced by compacting tall gyratory specimens (at least 170 mm tall and 150 mm diameter) and then coring out the inner 100 mm diameter core. The ends of the cored specimen are then trimmed to a final height of 150 mm. Similar to the flow number, it is more important for these test specimens than for the dynamic modulus specimens that both ends of the specimen be parallel and perpendicular to the vertical axis of the specimen. If the ends are not parallel to each other can cause the specimen to “walk” during the repeated load test. The test results from this condition will be inaccurate.

Slide 40



The test can exhibit high variability, so triplicate specimens and three test temperatures are recommended for use as noted above. The test procedure presented in NCHRP Report 719 allows one test temperature to be used and that temperature is the equivalent rut depth temperature. One temperature would be considered sufficient when used on a mixture design basis, while the three test temperatures are required for establishing the inputs to the MEPDG and other rut depth prediction models and transfer functions.

Slide 41



The plastic deformation or axial strain relationship (illustrated as the log of number of load cycles versus log of plastic axial strain) measured from repeated load plastic deformation tests is divided or grouped into three basic areas: the primary region, the secondary or steady state regions, and the tertiary region. The slope of the relationship is first determined in the steady state region. After the slope is determined, that value is then used to estimate the intercept from the steady state region as shown in the illustration. Both values can be used an inputs to the MEPDG and other rut depth prediction models. The rut depth transfer equation included in the MEPDG is shown in the slide along with the two parameters derived from the laboratory test.

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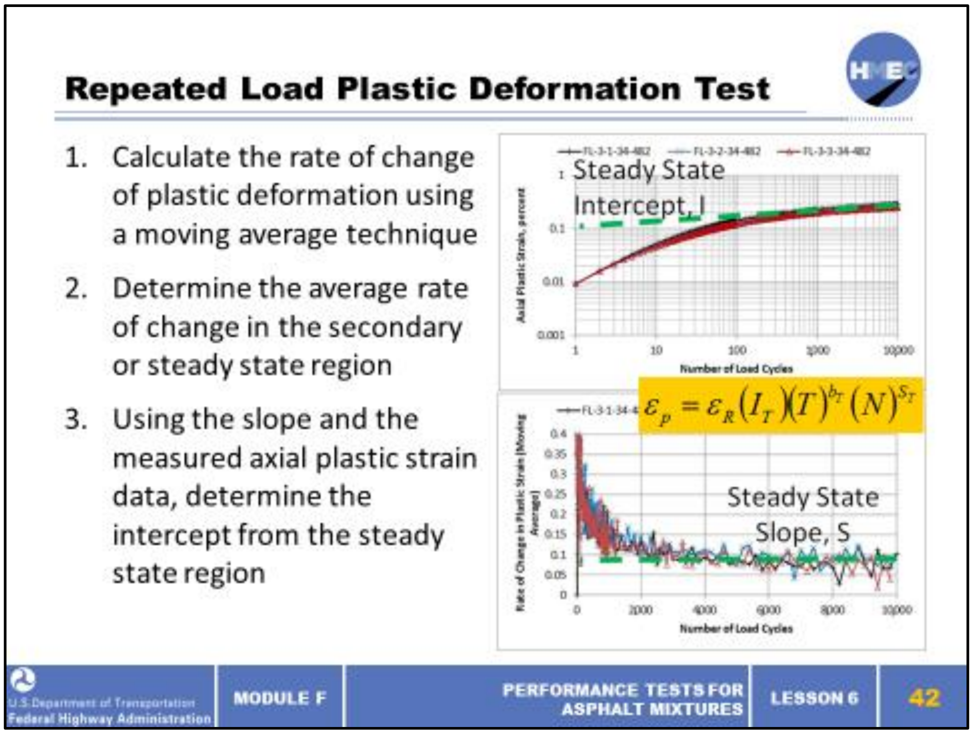
Slide 42



The data acquisition system is used to determine two parameters from the plastic deformation relationship. The slope of the relationship is first determined in the steady state or secondary region. This is done by calculating the average rate of change for the plastic axial strain using a moving average technique for the number of loading cycles. After the slope is determined, that value is then used to estimate the intercept from the steady state region, as shown in the illustration. Both values can be used an inputs to the MEPDG and other rut depth prediction models. These two values are then used within the rut depth transfer function for predicting rut depths over time.

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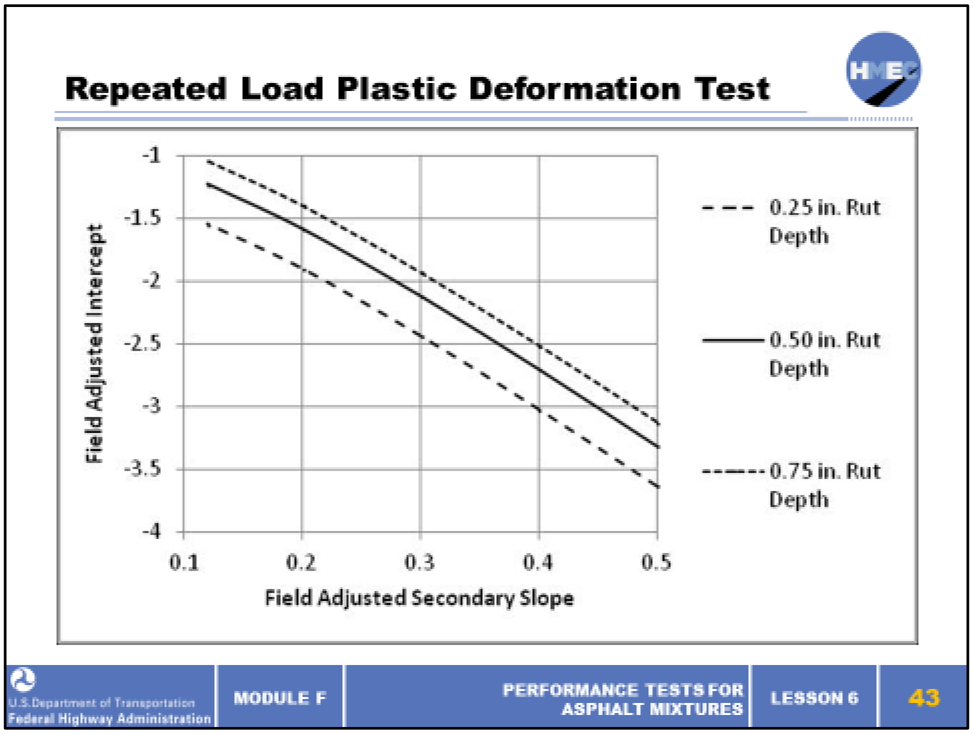
Slide 43



The two plastic deformation parameters are related to one another for a specific mix and the two parameters together can be used to determine whether that mix is susceptible to rutting. For this graph, the MEPDG was used to determine the relationship between the slope and intercept that will result in the same amount of rutting for a specific period of time and traffic level. Any combination of slope and intercept that fall below the rut depth selected as the critical value should have less than that amount of rutting. Conversely, if the two points fall above the designated line, they will exhibit more rutting than designated for that line.

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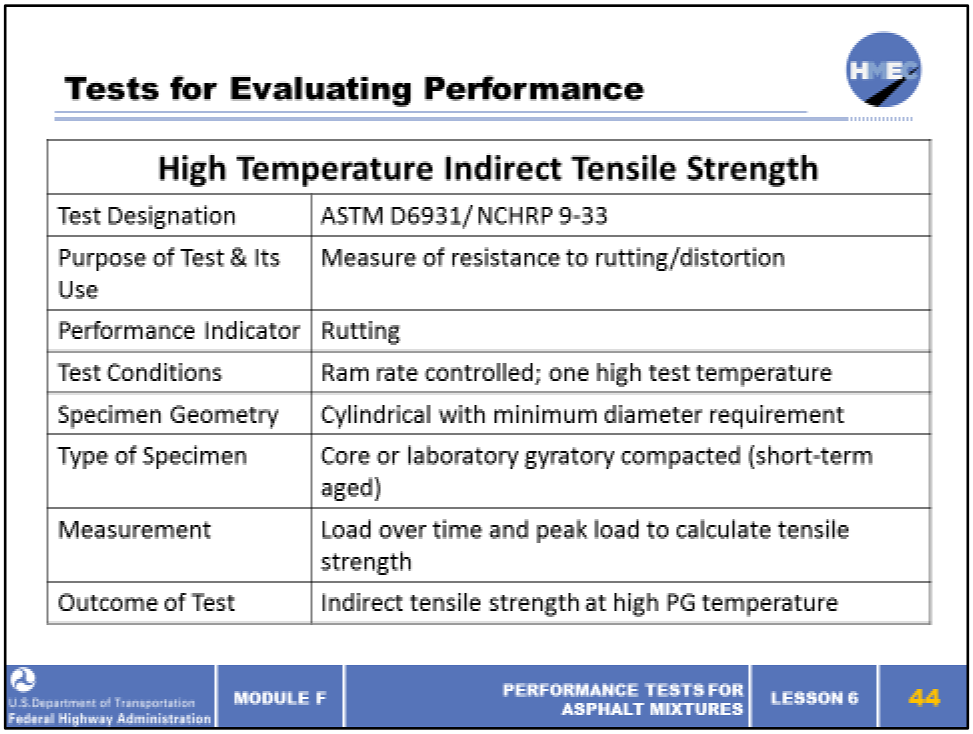
Slide 44



Tests for Evaluating Performance

High Temperature Indirect Tensile Strength	
Test Designation	ASTM D6931/ NCHRP 9-33
Purpose of Test & Its Use	Measure of resistance to rutting/distortion
Performance Indicator	Rutting
Test Conditions	Ram rate controlled; one high test temperature
Specimen Geometry	Cylindrical with minimum diameter requirement
Type of Specimen	Core or laboratory gyratory compacted (short-term aged)
Measurement	Load over time and peak load to calculate tensile strength
Outcome of Test	Indirect tensile strength at high PG temperature

The high temperature indirect tensile strength test was recommended under NCHRP Project 9-33 for use as a simple test to evaluate rutting susceptibility. It can be used to evaluate a mixture's resistance to rutting and distortion.

Slide 45

High Temperature Indirect Tensile Strength

- Test equipment and procedure follow AASHTO T 283, except:
  - Specimens are tested dry (unconditioned)
  - Test temperature is the 7-day maximum temperature 20 mm below the pavement surface at 50% reliability according to LTPPBind 3.1 software minus 10 °C

Traffic Level *(Million ESALs)*	Minimum IDT Strength *(kPa)*
< 3	---
3 to < 10	190
10 to < 30	270
> 30	420

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NCHRP Project 9-33 recommended the use of a mix design guide for HMA mixtures which used this IDT strength value at high temperature. The Researchers of NCHRP project 9-33 have included an array of performance tests to validate the mixture design based on volumetric properties, as discussed within Lesson 5. There has been extensive debate within industry on the use and validity of the IDT specimen geometry at high temperatures.

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Tests for Evaluating Performance

Asphalt Pavement Analyzer, APA	
Test Designation	AASHTO T 340
Purpose of Test & Its Use	Measure of resistance to rutting
Performance Indicator	Rutting
Test Conditions	Load/pressure controlled; equivalent temperature
Specimen Geometry	Cylindrical or beam specimen
Type of Specimen	Sawed or laboratory compacted (short-term aged)
Measurement	Average plastic deformation with load cycles
Outcome of Test	Number of load cycles to a maximum rut depth

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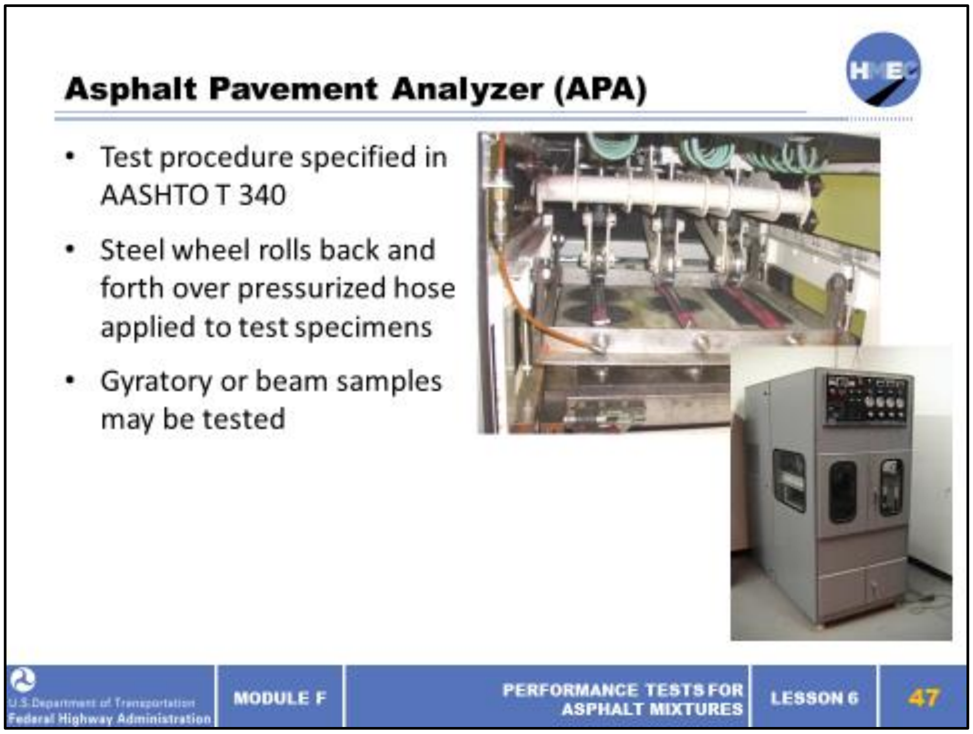
The APA is an empirical test but is used by multiple agencies to confirm mixture designs based on volumetric properties. Illustrations of this test, as well as test specimen geometries, follow in the next couple of slides.

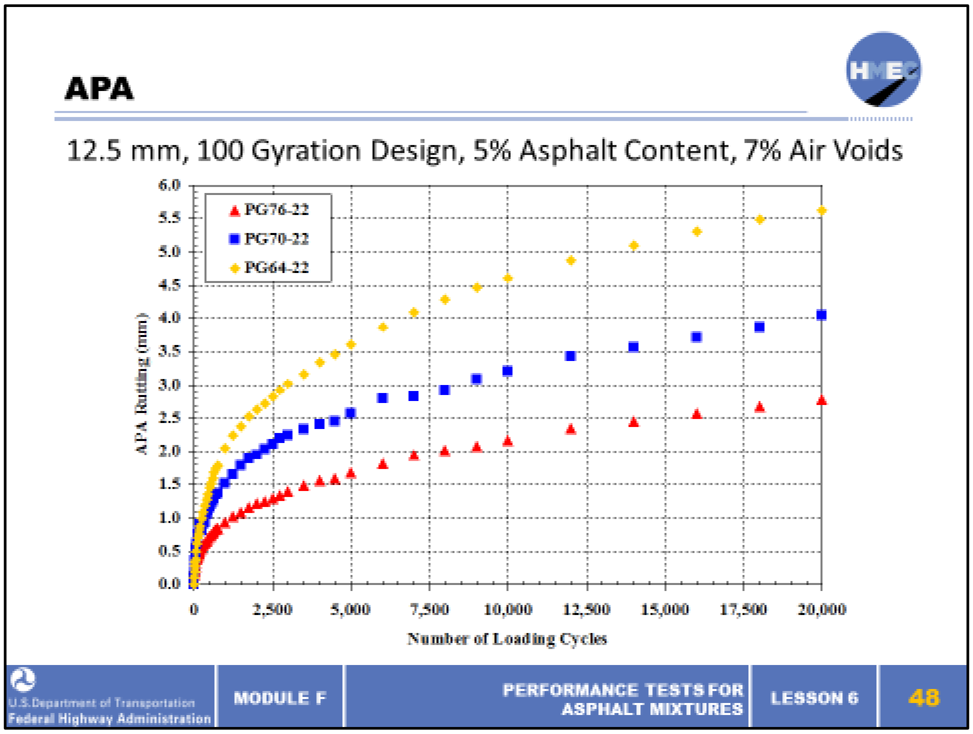
Slide 47



NCHRP Project 9-17 focused on evaluating the Asphalt Pavement Analyzer test and developing pass/fail criteria, which is being used by many agencies. Those criteria are provided in the end of this section in terms of test applications. The APA criteria is similar to that developed for the flow number.

In summary, the test temperature is determined as 7-day maximum temperature at 98% reliability according to LTPPBind 3.1 software, and the deformation from the number of load is measured after 8,000 loading and 50 seating cycles.

Slide 48

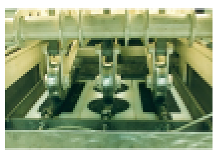


Some agencies and researchers have attempted to take the APA test results and correlate the slope and intercept from the repeated load plastic deformation test but without success. The graph included in this slide shows the results from the testing of a bituminous mixture with different asphalt grades. The harder the asphalt, the lower the slope and the smaller the rut depth. In summary, the APA results are being used on a pass-fail basis and do not correlate well with the time-series rut depth.

Slide 49

APA

- Test temperature is determined as 7-day maximum temperature at 98% reliability according to LTPPBind 3.1 software
- APA rutting measured after 50 seating cycles and 8,000 loading cycles



Application of Test Outcomes

Traffic Level *(Million ESALs)*	Maximum APA Rut Depth *(mm)*
< 3	---
3 to < 10	5
10 to < 30	4
> 30	3

Proposed criteria from NCHRP Project 9-33

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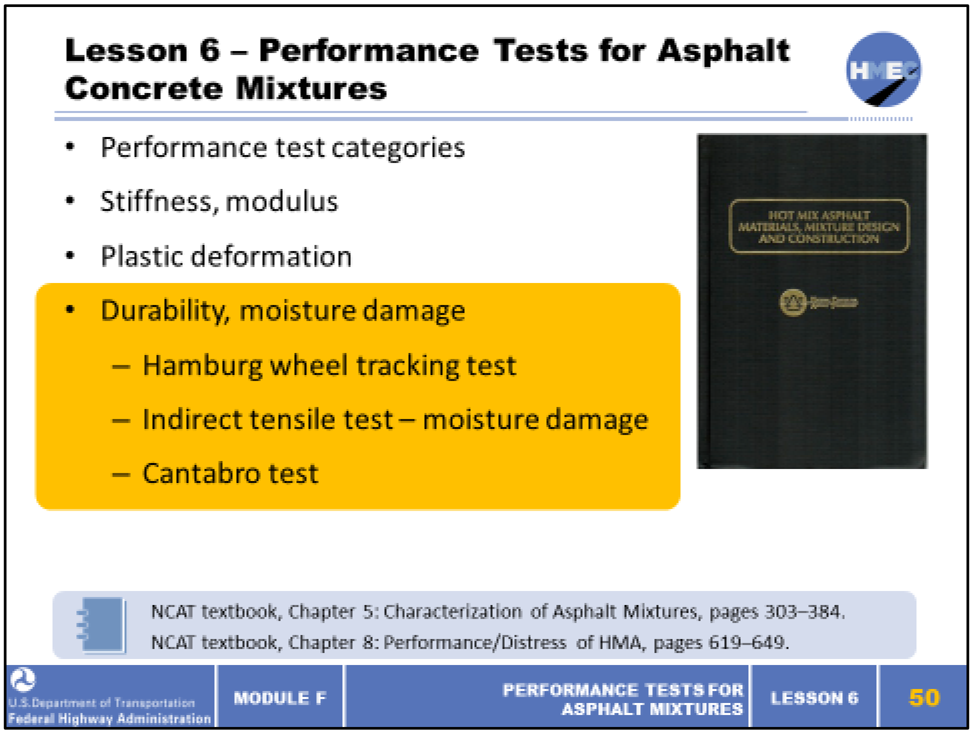
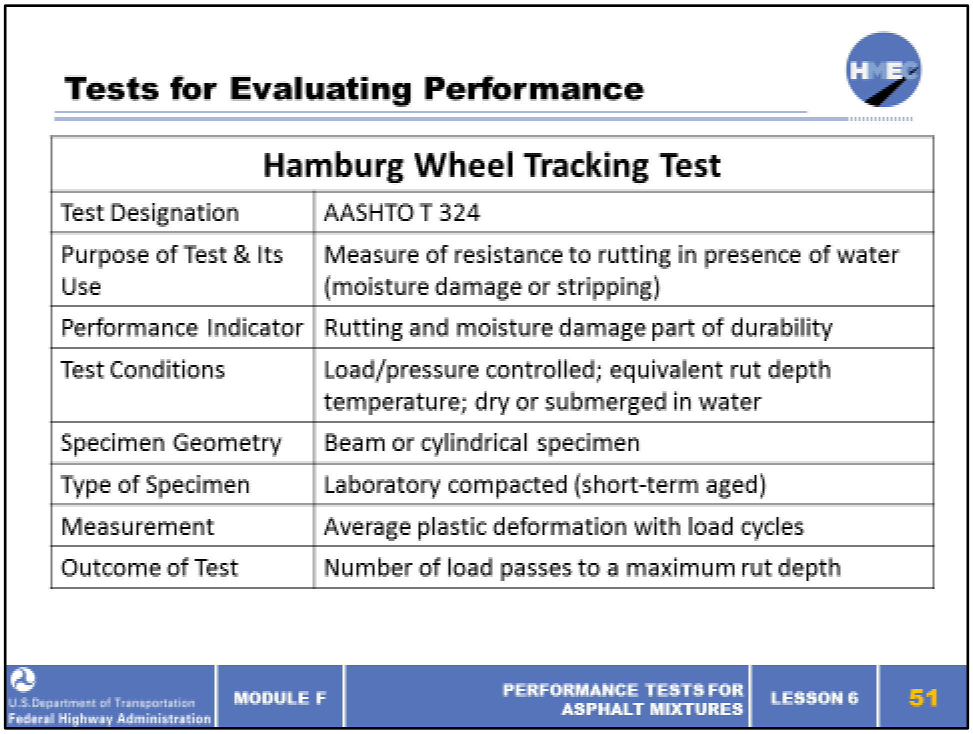
Many agencies have found and concluded that they needed to make some revisions to these criteria in applying the APA for mixture confirmation for their local conditions and materials.

Slide 50



Moisture damage and stripping potential by itself is not a mechanistic property of the mixture but can have a huge influence on the occurrence of pavement distresses such as rutting, cracking, and bleeding, as discussed in Lesson 1.

Slide 51

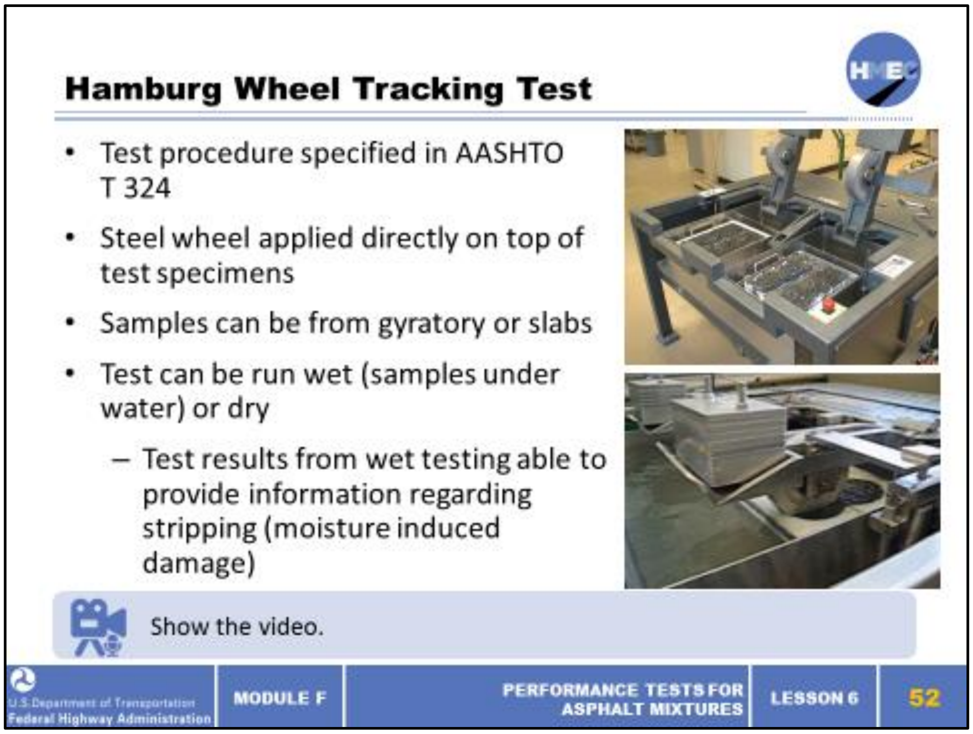


Tests for Evaluating Performance

Hamburg Wheel Tracking Test	
Test Designation	AASHTO T 324
Purpose of Test & Its Use	Measure of resistance to rutting in presence of water (moisture damage or stripping)
Performance Indicator	Rutting and moisture damage part of durability
Test Conditions	Load/pressure controlled; equivalent rut depth temperature; dry or submerged in water
Specimen Geometry	Beam or cylindrical specimen
Type of Specimen	Laboratory compacted (short-term aged)
Measurement	Average plastic deformation with load cycles
Outcome of Test	Number of load passes to a maximum rut depth

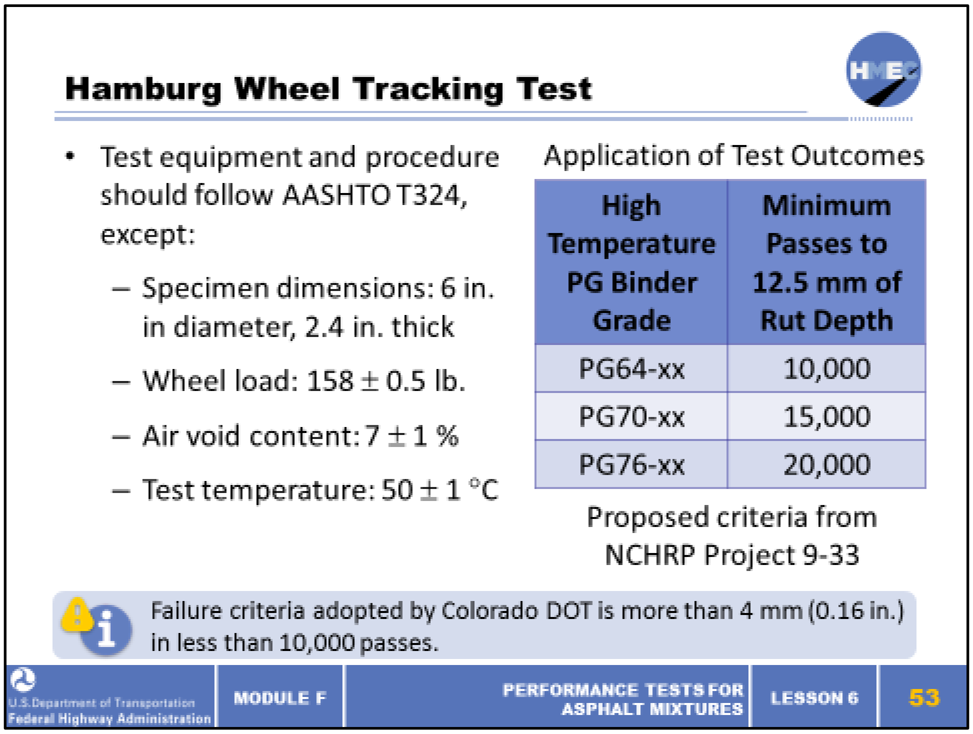
The Hamburg wheel tracking test measures the combined effects of rutting and moisture sensitivity. The asphalt mix specimens are immersed in hot water and a steel wheel rolls back and forth on the surface of the slab.

Slide 52



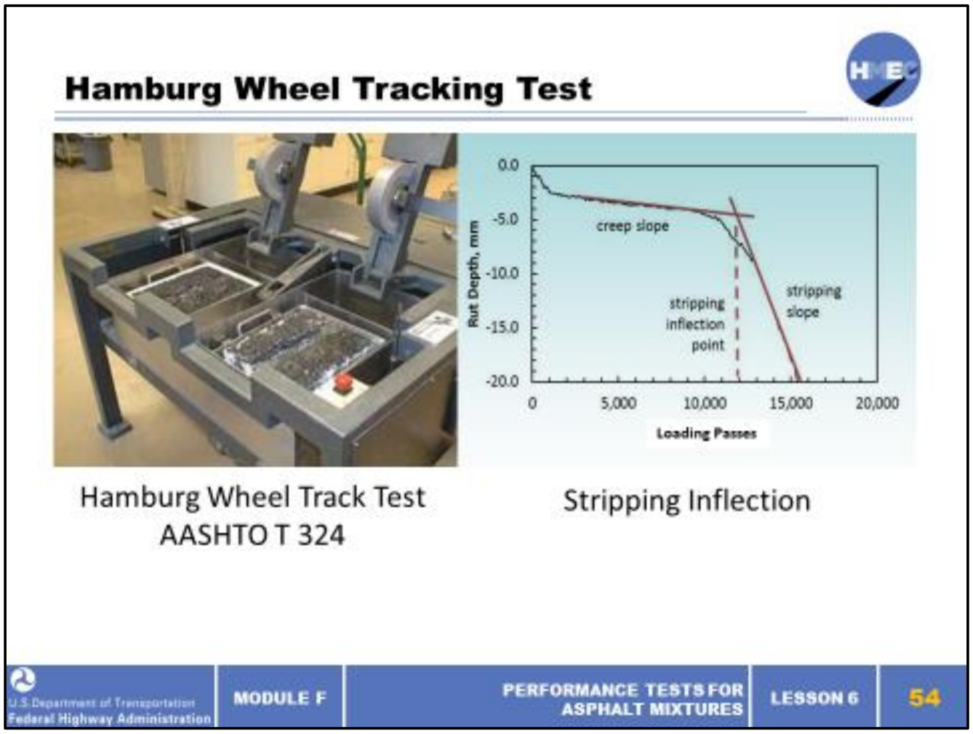
The Hamburg wheel tracking test measures the combined effects of rutting and moisture sensitivity. The asphalt mix specimens are immersed in hot water and a steel wheel rolls back and forth on the surface of the slab.

Slide 53



The Hamburg criteria is similar to that developed for the flow number.

Slide 54



This evaluation normally looks at the stripping inflection point shown in the illustration. If a mixture is susceptible to moisture damage the rutting curve will have two distinct slopes. The stripping inflection point is a measure of how many cycles it takes for stripping to occur in the mixture.

Slide 55

Tests for Evaluating Performance

Moisture Damage – Indirect Tensile Strength	
Test Designation	AASHTO T 283
Purpose of Test & Its Use	Measure of resistance to stripping and moisture damage
Performance Indicator	Moisture damage component of durability
Test Conditions	Controlled ram rate for measuring strength; dry or wet conditioned in water
Specimen Geometry	Cylindrical specimen
Type of Specimen	Laboratory compacted (short-term aged)
Measurement	Average indirect tensile strength for two sets of specimens: one dry and one wet conditioned
Outcome of Test	Ratio of wet to dry indirect tensile strengths

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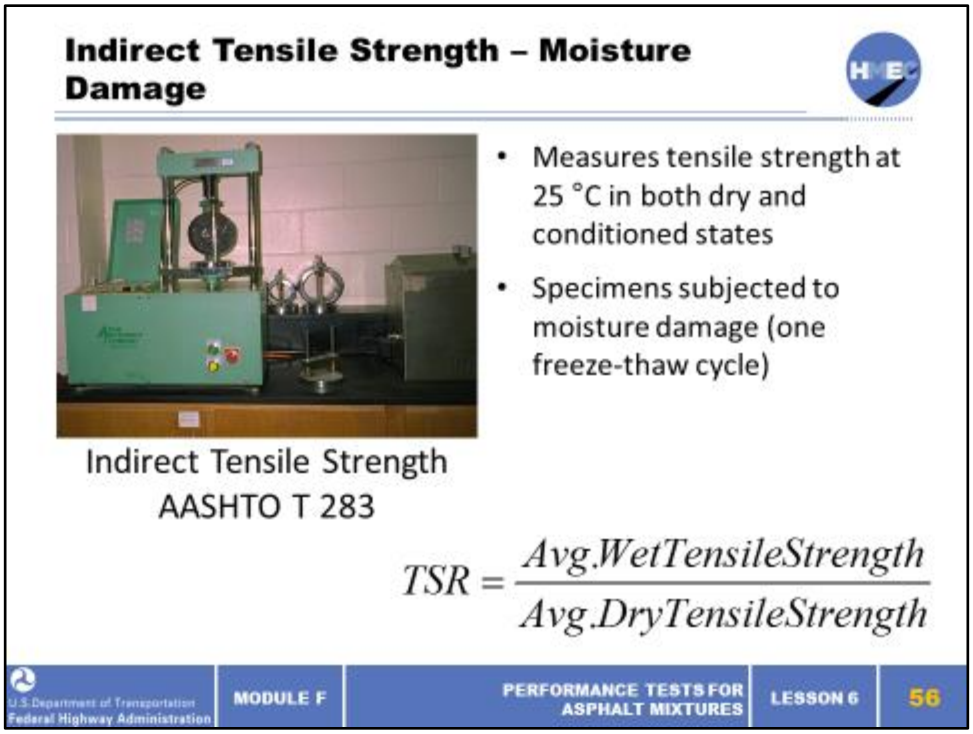
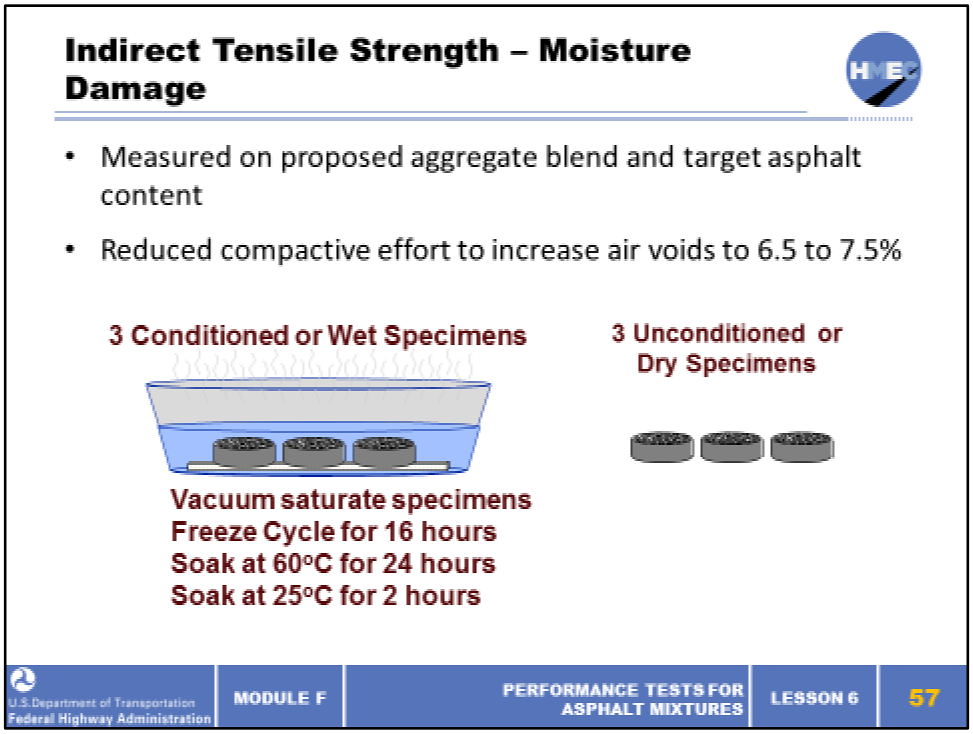
The indirect tensile strength test is used by many agencies for measuring moisture damage susceptibility and determining whether an additive needs to be added to the mixture to reduce the probability for stripping.

Slide 56



AASHTO T 283 involves measurements of indirect tensile strength for two sets of specimen: 1. unconditioned, dry specimens, and 2. specimens conditioned using a freeze-thaw cycle. The ratio between the dry and conditioned tensile strength is called the tensile strength ratio. Some engineers use the dry tensile strength at 25 °C as an indicator of the overall strength of an asphalt concrete mixture. Saturation requirement is 70-80%.

Slide 57



Many asphalt technologists use the terminology wet and dry for conditioned and unconditioned, respectively.

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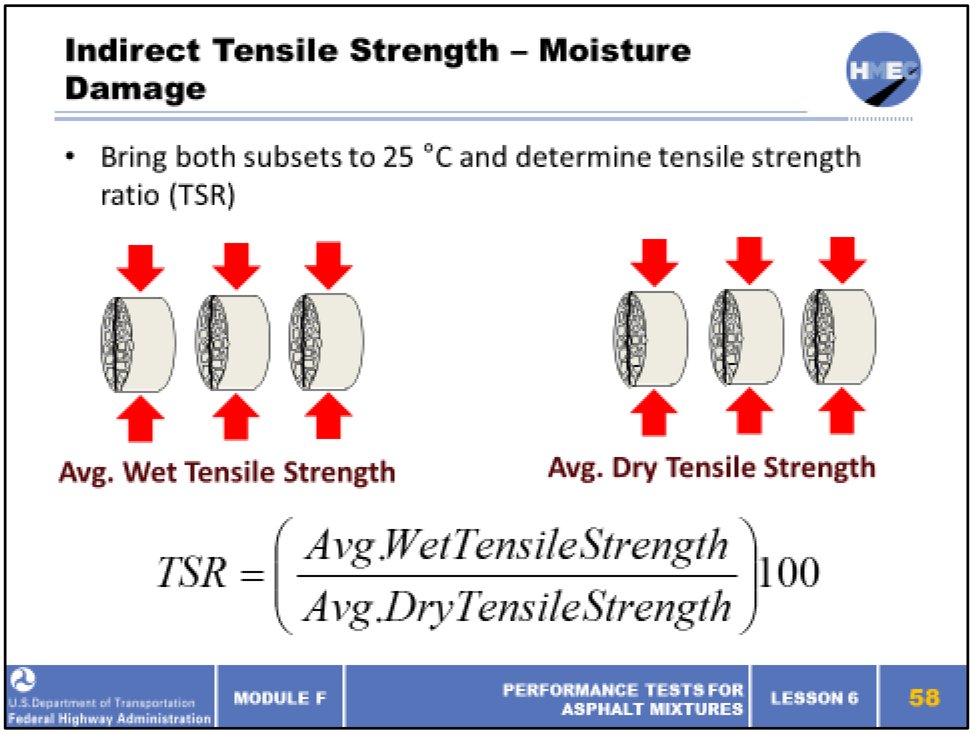

TSR is simply the ratio of the average tensile strength of a set of conditioned to a set of unconditioned specimens expressed as a percent. The minimum allowable TSR is 80%.

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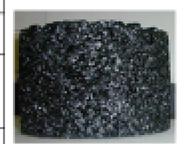
Tests for Evaluating Performance

Cantabro Test	
Test Designation	ASTM D6390
Purpose of Test & Its Use	Measure mixture's resistance to disintegration or abrasion
Performance Indicator	Mix disintegration component of durability
Test Conditions	Room temperature and 300 revolutions of the LA Abrasion machine
Geometry	Cylindrical specimen
Type of Specimen	Laboratory compacted (short-term or long-term aged)
Measurement	Weight of the initial and final specimen.
Outcome of Test	Ratio of weight loss of a compacted specimen



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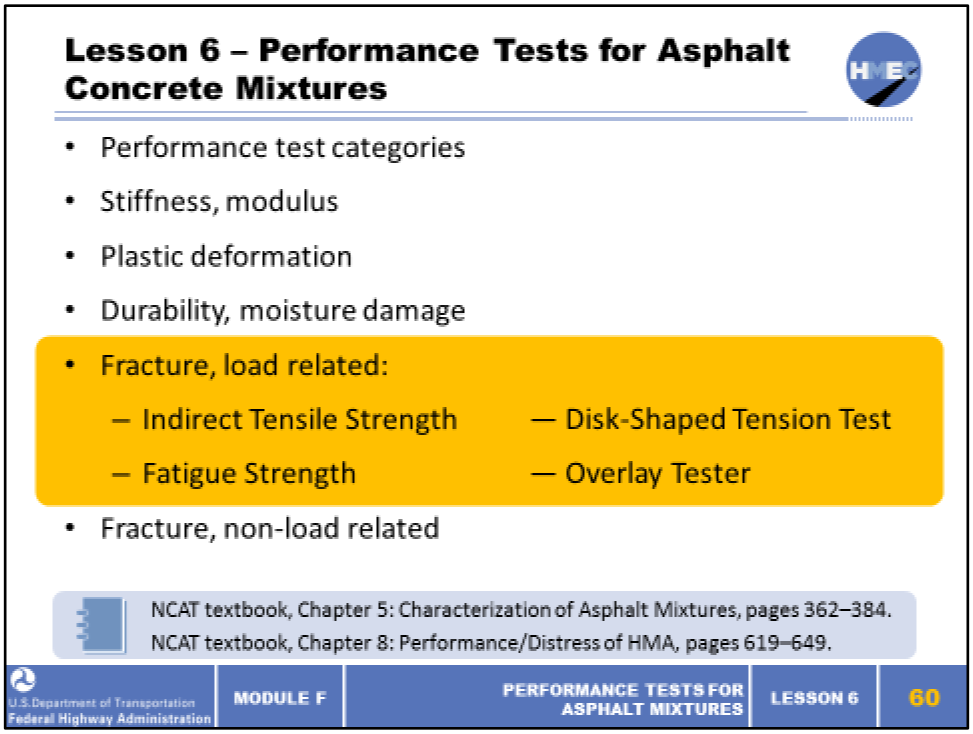
The Cantabro test is an empirical test that is being used by some agencies for designing open-graded mixtures as a wearing surface by measuring or determining the abrasion loss of compacted hot-mix asphalt specimens.

This test procedure measures the breakdown of compacted specimens utilizing the Los Angeles Abrasion machine. The percent of weight loss (Cantabro loss) is an indication of PFC durability and relates to the quantity and quality of the asphalt binder. A summary of the steps or procedure and the outcome from the test are briefly listed below:

- Compact and prepare a cylindrical specimen of the bituminous mixture.
- After weighing the specimen, place that mixture in the LA Abrasion machine.
- Rotate the Los Angeles machine at a speed of 30–33 revolutions per minute for 300 revolutions.
- After the 300 revolutions, discard the loose material broken off the test specimen. Do not include any of this material in the weight.
- Weigh the test specimen.
- Calculate the Cantabro Loss as the difference between the initial and final weight of the test specimen divided by the initial weight and report this value as a percentage of the initial weight.

Slide 60



The fundamental properties included related to evaluating fracture for predicting load related cracking include strength, modulus (which has already been discussed in an earlier section of this lesson), energy, dissipated energy, failure strain, and work.

Slide 61

Tests for Evaluating Performance

Indirect Tensile Strength	
Test Designation	ASTM D6931
Purpose of Test & Its Use	Measure of resistance to cracking within the intermediate test temperature range
Performance Indicator	Fatigue cracking
Test Conditions	Controlled ram rate for measuring tensile strength
Specimen Geometry	Cylindrical specimen with minimum height & diameter
Type of Specimen	Cores and laboratory compacted (short-term and long term aged)
Measurement	Load and displacement measured over time with constant ram rate
Outcome of Test	Tensile strength and failure strain as a function of test temperature

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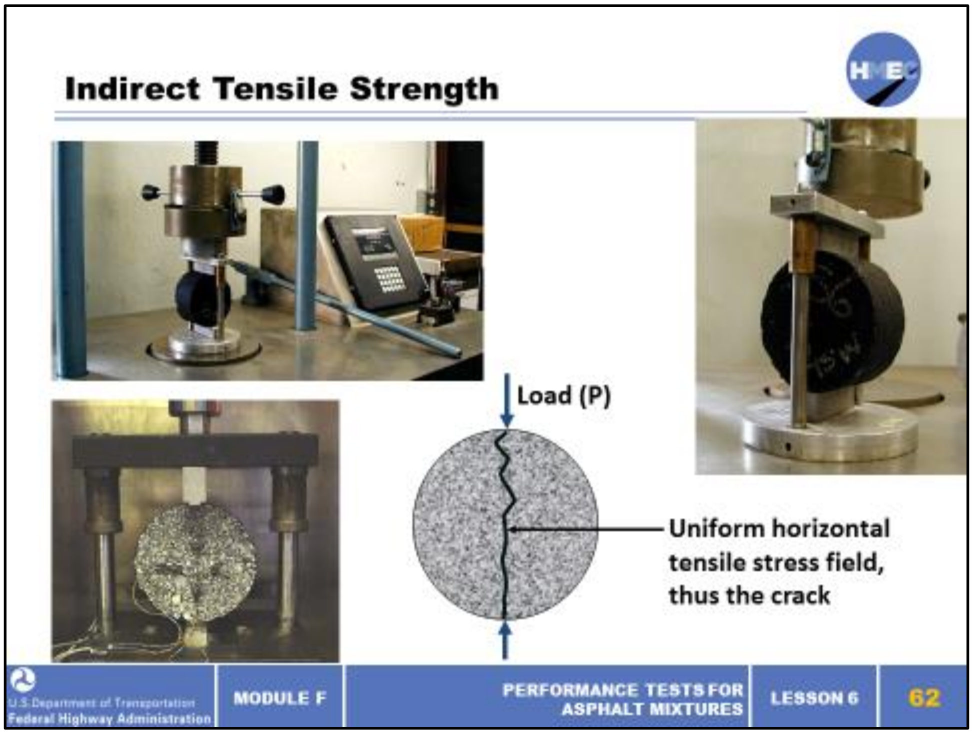
The indirect tensile strength test is used by some agencies for measuring two important properties: the indirect tensile strength and the tensile strain at failure. This test was used extensively within the Asphalt-Aggregate Mixture Analysis System (AAMAS) to estimate the fatigue damage relationship of tensile strain versus number of load cycles to failure.

Slide 62

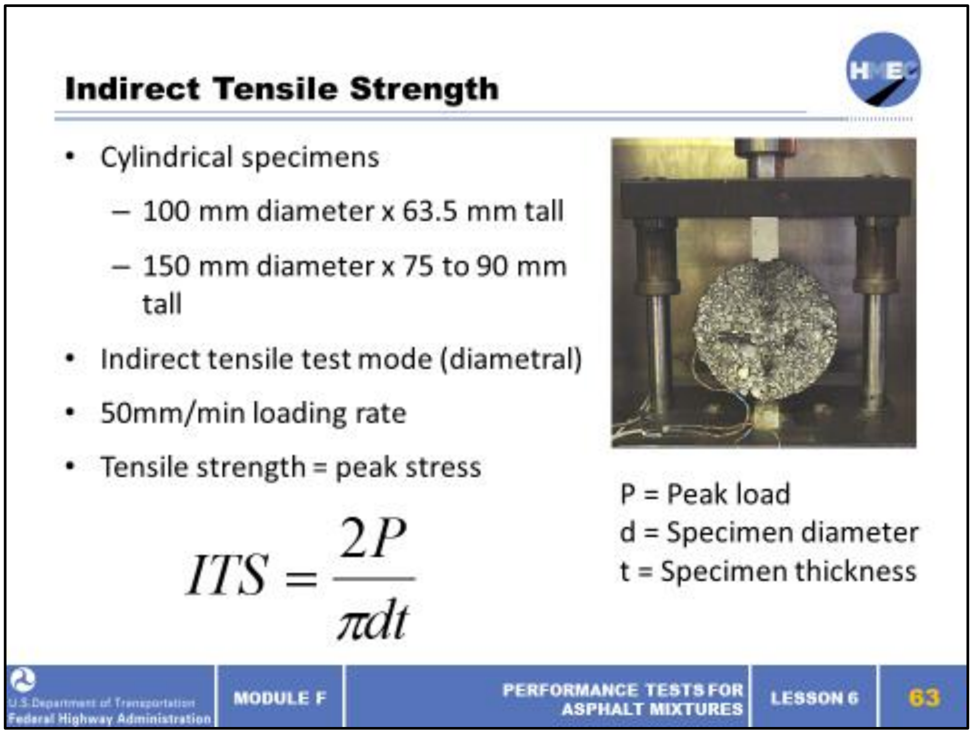


The indirect tensile test is performed in accordance with ASTM D6931 within the intermediate temperature range. The load is applied along the diametral axis of the indirect tensile test at a constant ram rate of 2 in. per minute. As noted in the previous slide, the total resilient modulus and tensile strain at failure, as well as the indirect tensile strength were used in the AAMAS procedure for evaluating the fatigue strength of asphalt concrete mixtures.

The test is presented schematically in this slide.

The test is destructive and can be performed in a short period of time. This is probably another reason for its popularity over the years. One of the primary disadvantages of the test is the actual loading condition. Because the specimen is in tension, the test is very sensitive to the asphalt binder used in the mixture. The test probably provides limited information about the aggregate contribution to strength.

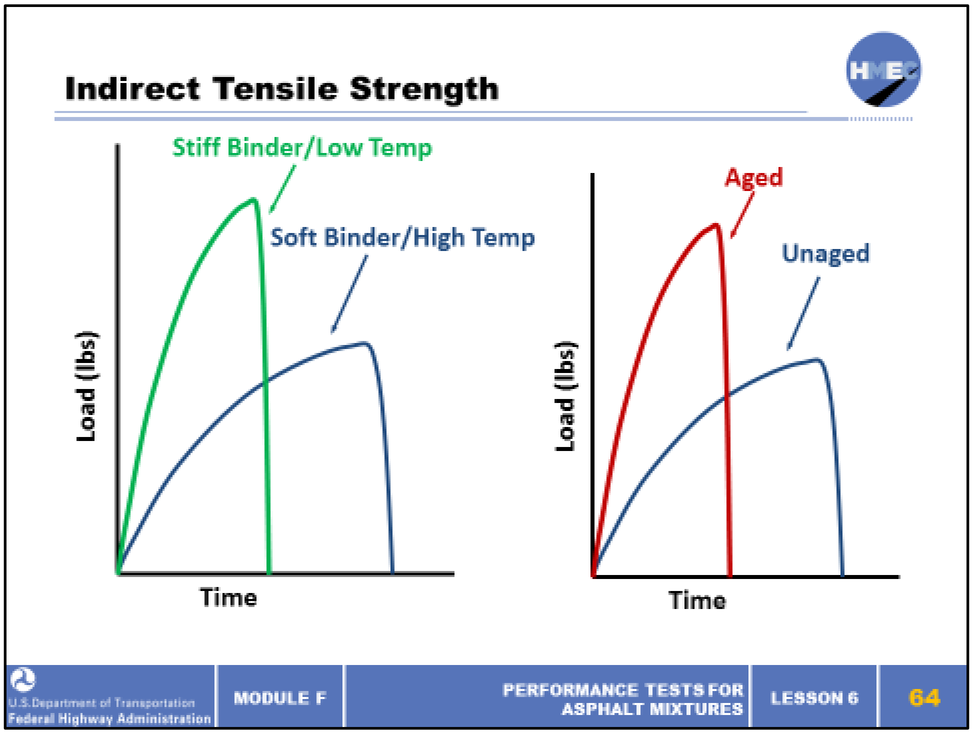
Slide 63



The test has historically been conducted on 63.5 mm tall by 100 mm diameter cylindrically shaped specimens compacted using impact or kneading methods and field cores may also be used. The test is conducted in the indirect tensile mode (diametral). Larger diameter specimens (150 mm diameter) are now typically prepared and tested. The height of the 150 mm diameter specimens can vary between 75 to 90 mm.

Load is applied at a constant strain rate of 50 mm/min (2 in. /min) until failure occurs. The strength of the mixture is simply the maximum stress observed. The relationship used to calculate tensile strength is included in the slide. The test may be conducted over a range of temperatures and aging conditions. Strain is sometimes monitored during the test also, depending on how the data will be used.

Slide 64



As shown, the results or outcome from the test are sensitive to the binder and age of the test specimen, as well as temperature. The work or energy can be computed or determined from the test if the horizontal displacements are measure during the test.

Slide 65

Tests for Evaluating Performance

Fatigue Strength/Cracking	
Test Designation	AASHTO T 321
Purpose of Test & Its Use	Measure of resistance to fatigue cracking; provides input to the MEPDG & other fatigue cracking models
Performance Indicator	Fatigue cracking
Test Conditions	Controlled strain or stress for measuring number of load cycle to failure
Specimen Geometry	Beam specimen
Type of Specimen	Laboratory compacted (short- or long-term aged)
Measurement	Displacement and applied load of flexural beam for monitoring beam stiffness
Outcome of Test	Fatigue cracking coefficients and number of load cycles to defined "failure" for different strain levels

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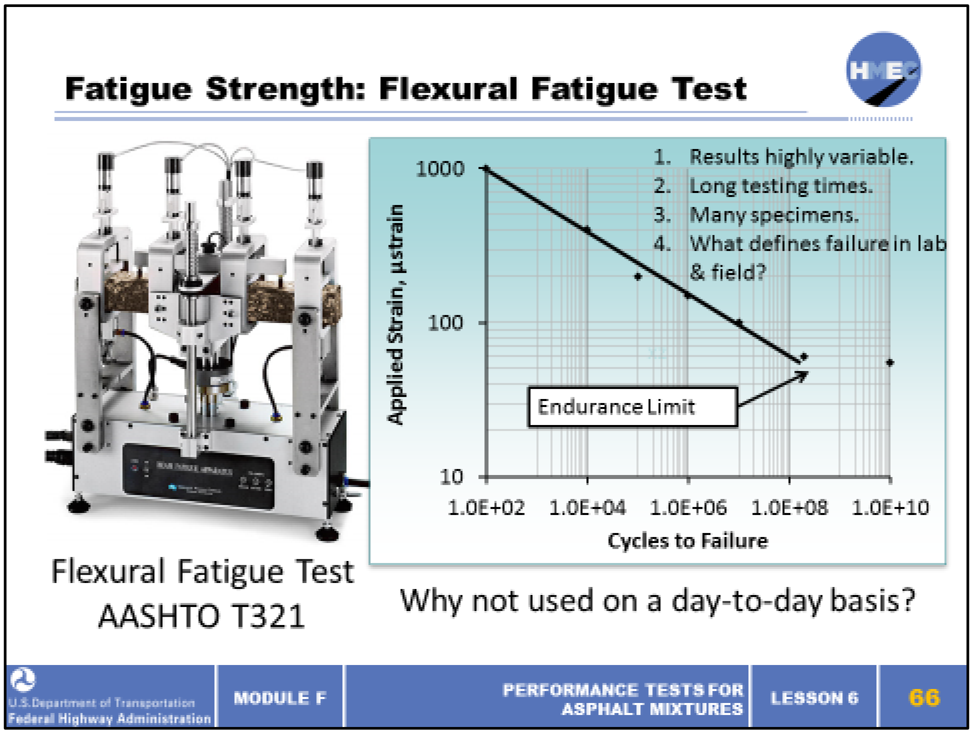
The fatigue strength or cracking test is used by few agencies for measuring the fatigue cracking coefficients. This test is primarily used for research purposes and is not used on a day-to-day basis for mixture design.

Slide 66


Fatigue Strength: Flexural Fatigue Test
HMEC
1000
100
10
Applied Strain, µstrain
1.0E+02
1.0E+04
1.0E+06
1.0E+08
1.0E+10
Cycles to Failure
1. Results highly variable.
2. Long testing times.
3. Many specimens.
4. What defines failure in lab & field?
Endurance Limit
Flexural Fatigue Test
AASHTO T321
Why not used on a day-to-day basis?
U.S. Department of Transportation
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MODULE F
PERFORMANCE TESTS FOR ASPHALT MIXTURES
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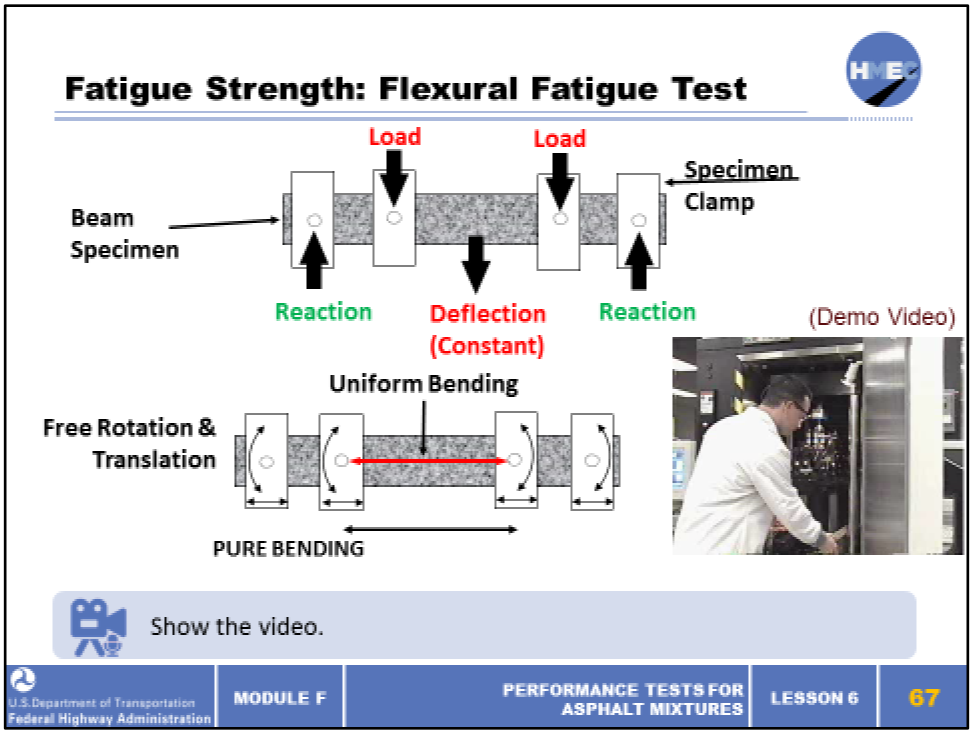
Slide 67



Fatigue Strength: Flexural Fatigue Test
HMEC
Load
Load
Specimen
Clamp
Beam
Specimen
Reaction
Deflection
(Constant)
Reaction
(Demo Video)
Uniform Bending
Free Rotation &
Translation
PURE BENDING
Show the video.
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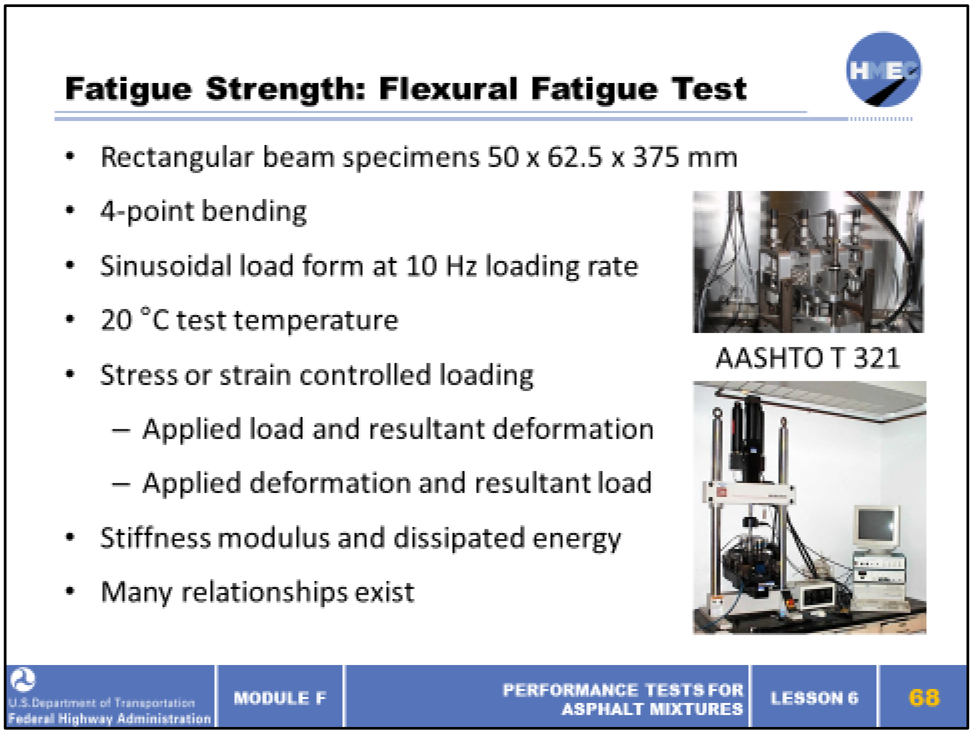
Slide 68



A flexural fatigue test apparatus is shown in this slide. The top photo shows a close-up of the test apparatus which is located in the test machine in the bottom photo. Small closed-loop servo-hydraulic (top) or servo-pneumatic (bottom) test machines are used with highly specialized four point loading fatigue fixtures that are very expensive to manufacture.

Slide 69

Tests for Evaluating Performance

Disk-Shaped Compact Tension Test	
Test Designation	ASTM D7313
Purpose of Test & Its Use	Measure of resistance to cracking and determines crack propagation parameters for fracture mechanic-based models
Performance Indicator	Fatigue (bottom-up and top-down) and low temperature cracking
Test Conditions	Controlled ram rate for measuring the opening of a crack
Specimen Geometry	Cylindrical specimen with notch to create crack
Type of Specimen	Core or laboratory compacted (long-term aged)
Measurement	Applied load & displacement across a crack over time
Outcome of Test	Fracture energy; area under load-displacement curve

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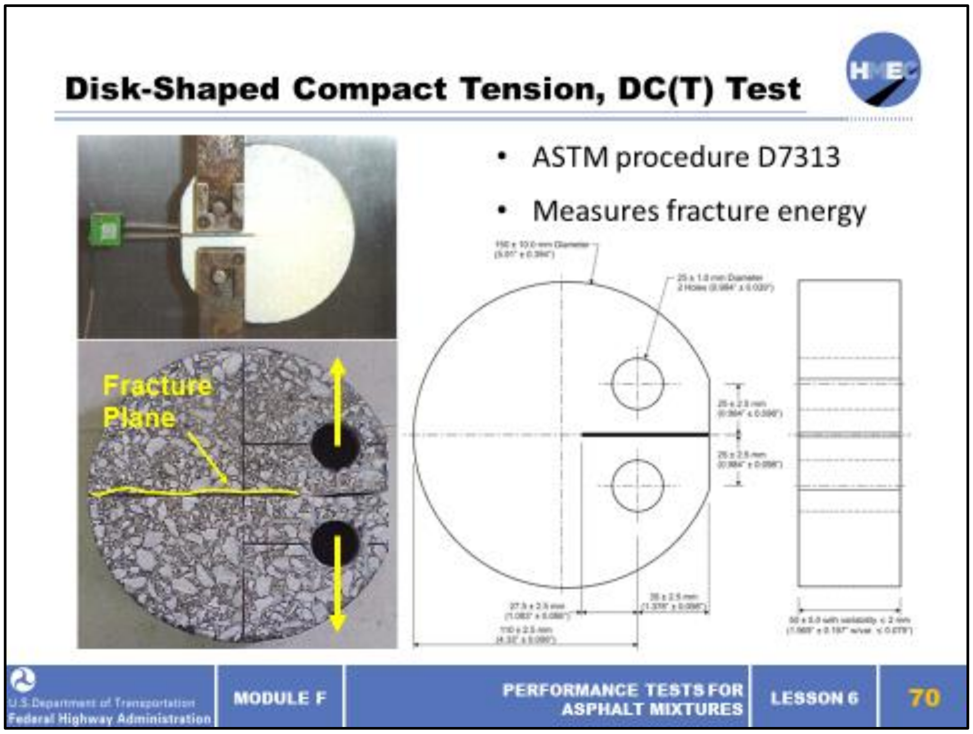
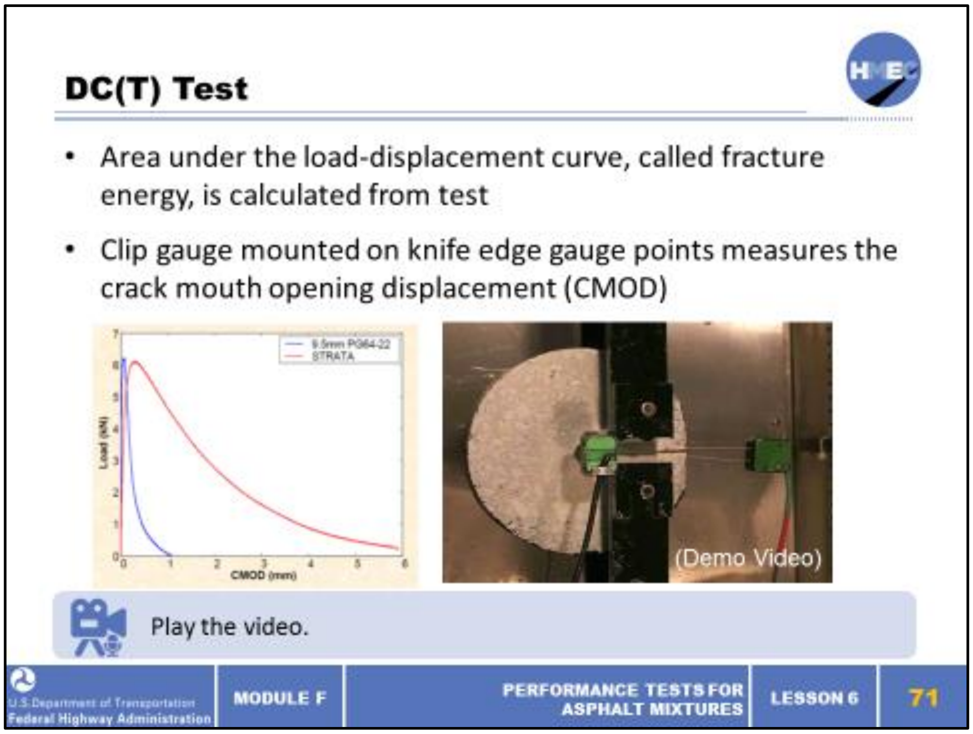
The disk-shaped compact tension test is a new test that was recently developed and no agency is using it on a routine basis; some agencies are using the test for research purposes.

Slide 70



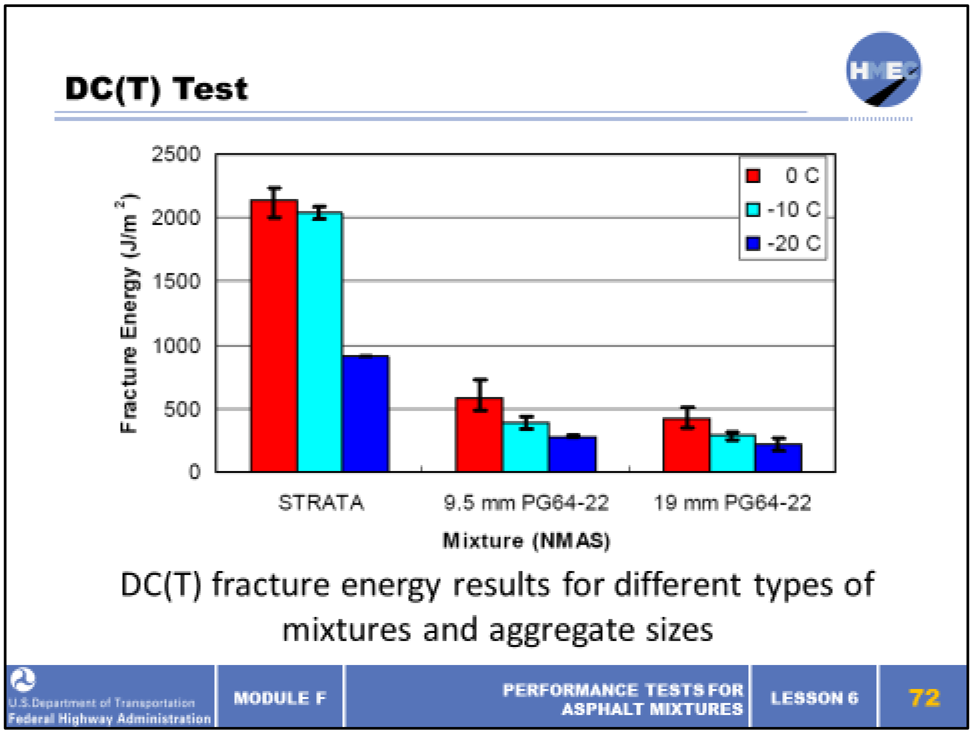
The test method measures the resistance to crack opening, while also measuring the vertical expansion of the crack opening on the edge of the specimen. By determining the area under the load-displacement curve, one can calculate "fracture energy." Mixtures with higher fracture energy will have a higher resistance to cracking.

Slide 71



The higher the fracture energy of a mixture, the better the mixture's resistance to cracking and crack propagation. Higher fracture energies simply means that the mixture can resist greater tensile loading conditions without cracking. Higher fracture energy also means it will take longer for a crack to propagate once it develops.

Slide 72



The material called "strata" is a reflective crack relief interlayer mixture that is specifically design to be resistant to low and intermediate temperature thermal and load associated cracking. As shown in the slide, the strata mixture has much higher fracture energy. In addition, the mixture with the smaller size aggregate (9.5 mm) has slightly greater fracture energy than the same mix but with the larger size aggregate.

Slide 73

Tests for Evaluating Performance

Overlay Tester	
Test Designation	TxDOT Tex-248-F
Purpose of Test & Its Use	Measure of resistance to cracking
Performance Indicator	Reflection and fatigue cracking
Test Conditions	Controlled repeated horizontal displacements and tensile loadings (10 sec. load pulse, triangular load pulse)
Specimen Geometry	Beam specimen
Type of Specimen	Laboratory compacted (short- and Long-term aged)
Measurement	Average indirect tensile strength for two sets of specimens: one dry and one wet conditioned
Outcome of Test	Number of load cycles to failure

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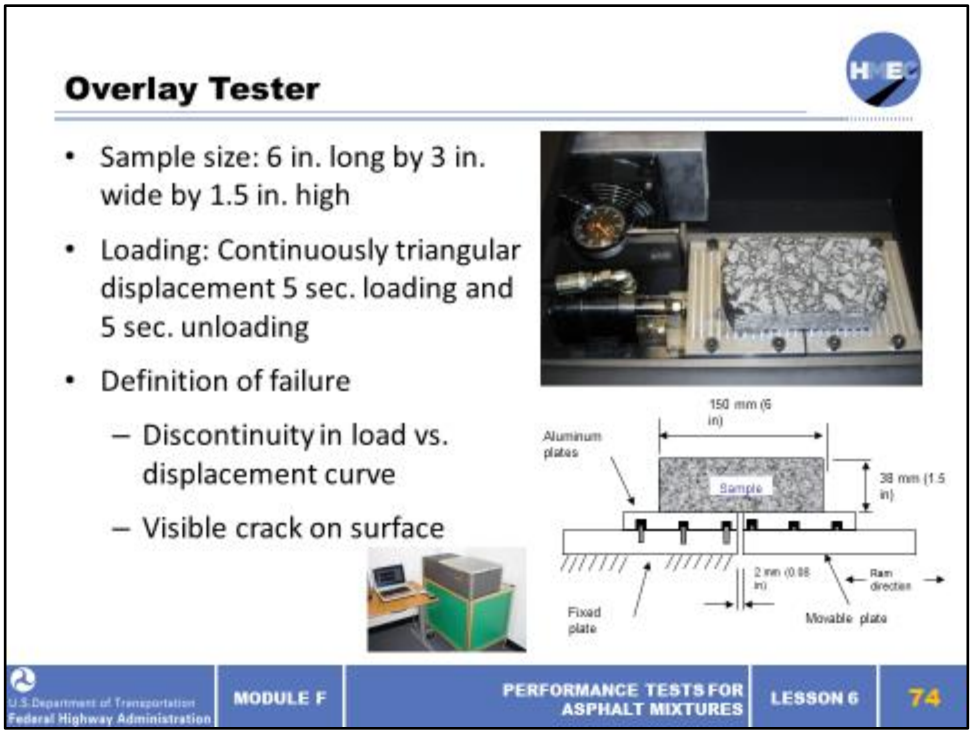
The overlay tester is used by some agencies for measuring the mixture's resistance to repeated tensile loadings; tension and reflection cracking. This test is used by a few agencies for confirming the mixture design for balancing between distortion and fracture.

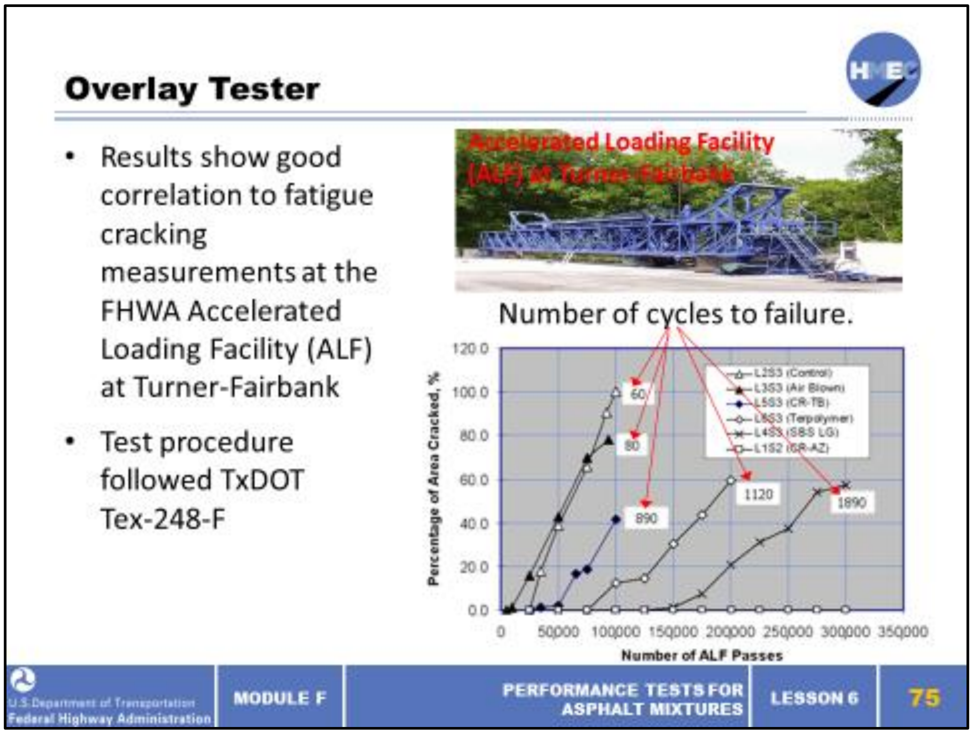
Slide 74



The overlay tester, originally developed by the Texas Transportation Institute (TTI), has shown to provide good correlations to field cracking performance.

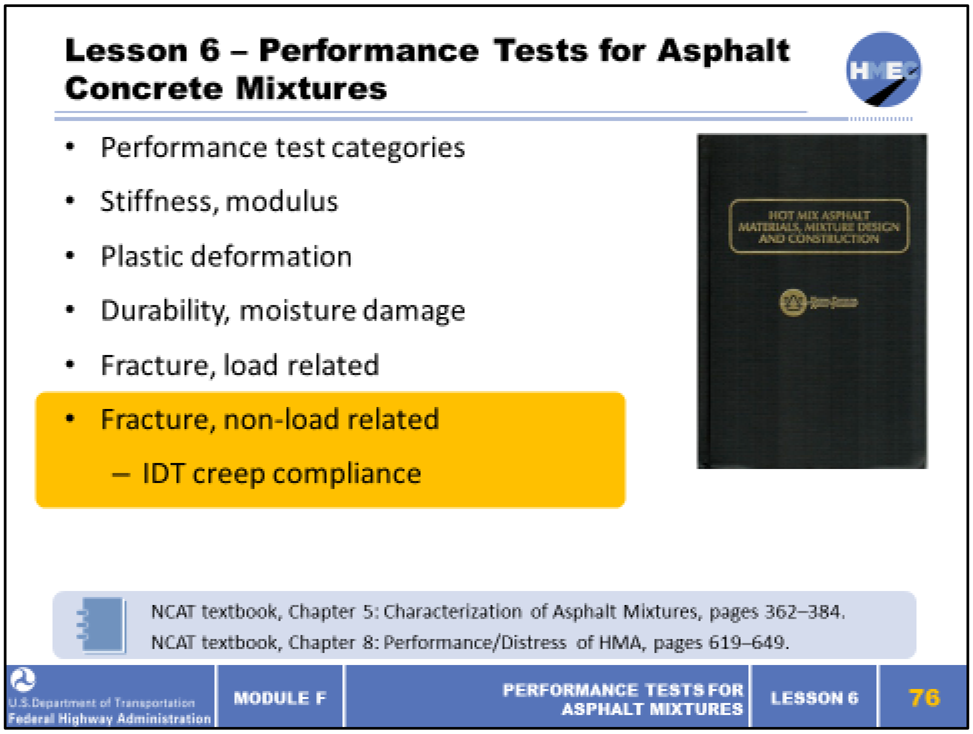
Continuously triangular displacement: loading is applied to the test specimen so that an increasing displacement is applied at a constant rate of increase for 5 seconds and then the load is removed for 5 seconds so the specimen can recover some of the displacement.

Slide 75



The ALF at Turner-Fairbank is an accelerated loading facility used to verify/validate materials and design methodologies for pavement design. The ALF is operated by the FHWA.

Slide 76



The fundamental properties included related to evaluating fracture for predicting non-load related cracking include creep compliance and strength. The TSRST only provides the critical cracking temperature and is considered an empirical test.

Slide 77

Tests for Evaluating Performance

Indirect Tensile Creep Compliance & Strength	
Test Designation	AASHTO T 322
Purpose of Test & Its Use	Measure of resistance to low temperature cracking.
Performance Indicator	Thermal cracking
Test Conditions	Constant applied stress to a specimen over time; measurements made over a range of temperature and times
Specimen Geometry	Cylindrical specimen
Type of Specimen	Cores or laboratory compacted (long-term aged)
Measurement	Average horizontal and vertical displacements along with the peak load
Outcome of Test	Creep compliance and strength

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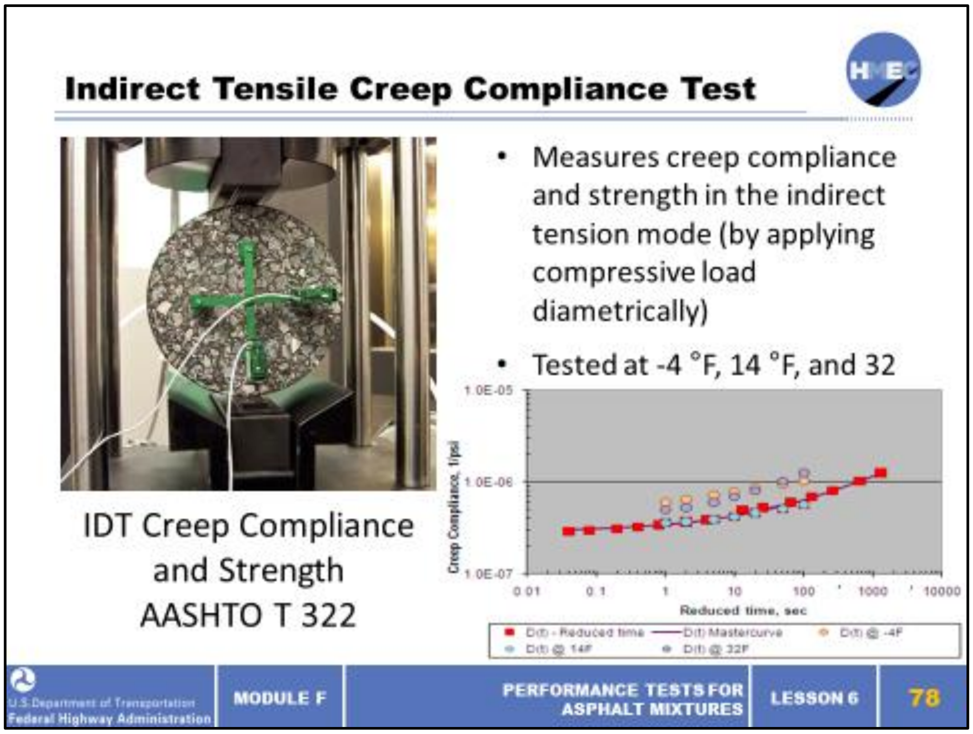
The indirect tensile creep compliance and strength test is used by mostly those agencies in cold climates for predicting thermal cracking.

Slide 78



This test is used to measure the IDT creep compliance and strength to evaluate the mixture's resistance to low temperature cracking.

The compliance and strength must be used in a thermo-viscoelastic analysis to characterize the low temperature cracking potential of a mixture. There are two methods that can be used:

1. DARWin-ME: Provides a prediction of the extent of thermal cracking.
2. LTSTRESS: Provides a critical cracking temperature. (This is a program that calculates the critical cracking temperature of an asphalt concrete mixture (similar to the DARWin-ME software program).

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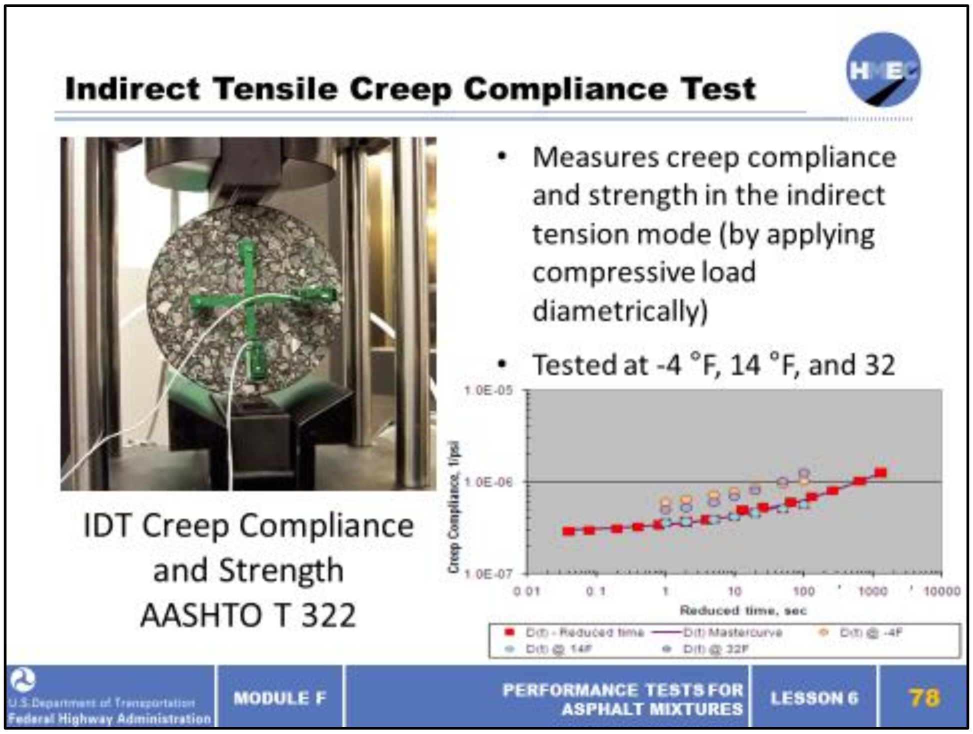
Slide 79

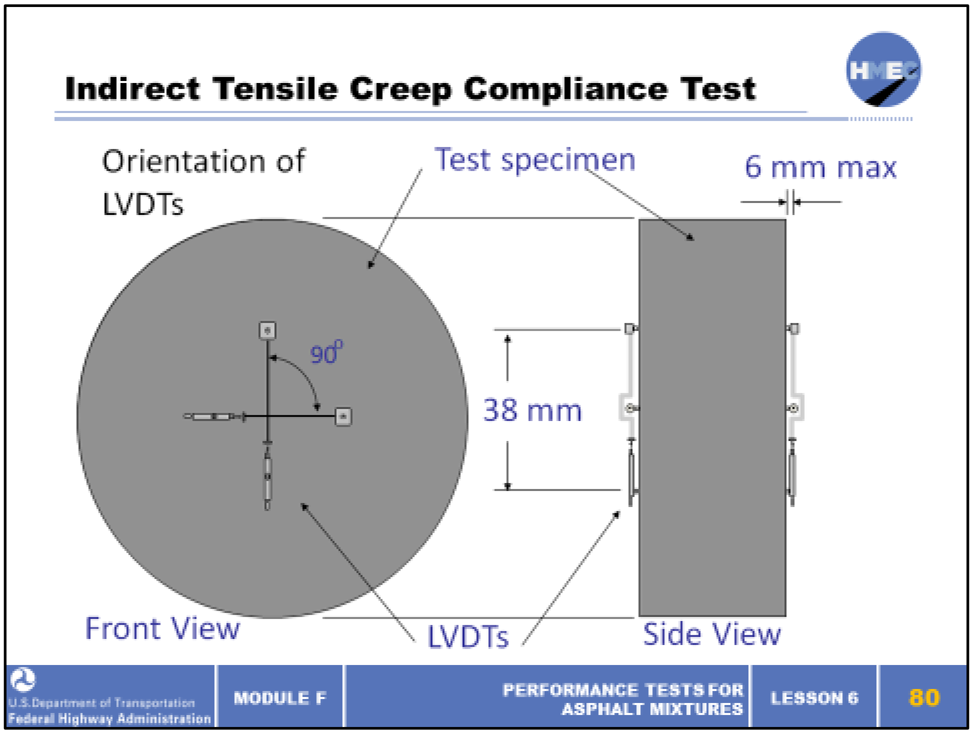


Indirect tensile creep and strength test methods were refined for use in Superpave during SHRP. The methods are currently found in the AASHTO T 322. The strength test is essentially the ASTM D4123 strength test performed on 150 mm (6 in) diameter SGC specimens sawn to a thickness of 50 mm with the addition of measuring strains during loading.

The creep test is conducted on the same specimens prior to strength testing and both horizontal and vertical strains are monitored.

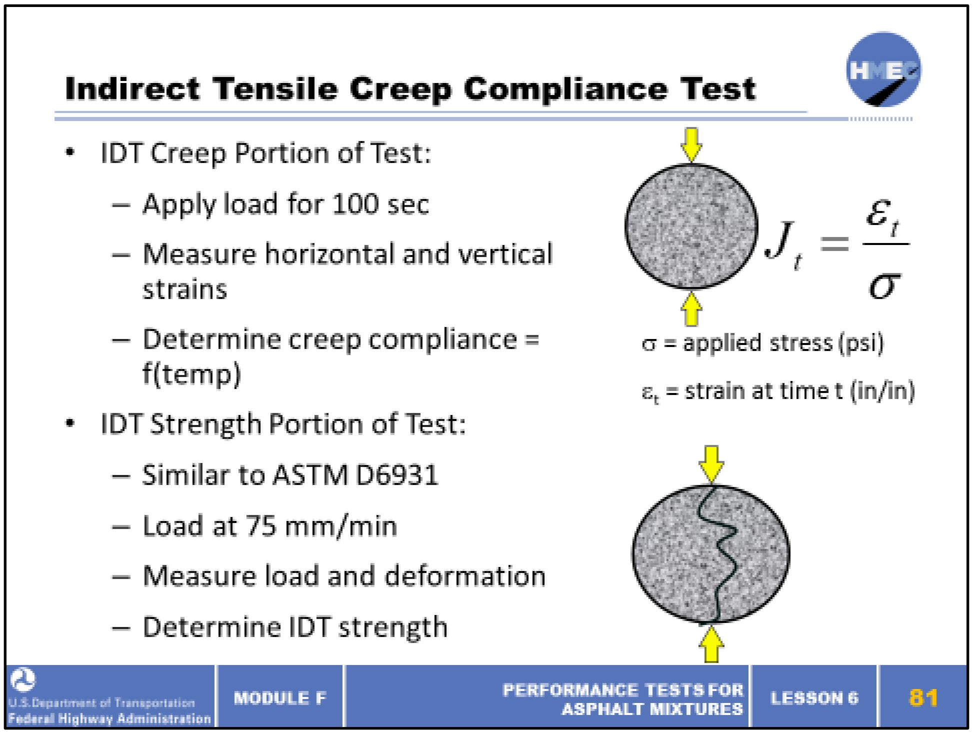
The testing is conducted at temperatures ranging from 0 to -30 °C. The test is conducted in the indirect tensile mode (diametral). The tests may be conducted over a range of temperatures and aging conditions to determine strength and creep compliance.

Slide 80



Typical range of creep compliance results, as noted in SHRP Report A-357, was 3 x 10^{-11} Pa^{-1} to 4 x 10^{-9} Pa^{-1}.

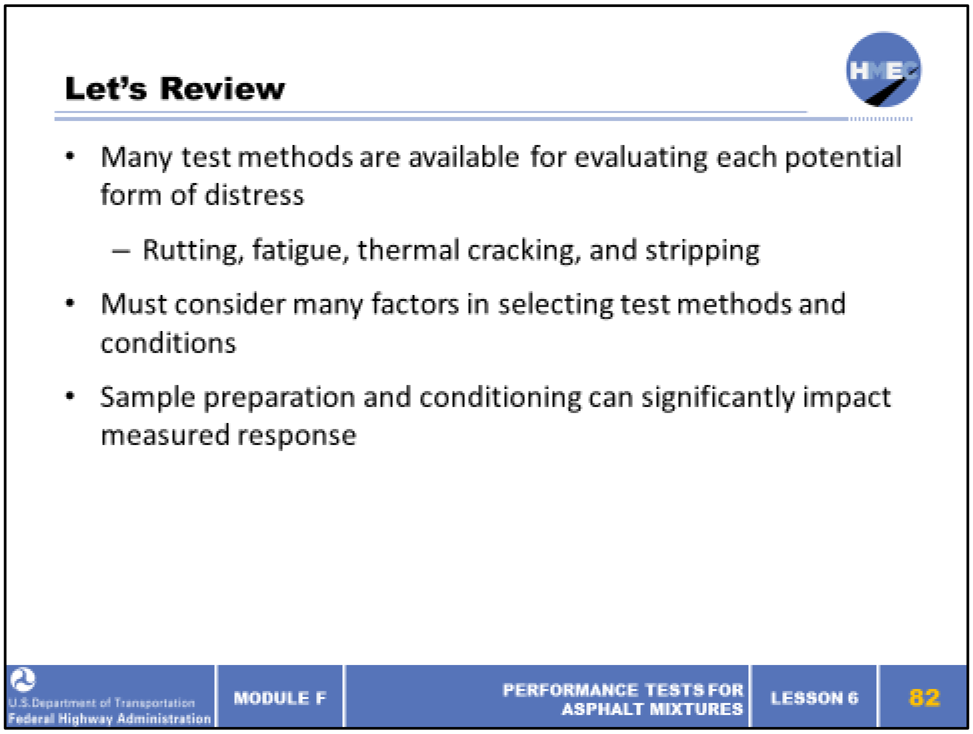
Slide 81



The stress and strains measured are used to determine the J_t and Poisson's ratio of the material.

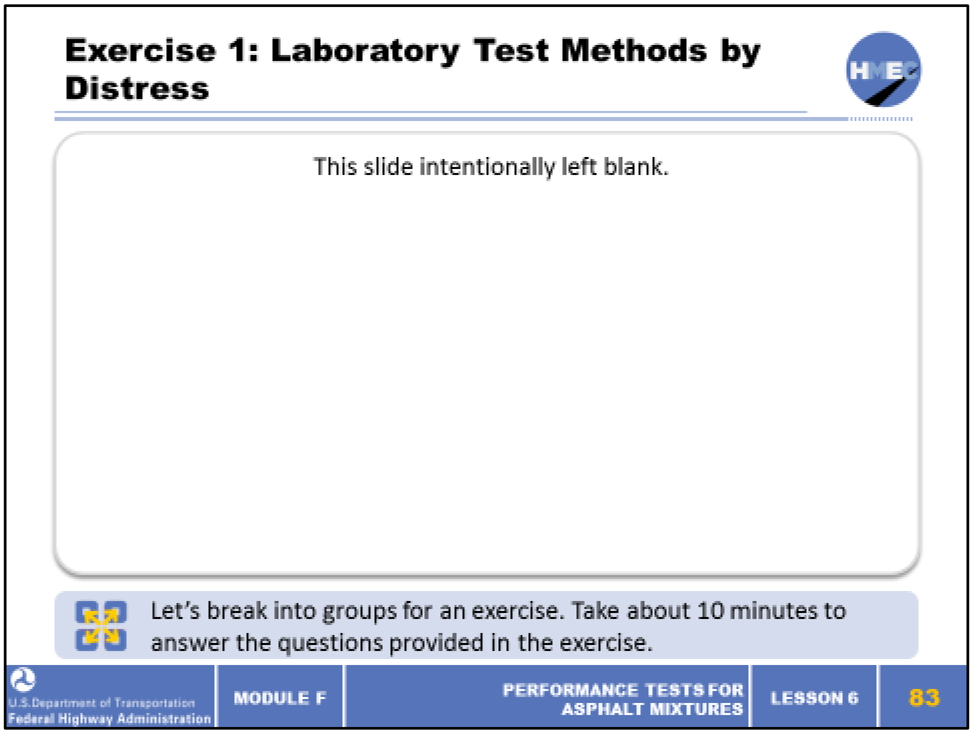
The Superpave indirect tensile strength test is illustrated schematically in this slide. It is very similar to the ASTM D6931 strength test but is performed on SGC specimens trimmed to 50 mm. The test may be conducted over a range of the temperatures and aging conditions.

Slide 82



As noted at the beginning of this lesson there are many tests available for use to evaluate a mixture's resistance to one or more distresses. As demonstrated, however, there is no one test that can be used to predict or design mixtures for all distresses. The user needs to use multiple tests for predicting performance and balancing or optimizing the mixture to ensure long-lasting service.

Slide 83


Exercise 1: Laboratory Test Methods by Distress
HMEC
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Let's break into groups for an exercise. Take about 10 minutes to answer the questions provided in the exercise.
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MODULE F
PERFORMANCE TESTS FOR ASPHALT MIXTURES
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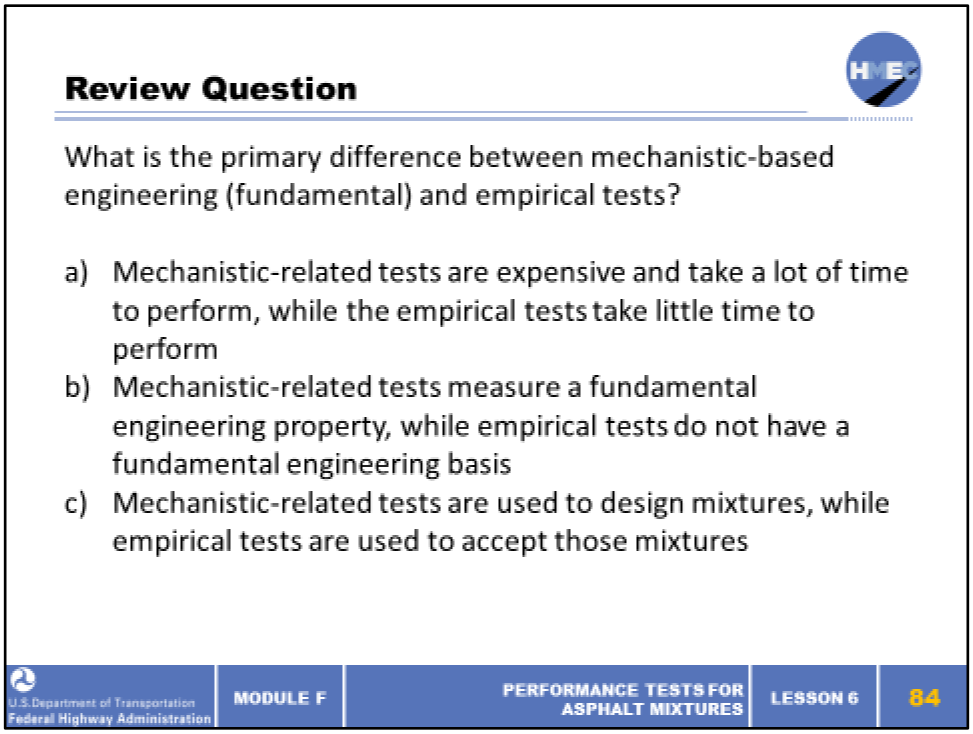
Slide 84


Review Question
What is the primary difference between mechanistic-based engineering (fundamental) and empirical tests?
a) Mechanistic-related tests are expensive and take a lot of time to perform, while the empirical tests take little time to perform
b) Mechanistic-related tests measure a fundamental engineering property, while empirical tests do not have a fundamental engineering basis
c) Mechanistic-related tests are used to design mixtures, while empirical tests are used to accept those mixtures
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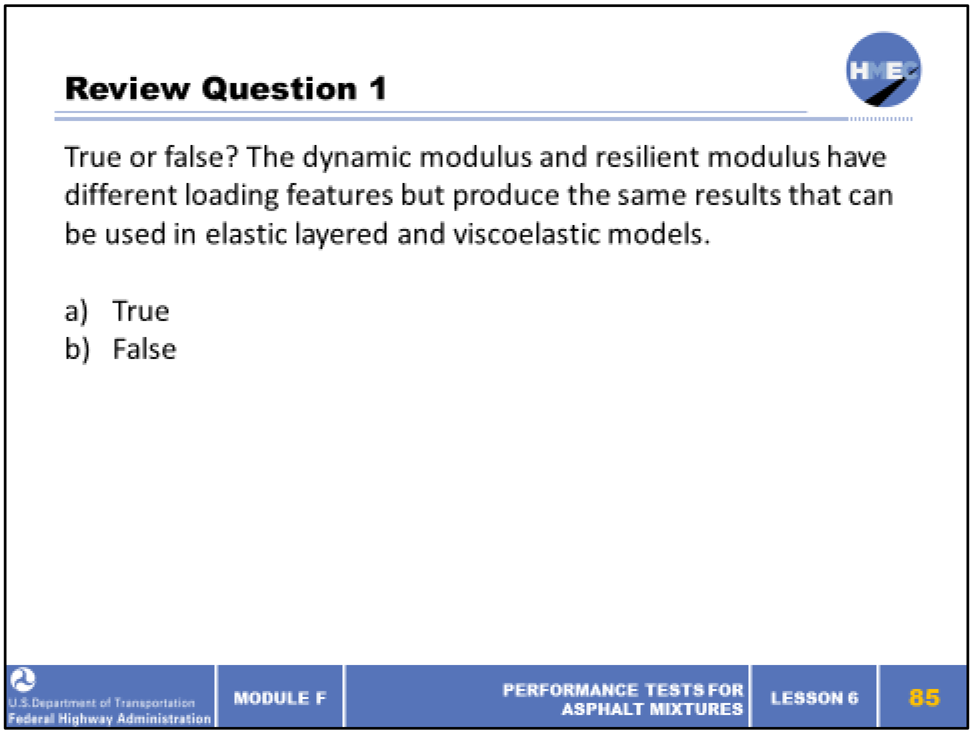
Slide 85


Review Question 1
HMEC
True or false? The dynamic modulus and resilient modulus have different loading features but produce the same results that can be used in elastic layered and viscoelastic models.
a) True
b) False
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PERFORMANCE TESTS FOR ASPHALT MIXTURES
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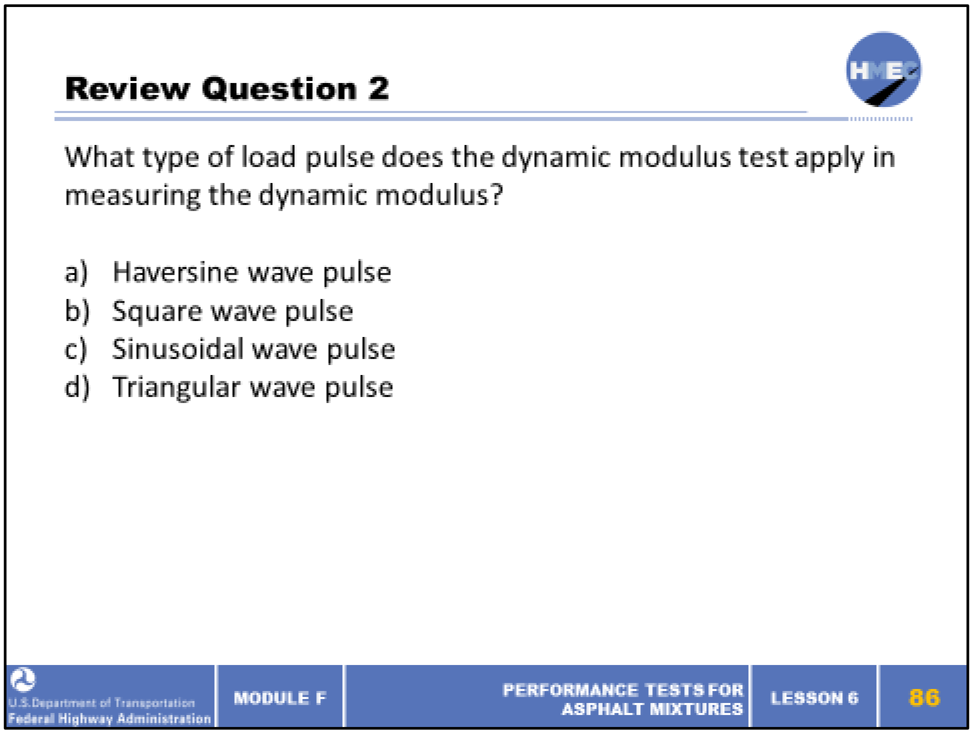
Review Question 2
What type of load pulse does the dynamic modulus test apply in measuring the dynamic modulus?
a) Haversine wave pulse
b) Square wave pulse
c) Sinusoidal wave pulse
d) Triangular wave pulse
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PERFORMANCE TESTS FOR ASPHALT MIXTURES
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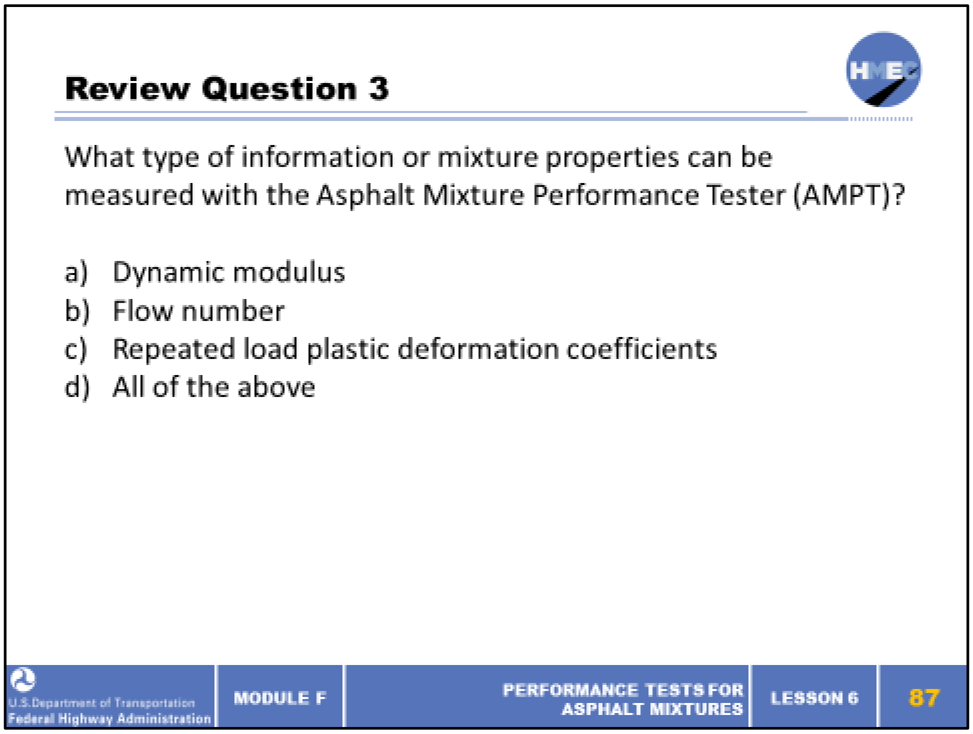
Slide 87


Review Question 3
HMEC
What type of information or mixture properties can be measured with the Asphalt Mixture Performance Tester (AMPT)?
a) Dynamic modulus
b) Flow number
c) Repeated load plastic deformation coefficients
d) All of the above
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Review Question 4

True or false? The flow number is defined as the number of loading cycles where the plastic axial strain rates starts to increase at an increasing rate.

a) True
b) False

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Review Question 5

How are the results from repeated load plastic deformation test used?

a) Determine the mixture's resistance to rutting
b) Determine the coefficients of the rut depth transfer function for inputs to the MEPDG
c) Predicting rutting over time
d) All of the above

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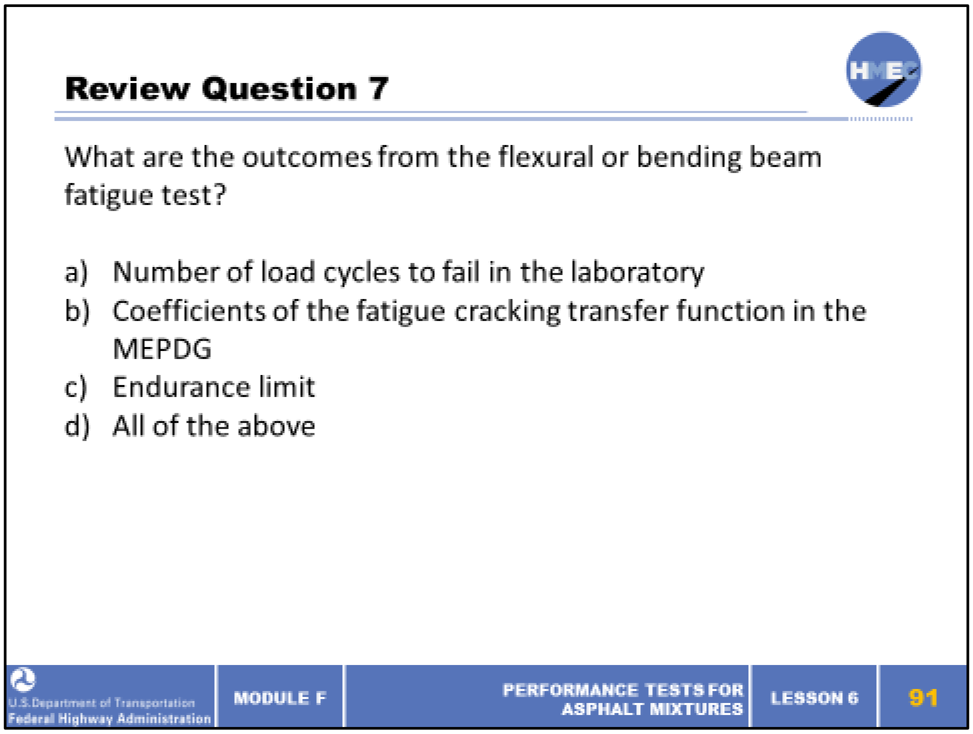
Slide 90

Review Question 6

What distresses or performance indicators are evaluated from the Hamburg wheel tracking test?

a) Fatigue cracking
b) Rutting
c) Rutting and durability
d) Thermal cracking

Slide 91


Review Question 7
What are the outcomes from the flexural or bending beam fatigue test?
a) Number of load cycles to fail in the laboratory
b) Coefficients of the fatigue cracking transfer function in the MEPDG
c) Endurance limit
d) All of the above
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Review Question 8

True or false? The IDT creep compliance test applies a repeated load to the diametral axis of the test specimen for predicting low temperature cracking.

a) True
b) False

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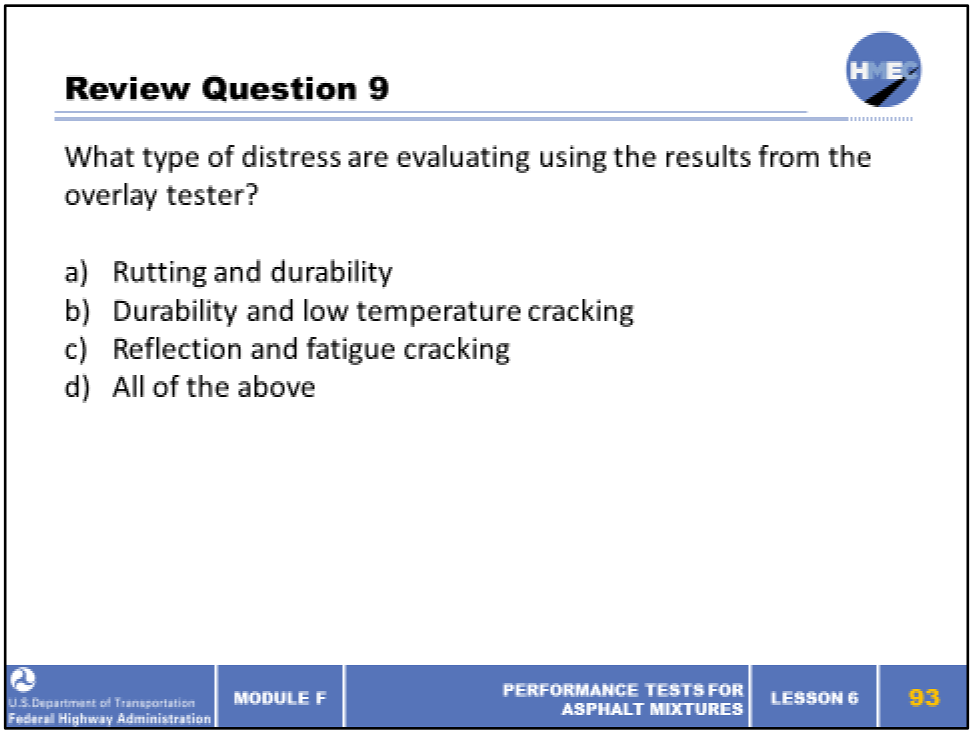
Slide 93


Review Question 9
What type of distress are evaluating using the results from the overlay tester?
a) Rutting and durability
b) Durability and low temperature cracking
c) Reflection and fatigue cracking
d) All of the above
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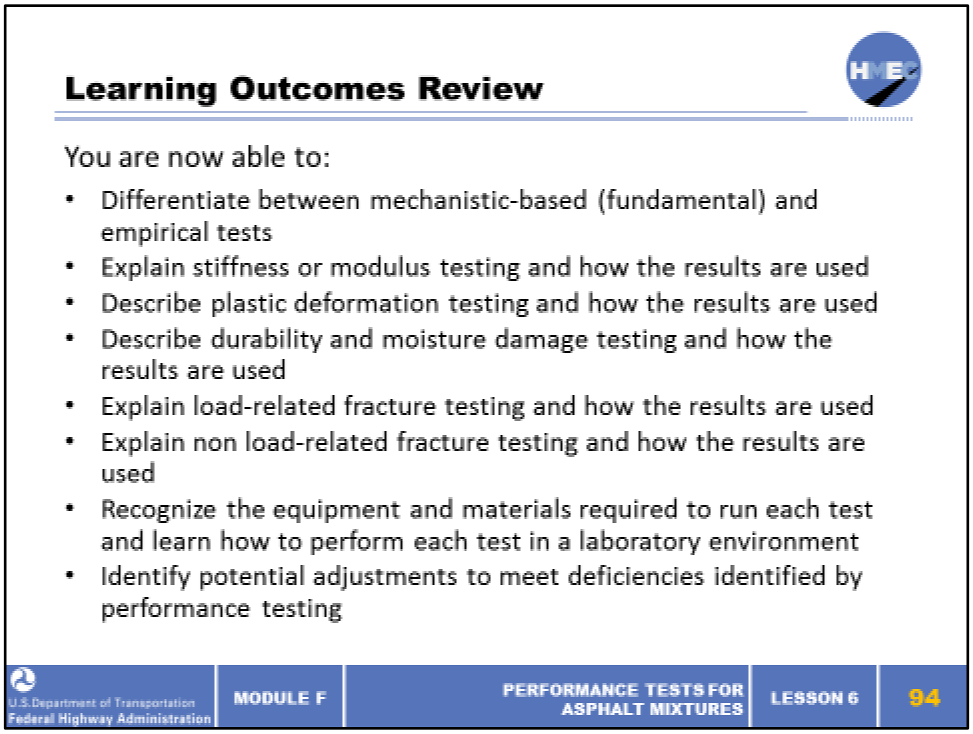
Slide 94


Learning Outcomes Review
HMEC
You are now able to:
• Differentiate between mechanistic-based (fundamental) and empirical tests
• Explain stiffness or modulus testing and how the results are used
• Describe plastic deformation testing and how the results are used
• Describe durability and moisture damage testing and how the results are used
• Explain load-related fracture testing and how the results are used
• Explain non load-related fracture testing and how the results are used
• Recognize the equipment and materials required to run each test and learn how to perform each test in a laboratory environment
• Identify potential adjustments to meet deficiencies identified by performance testing
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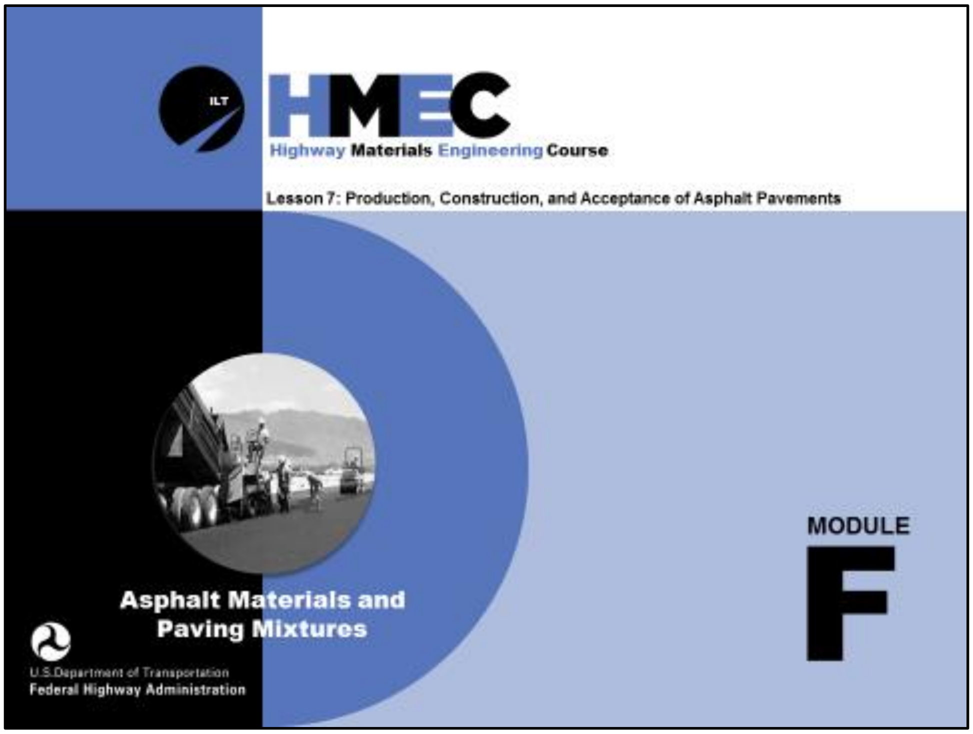
Slide 1


ILT
HMEC
Highway Materials Engineering Course
Lesson 7: Production, Construction, and Acceptance of Asphalt Pavements
Asphalt Materials and Paving Mixtures
U.S.Department of Transportation
Federal Highway Administration
MODULE
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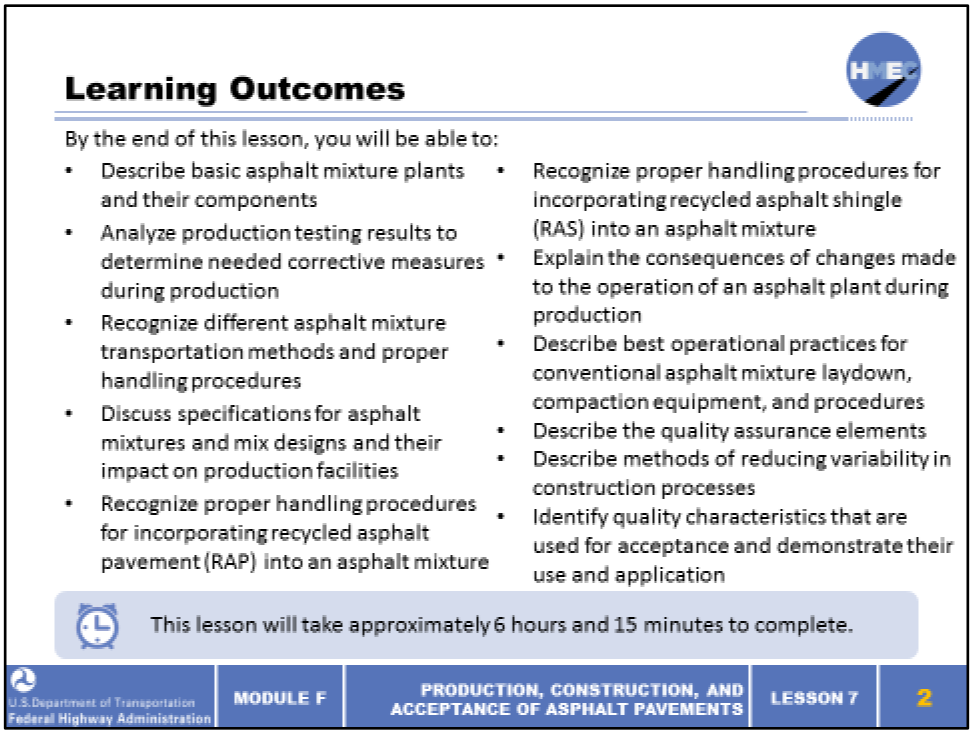
Slide 2


Learning Outcomes
HMEC
By the end of this lesson, you will be able to:
• Describe basic asphalt mixture plants and their components
• Analyze production testing results to determine needed corrective measures during production
• Recognize different asphalt mixture transportation methods and proper handling procedures
• Discuss specifications for asphalt mixtures and mix designs and their impact on production facilities
• Recognize proper handling procedures for incorporating recycled asphalt pavement (RAP) into an asphalt mixture
• Recognize proper handling procedures for incorporating recycled asphalt shingle (RAS) into an asphalt mixture
• Explain the consequences of changes made to the operation of an asphalt plant during production
• Describe best operational practices for conventional asphalt mixture laydown, compaction equipment, and procedures
• Describe the quality assurance elements
• Describe methods of reducing variability in construction processes
• Identify quality characteristics that are used for acceptance and demonstrate their use and application
This lesson will take approximately 6 hours and 15 minutes to complete.
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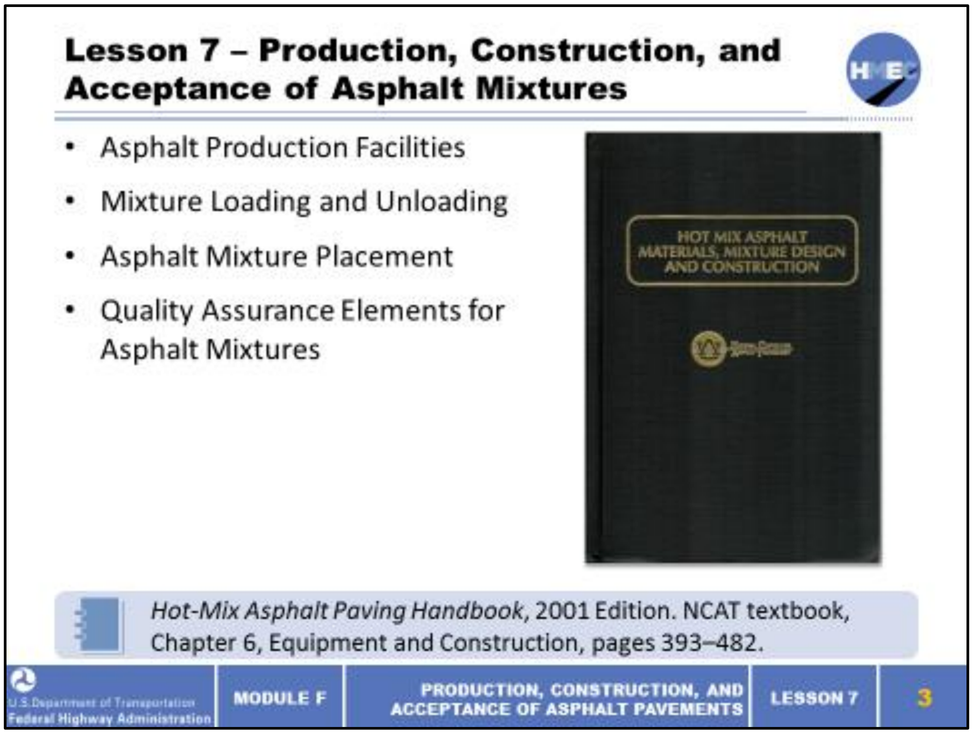
MODULE F
PRODUCTION, CONSTRUCTION, AND ACCEPTANCE OF ASPHALT PAVEMENTS
LESSON 7
2

Slide 3


Lesson 7 – Production, Construction, and Acceptance of Asphalt Mixtures
Asphalt Production Facilities
Mixture Loading and Unloading
Asphalt Mixture Placement
Quality Assurance Elements for Asphalt Mixtures
HOT MIX ASPHALT MATERIALS, MIXTURE DESIGN AND CONSTRUCTION
Hot-Mix Asphalt Paving Handbook, 2001 Edition. NCAT textbook, Chapter 6, Equipment and Construction, pages 393–482.
U.S. Department of Transportation Federal Highway Administration
MODULE F
PRODUCTION, CONSTRUCTION, AND ACCEPTANCE OF ASPHALT PAVEMENTS
LESSON 7
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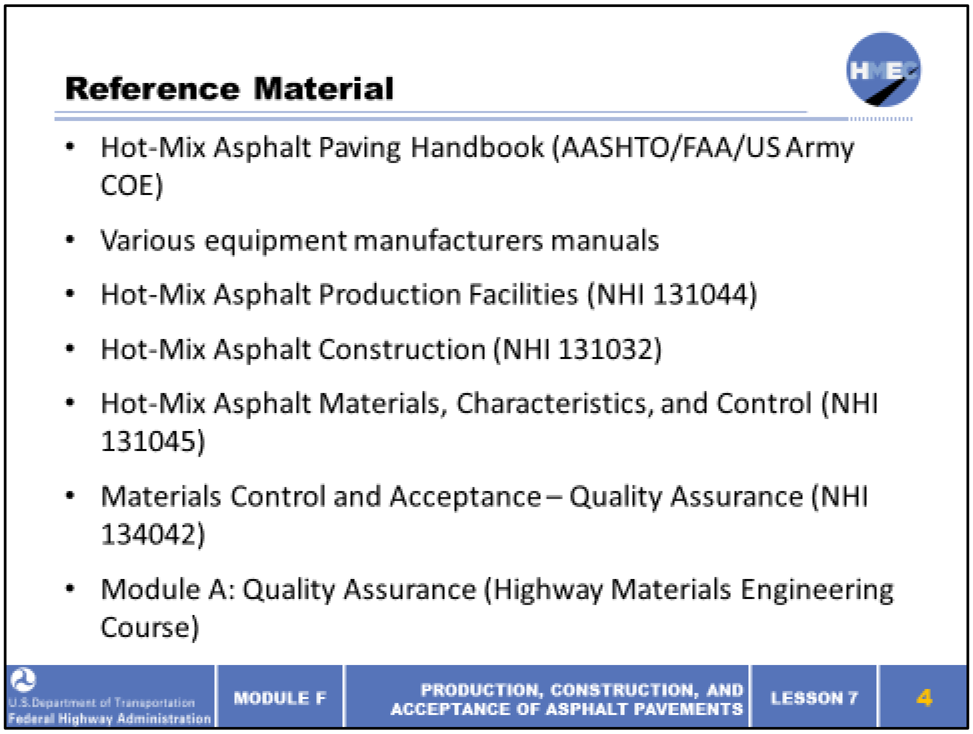
Slide 4


Reference Material
HMEC
• Hot-Mix Asphalt Paving Handbook (AASHTO/FAA/US Army COE)
• Various equipment manufacturers manuals
• Hot-Mix Asphalt Production Facilities (NHI 131044)
• Hot-Mix Asphalt Construction (NHI 131032)
• Hot-Mix Asphalt Materials, Characteristics, and Control (NHI 131045)
• Materials Control and Acceptance – Quality Assurance (NHI 134042)
• Module A: Quality Assurance (Highway Materials Engineering Course)
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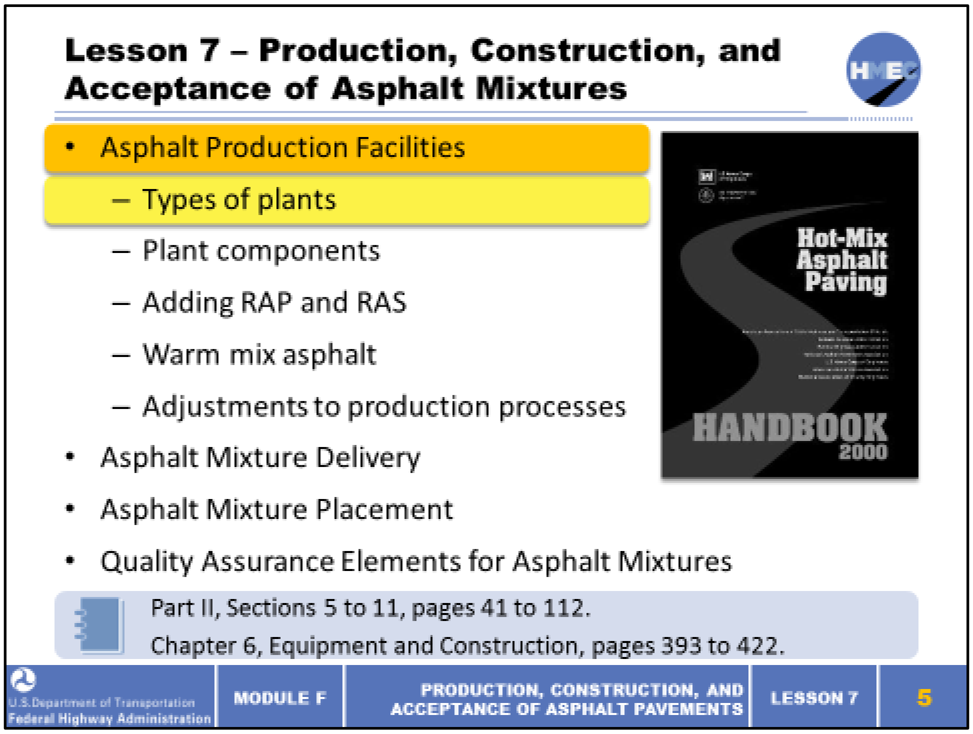
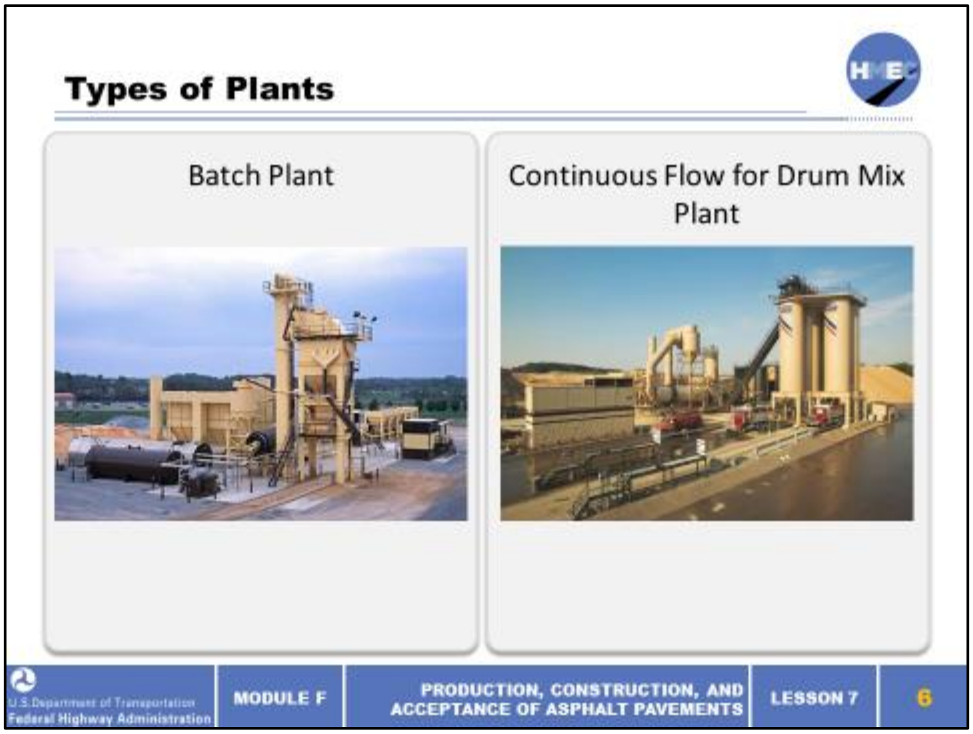
Slide 5



Most of the information covered within this lesson comes from many different sources but was generally taken from Part II of *the Hot-Mix Asphalt Paving Handbook*. However, the first section of Chapter 6 of the NCAT textbook (*Hot Mix Asphalt Materials, Mixture Design and Construction*) is on hot mix asphalt facilities.

Slide 6



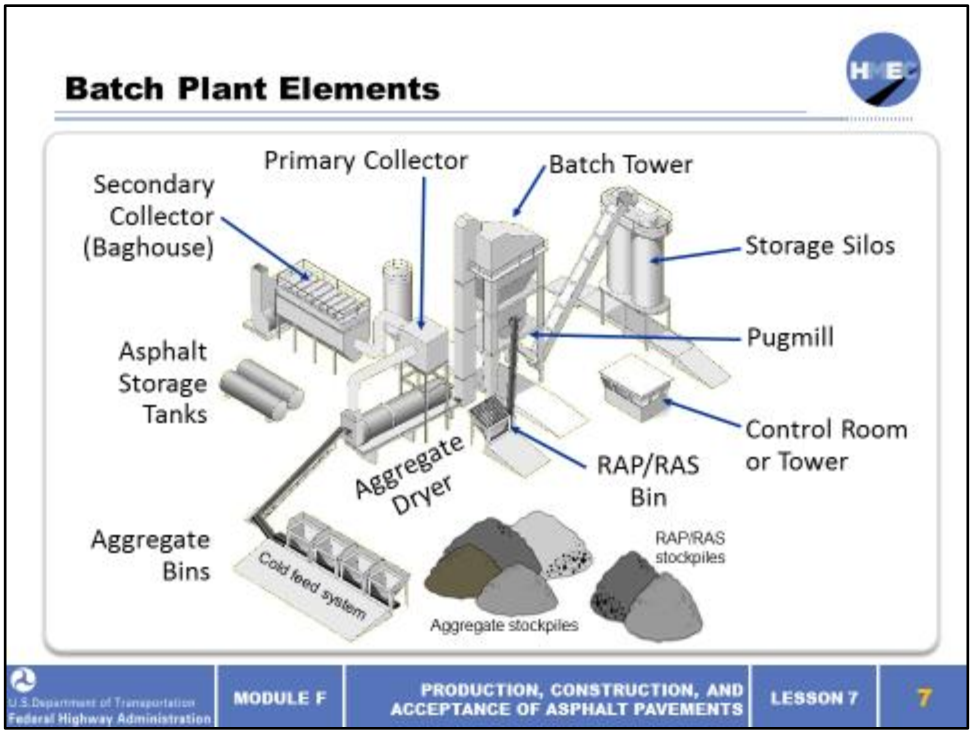
1. Asphalt concrete is a mixture of mineral aggregates, asphalt binder, and additives when required, that is designed to meet specific engineering properties. It is produced at elevated temperatures to properly dry the aggregates, coat them with asphalt binder, and provide a mixture that can be placed and compacted. Therefore, the primary functions of an asphalt plant are: (1) to accurately proportion the aggregates, asphalt, and additives, (2) to dry the aggregates, and heat the aggregates and asphalt binder to proper temperature for mixing, (3) to properly mix the aggregates, asphalt, and additives, (4) to store the mixture at an elevated temperature, and (5) to weigh and dispense the asphalt mixture into trucks for transport to the project site.

2. Both batch and drum mix plants accomplish the five functions of: (1) proportioning, (2) drying and heating, (3) mixing, (4) storing, and (5) dispensing of the finished product. The primary difference is in how the mixing is done. In batch plants, the mixing is done in 3- to 5-ton batches while continuous mix plants mix continuously while in operation.

3. The choice of the type of plant is based on the economics of the production operation. Batch plants are ideal for metropolitan areas where a large number of different mixes are required

daily. When larger quantities of fewer mixtures are required, continuous plants become the plant of choice.

Slide 7



See pages 67 to 80 under Part II, Section 8 of the *Hot-Mix Asphalt Paving Handbook*. In addition, see pages 409 to 414 of Chapter 6 of the NCAT textbook, *Hot Mix Asphalt Materials, Mixture Design and Construction*.

The following provides a brief explanation of the major components of a batch plant. The instructor should briefly identify each so that the participants understand the material flow and how all of the components tie together at the batch plant.

Aggregate Bins and Aggregate Stockpiles
Stockpiled aggregates are loaded into the aggregate bins for delivery into the aggregate dryer. Each bin holds a separate aggregate size or gradation. They have an adjustable gate that meters the aggregate onto a moving conveyor belt. The gate openings and the conveyor belt speed controls the amount of aggregate introduced into the plant, and that controls the mix gradation. These bins should be loaded consistently from the stockpiled aggregate by a loader operator. Some RAP is used in just about all asphalt mixes today and is added to the aggregate blend in the batch tower or aggregate dryer. When used, RAS are added to the mix at the same location as RAP.

Aggregate Dryer
The aggregate dryer dries the aggregate before they enter the batch tower.
Asphalt Binder Storage Tanks
Asphalt binder is stored in tanks while awaiting delivery to the pugmill for mechanical mixing with the aggregate. Asphalt cement in the tanks is heated between 300 °F and 350 °F depending on the grade and type of asphalt (remember this from Lesson 3).

Particulate Primary Collector
The primary collector is located between the dryer and secondary collector, and removes large dust particles from the exhaust gases before entering the more efficient secondary collector, the baghouse.

Baghouse (Particulate Secondary Collector)
The baghouse removes fine particulate matter from the dryer exhaust gases before they are released into the atmosphere.

Fines/Additive Silo
Some plants may have additional silos or equipment for storing mineral filler, fines from the baghouse, or special additives that are added to the aggregate blend in the pugmill or batch tower/hot bins. This plant component is not shown in the slide.

Batch Tower and Hot Bins
The batch tower contains the hot bins that separate the aggregates into specific sizes (sieves the aggregate from the dryer) and then blends or combines the aggregate sizes into the proper amounts before entering the pugmill. The next slide shows more details on the batch tower.

Pugmill
This is the part of the batch plant where the proper amount of asphalt is added to a specific batch or weight of aggregate and mixes the asphalt, aggregate blend, and any additive used in the asphalt mix. It is located at the bottom of the batch tower.

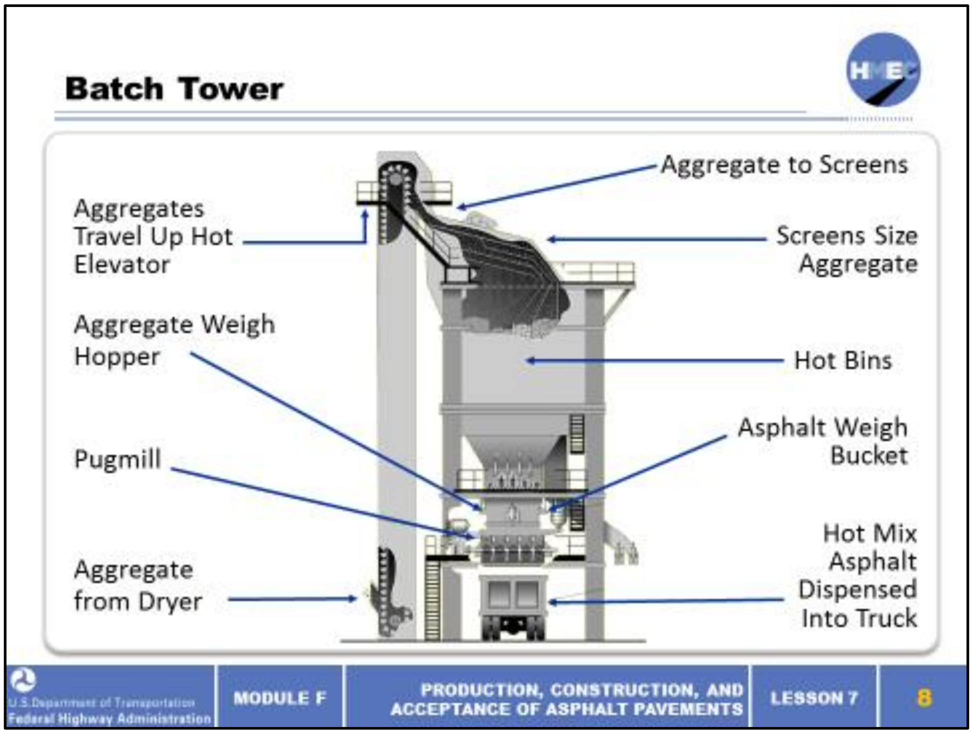
Storage Silos
Many batch plants do not have storage silos (sometimes referred to as mix silos). After the aggregate and asphalt binder are mixed in the pugmill, the resulting asphalt mix is discharged directly into the trucks or is diverted to silos. In general, silos (mix silos) are grouped into two types: surge and storage silos. Surge silos are usually insulated but unheated and are designed to hold HMA for short periods of time (several hours) between truck arrivals. Storage silos are well insulated, heated, near air-tight, and are designed to hold HMA for longer periods of time (overnight). For this lesson, the term storage silos is used as the general term for mix silos.

Truck Loading Area
Trucks are loaded directly under the batch tower or from the storage/surge silos. A weigh scale can be located in the loading area to ensure trucks are loaded with the correct amount of mix.

However, most current plants are automated so that a specific amount of asphalt mix per dump is released into the truck. These batches can be preprogrammed for specific sizes of trucks.

Control Center
The control center manages operations from a central location. Most modern asphalt plants are sophisticated facilities and are computer controlled.

Slide 8

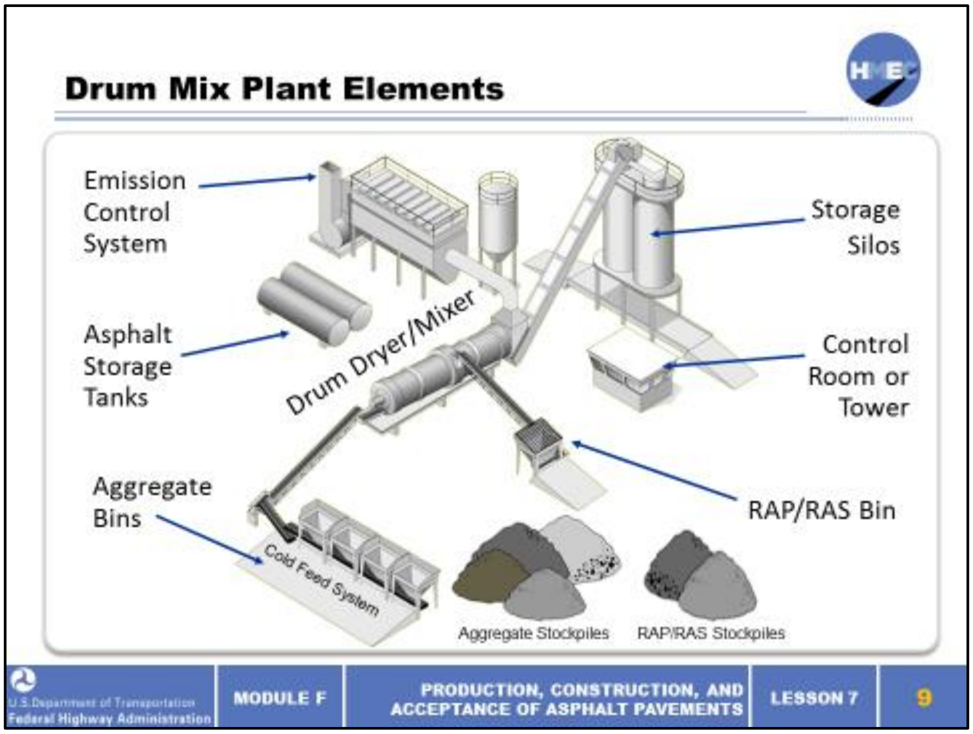


The primary difference between drum-mix production facilities and batch style production facilities is that the aggregate is sized, blended, dried, and mixed with the heated asphalt binder in a one-step continuous process, rather than the batch-at-a-time process common to batch plants. This is the primary reason for the batching tower.

A benefit of batch plants is that aggregates are essentially graded twice, rather than once, as is the case with drum-mix plants, which could lead to better gradation control if significant variability exists in stockpiled aggregates. However, this typically is not the case with modern-day production procedures that normally rely on four to six stockpiles for production.

The hot aggregate from the dryer travels up the hot elevator to the top of the batch tower. It then passes over a series of screens that divert the hot aggregates to bins of different sizes. These bins are called hot bins. Batches of different mixes are made by weighing the appropriate amount of aggregate from each of the hot bins in the aggregate weigh hopper. The required quantity of asphalt cement for the batch is weighed into the asphalt weigh bucket. The aggregate from the weigh hopper, the asphalt from the weigh bucket, and additive (if used) are dropped into a pugmill where they are mixed for a specified length of time. If RAP or RAS is added, it can be added cold to the weigh hopper or some plant configurations allow it to be added at the base of the hot elevator to allow more time for heat transfer from the aggregates to the RAP and RAS. Once mixed, the batch in the pugmill can be dropped directly into trucks for transport to the paving project, or diverted to a silo for final load-out.

Slide 9



The schematic in this slide presents the major components of a continuous-flow drum mix plant. A typical drum plant includes five major components: the aggregate cold-feed system, liquid asphalt supply system, a drum, an emission-control system, and silos.

Unlike a batch plant, a drum mix plant does not contain separate parts for drying and mixing. Drying and mixing happens in the same drum. Drum mix plants are also grouped into two major variations: parallel-flow drum and counter-flow drum. The differences between these types will be discussed later under this topic or section.

As noted earlier, batch plants are becoming largely obsolete and/or confined to urban areas.

See pages 67 to 80 under Part II, Sections 9 and 10 of the *Hot-Mix Asphalt Paving Handbook*. In addition, see pages 398 to 409 of Chapter 6 of the NCAT textbook, *Hot Mix Asphalt Materials, Mixture Design and Construction*.

The following provides a brief explanation of the major components of a drum mix plant. The instructor should briefly identify each so the participants understand the material flow and how all components tie together at the drum mix plant.

The flow of material through a drum-mix plant is as follows: Aggregates are metered from cold feed bins to a collector belt that typically incorporates a scalping screen prior to a belt scale (weigh bridge). The aggregates are deposited in the drum drying and heating while asphalt is metered into the drum based on the aggregate mass determined from the belt scale. Mixing occurs in the end of the drum, and mixture is then typically elevated to storage silo(s).

Aggregate Bins and Aggregate Stockpiles

Stockpiled aggregates are loaded into the aggregate bins for delivery into the drum. The cold feed bins should be loaded consistently from the stockpiled aggregate by a loader operator. Regarding aggregate storage and delivery into drum mix plants, there are two major points that have a significant impact on the variability of the mix: variability of the stockpiles and accurately accounting for the moisture content of the aggregates in the stockpiles, which are discussed below.

• It is critically important for multiple cold feed bins to be used in drum mix plants for good gradation control, assuming the gradation of each stockpile is properly controlled in building those stockpiles. Each cold feed bin has an adjustable gate that meters the aggregate onto a moving conveyor belt. The gate openings and the conveyor belt speed controls the amount of aggregate introduced into the drum, and that controls the mix gradation. If the gradation of the stockpiles changes significantly over time or if the loader operator segregates the aggregate in transporting the aggregate from the stockpiles to the cold feed bins, that segregated aggregate, or increased variation in gradation will make its way through the plant and be dumped into the trucks. Thus, it is important that cross contamination between the cold feed bins does not occur and that each cold feed bin be loaded consistently.

• The other major issue with the aggregate in a drum mix plant is the moisture content of the aggregates. The aggregates are weighed on a belt scale located between the cold feed bins and drum (not shown in the slide). This weight includes the aggregate and water in and around the aggregate. It is critically important that the water content of the blended aggregate or stockpiles be measured periodically so that the amount of water in the aggregate can be properly accounted for because the amount of asphalt added in the drum is based on the weight of the dry aggregate. Any deviation in the water content in the aggregate will result in additional variability of the asphalt content in the final mix.

Some RAP is used in just about all asphalt mixes today and is added to the aggregate blend in the drum collar. RAP is generally loaded into its own cold feed bin then moved by conveyor belt to be discharged directly into the drum, where it is heated by the already-hot aggregate. When used, RAS is added to the mix at the same location as RAP.

Asphalt Binder Storage Tanks

Asphalt binder is stored in tanks while awaiting delivery to the pugmill for mechanical mixing with the aggregate. Asphalt binder in the tanks is heated between 300 °F and 350 °F depending on the grade and type of asphalt (remember from Lesson 3).

Drum
The rotating drum first heats the aggregate then mixes the hot aggregate with asphalt cement and other material additives if used. There are two basic types of drum mixers: (1) parallel flow (where the aggregate enters the drum at the same end as the burner and travels parallel to the hot air steam) and (2) counter flow (where the aggregate enters the drum at the opposite end from the burner and travels counter to the hot air stream). Asphalt is added to, and mixed with, the hot aggregate in the drum at different locations depending upon the design of the drum mixer.

Emission Control System
The emission control system or baghouse removes fine particulate matter from the dryer exhaust gases before they are released into the atmosphere. In some cases, the baghouse fines can be returned to the mix in the drum as the asphalt is added to the aggregate blend so they are captured in the binder.

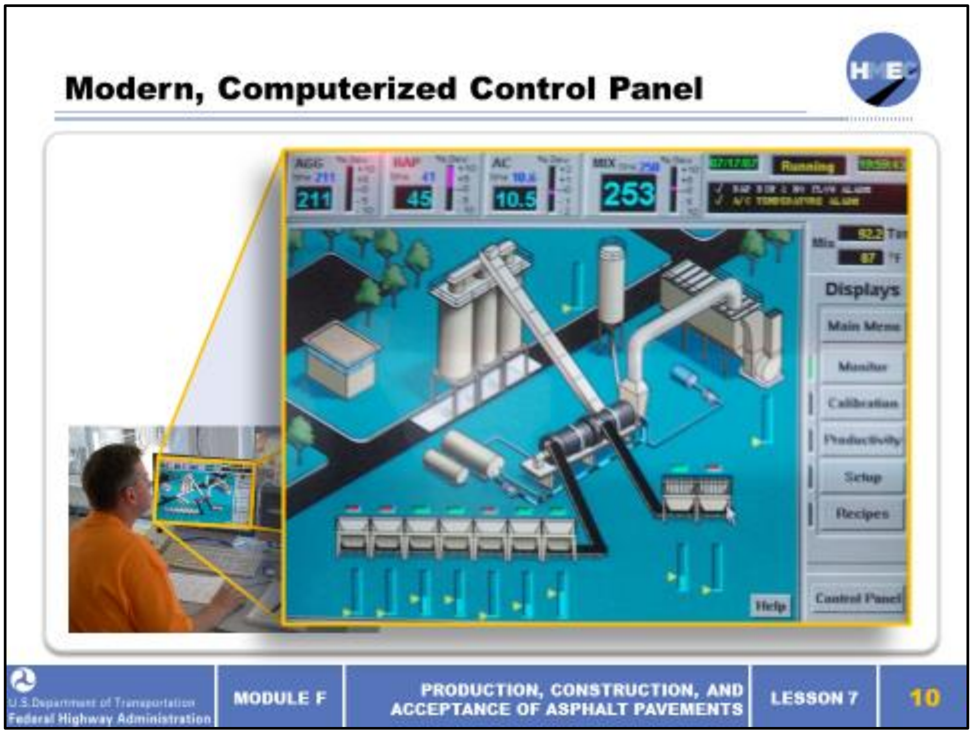
Fines/Additive Silo
As for batch plants, some drum mix plants have additional silos or equipment for storing mineral filler, fines from the baghouse, or special additives that are added to the aggregate blend in the drum at different locations. Some aggregate additives are added to the aggregate blend on the conveyor belt leading into the drum from the cold feed binds. This plant component is not shown in the slide.

Storage Silos
All drum mix plants have silos. After the aggregate and asphalt binder are mixed in the drum, the resulting asphalt mix is discharged via a conveyor into an appropriate storage or surge silo. As discussed under batch plants, silos are generally grouped into two types: surge and storage silos. Surge silos are usually insulated but unheated and are designed to hold HMA for short periods of time (several hours) between truck arrivals. Storage silos are well insulated, heated, near air-tight and are designed to hold HMA for longer periods of time (over night or for longer periods of time).

Truck Loading Area
Trucks are loaded directly under the silos. Drum mix plants can have multiple storage silos because it is much easier for a drum mix plant to produce different mixes which can be diverted into different storage silos. A weigh scale can be located in the loading area to ensure trucks are loaded with the correct amount of mix. However, most current plants are automated so that a specific amount of asphalt mix per dump is released into the truck. These dumps can be preprogrammed for specific sizes of trucks.

Control Room or Tower
The control room or tower (sometimes referred to as the control center) manages operations from a central location. Most modern asphalt plants are sophisticated facilities and are computer controlled.

Slide 10



For drum mix plants, the moisture content of the aggregates are key to getting the right amount of asphalt added to the aggregate blend in the drum. Moisture compensation is done in the plant control computer once the moisture of the aggregates is known. Aggregate moisture content can vary throughout a given day, so it is important that samples be taken for measuring the moisture content. The key is getting the quality control (QC) technician to notify the plant operator of changes in the moisture content when they are detected.

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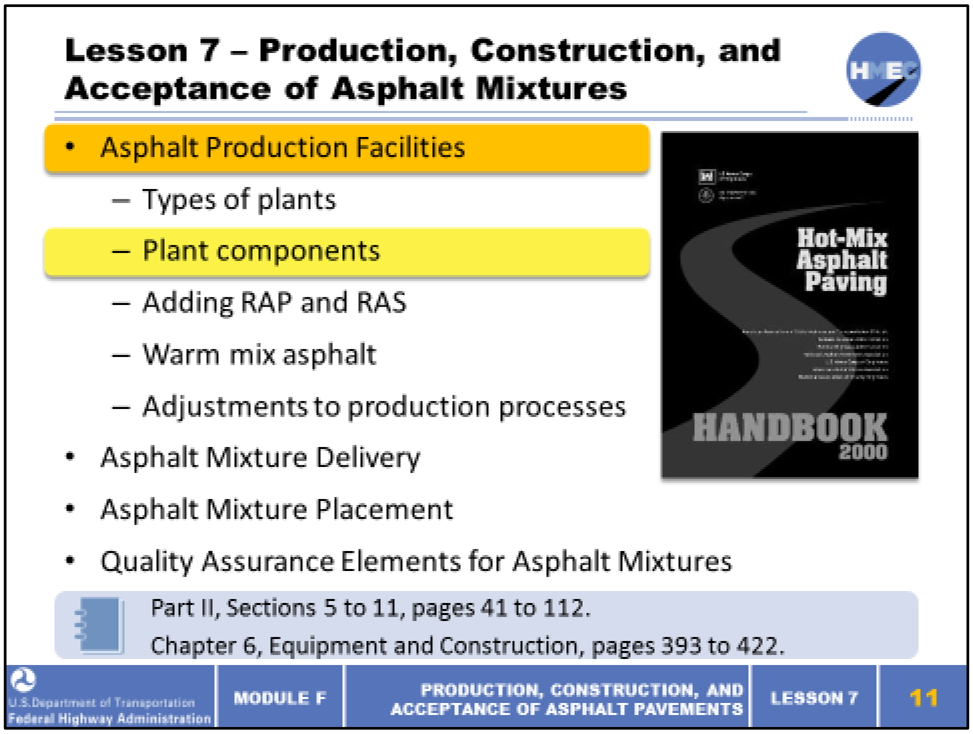
Slide 11



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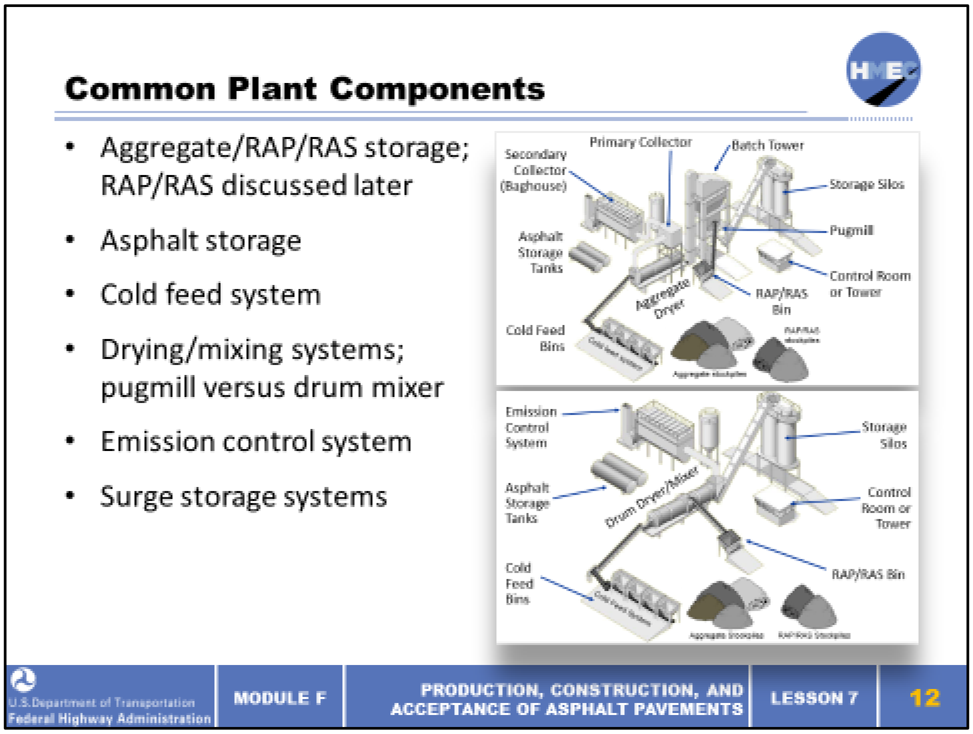
Slide 12



Each component/operation is described in some detail in the subsequent slides.

Slide 13


Asphalt Delivery and Storage
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PRODUCTION, CONSTRUCTION, AND ACCEPTANCE OF ASPHALT PAVEMENTS
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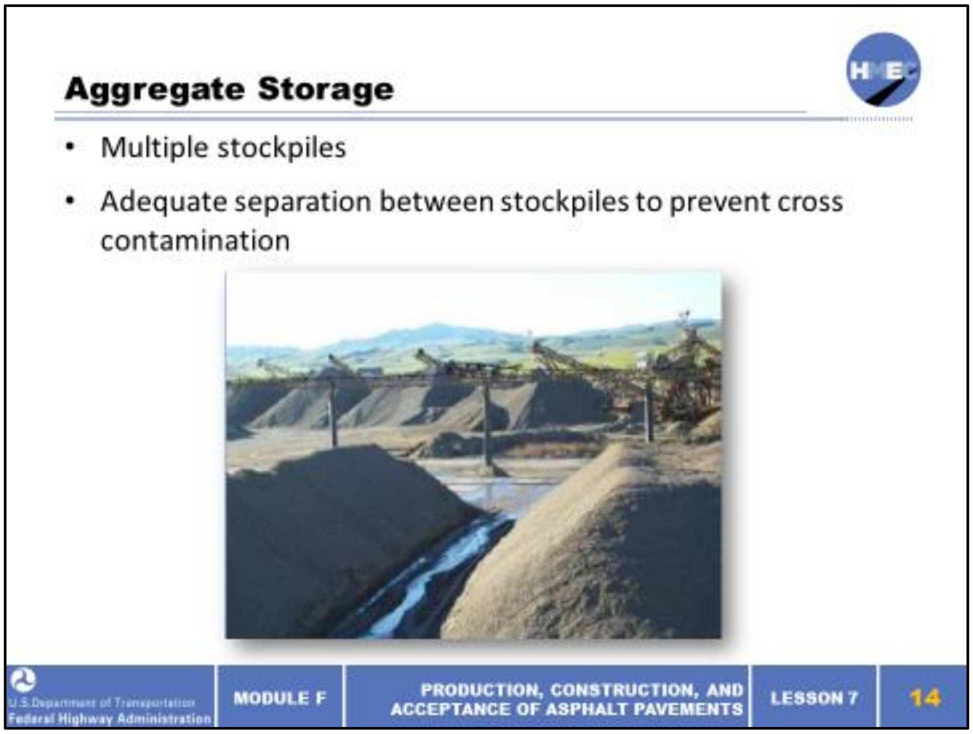
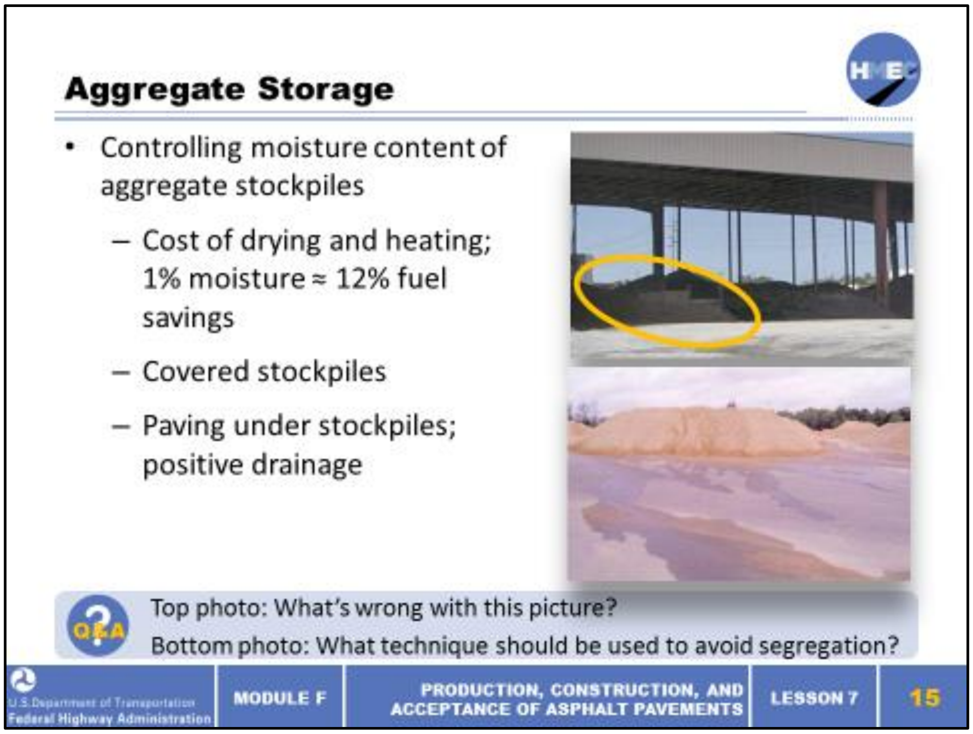
Slide 14



The quality and the consistency of the aggregate should not be significantly altered during the transport and delivery of the aggregate from its manufacturing location (pit or quarry) to the asphalt mix plant or during the production, placement, and compaction of the asphalt mix.

To ensure the quality of aggregates in an asphalt mix, good stockpiling and material feeding practices must be utilized, including those for RAP and RAS stockpiling operations. In the stockpiling process, key items of concern are adequate stockpile separation, cross contamination between the cold feed bins, controlling moisture in the stockpiled material, controlling dust in the plant area potentially generated by stockpiling operations, and minimizing segregation in removing the aggregates from the stockpile by the loader operator. These operations are especially important for drum mix plants because what goes in the drum, comes out of the drum. Remember, in a batch plant, the aggregate blend from the stockpiles are resized through the batch tower and hot bins.

Slide 15



Stockpile covering can consist of placing moisture-proof plastic materials directly on the piles and providing an open walled, roofed structure. When utilizing open-walled buildings, consideration should be given to airflow through the structure and the direction of wind during rain or snow storms.

Paving under stockpiles allows for drainage to take place under the stockpile. A cross-slope should always be placed on the hot-mix asphalt drainage pad to divert water away from the stockpile. The use of paved pads under stockpiles also reduces the waste or material losses associated with placing stockpiles in yards with gravel operating areas.

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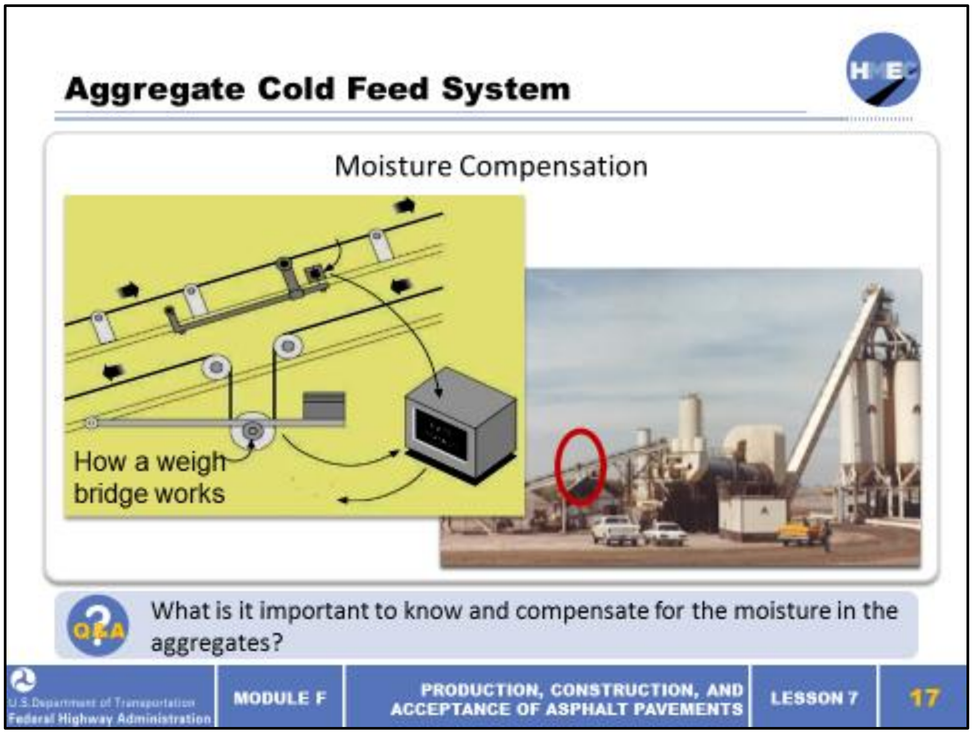
Slide 16

Ensuring that each cold feed bin contains the appropriate virgin aggregate sizes and are not contaminated by aggregates from different stockpiles or cross-contamination between the cold feed bins will significantly reduce variability during the production process and ultimately in the finished HMA mat.

Slide 17


Aggregate Cold Feed System
Moisture Compensation
How a weigh bridge works
What is it important to know and compensate for the moisture in the aggregates?
U.S. Department of Transportation
Federal Highway Administration
MODULE F
PRODUCTION, CONSTRUCTION, AND ACCEPTANCE OF ASPHALT PAVEMENTS
LESSON 7
17

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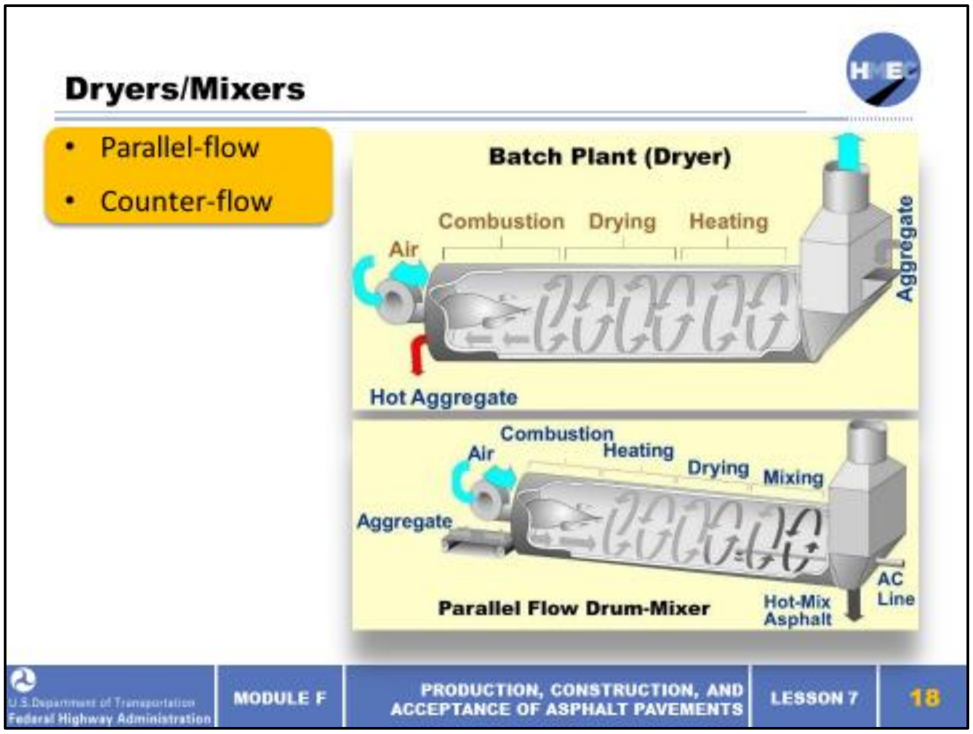
Slide 18



The two common types of dryers utilized in hot-mix asphalt production are:

• Parallel-flow; and
• Counter-flow.

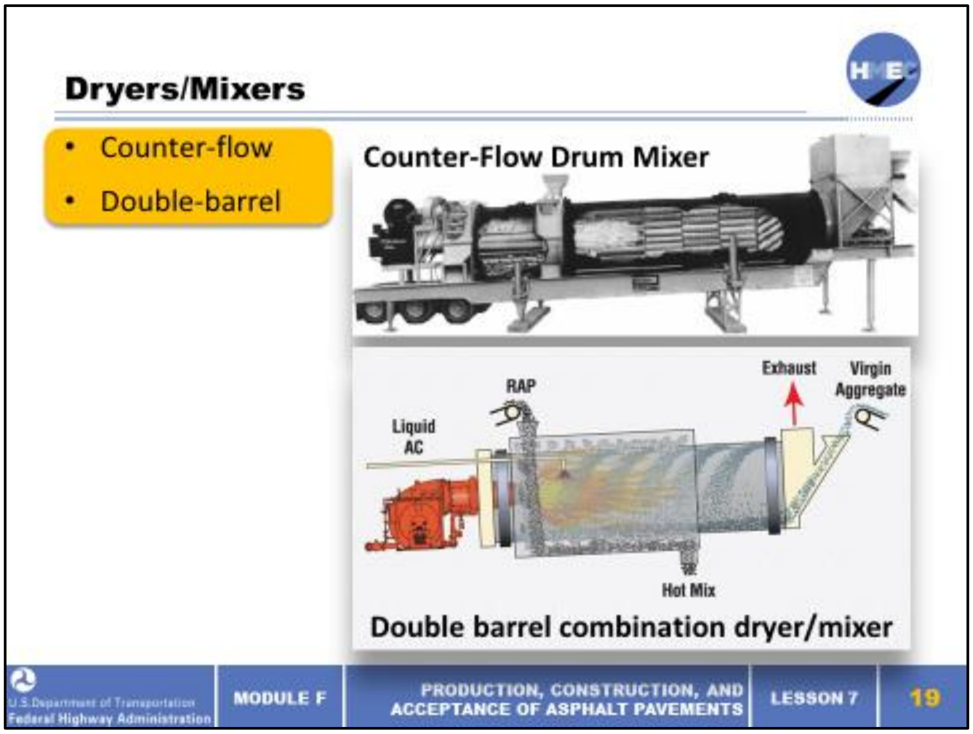
The parallel-flow and counter-flow dryers are shown on this slide, which are used with both the batch and drum mix plants. In the slide, a typical parallel flow drier is shown in the bottom illustration as part of a drum mix or continuous flow plant, while the counter flow drier is shown in the upper illustration in the slide for a typical component of the batch plant.

Parallel-flow dryers are more common to drum mix plants produced during the period from the early 1970s to the late 1980s. In parallel-flow dryers, the direction of the flow of the aggregate is parallel to the flow of the hot gases used to heat and dry the aggregate—hence the name parallel-flow drums.

The combustion, heating, drying, and mixing zones of the parallel flow dryers when used as drum mixers are shown in the schematic. For parallel-flow mixers, the asphalt is introduced in the drum near the end of the drum-dryer-mixer where the gas temperature is relatively low. There are two versions of the parallel flow drum mix plants: the typical parallel flow drum mixer where the asphalt is sprayed inside the drier-mixing drum (illustrated in the slide), and a modified version where the asphalt is added to the aggregate in a separate chamber called the mixing chamber (not illustrated in the slide). This modified version of the parallel flow drum mixer is termed the "coater type of drum mix plant." There are relatively few of these coater type drum mix plants in operation in the U.S. today.

Counter-flow dryers are commonly used with batch plants to dry the aggregate. As shown in the slide, aggregate enters the dryer at the opposite end from the burner and flows in the opposite direction to the exhaust gases—hence the name counter-flow drum driers. The combustion, drying, and heating stages or areas in a counter-flow dryer are shown in the schematic in the slide.

Slide 19

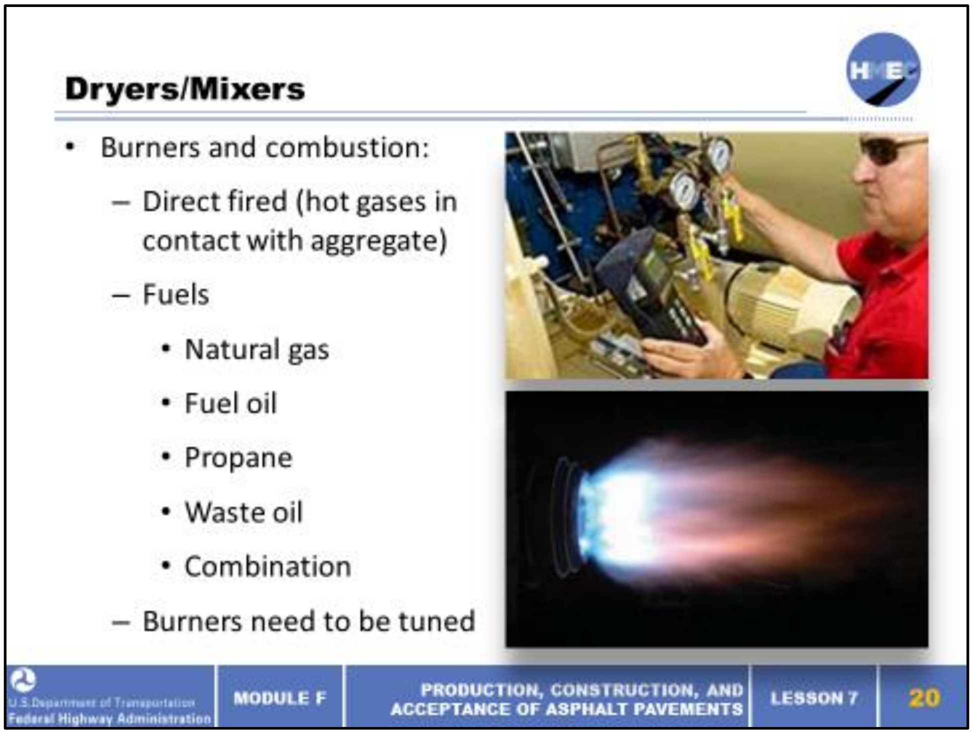


Counter-flow dryers/mixers have been used in drum mix operations since the late 1980s. The aggregate enters the low gas temperature end of the drum and flows toward the combustion end of the dryer. Since the burner position is located relatively far inside of the dryer, there is a sufficient length of the drum remaining for the introduction of the asphalt and the mixing of the asphalt with the heated aggregate. The drying zone of the counter-flow drum mixer was shown in the slide previous slide. The combustion zone of the dryer is the location of the burning of the fuel and oxygen, and the temperatures are relatively high. In the heating zone, the aggregate is raised to approximately the boiling point of water (212 °F), and in the drying zone the moisture is vaporized and the aggregate temperatures are raised.

A variety of other types of counter-flow drum mixer or continuous plants are utilized to produce HMA. Some of these continuous plants mix the asphalt and aggregate external to the counter-flow dryer in a continuous pugmill. Other plants are configured to dry in the internal area of the counter-flow drum and mix the asphalt and aggregate in the external collar of the drum.

Counter flow dryer section of the Double Barrel® dries all the aggregate. The mixing unit is folded back around the aggregate dryer portion of the drum. A special diverter gate routes the aggregate, leaving the dryer section to be mixed with the asphalt. The asphalt is introduced with the aggregate outside of the drying portion of the drum. The mixing of the asphalt and heated aggregate is completed outside the exhaust gas stream and is behind or underneath the burner.

Slide 20

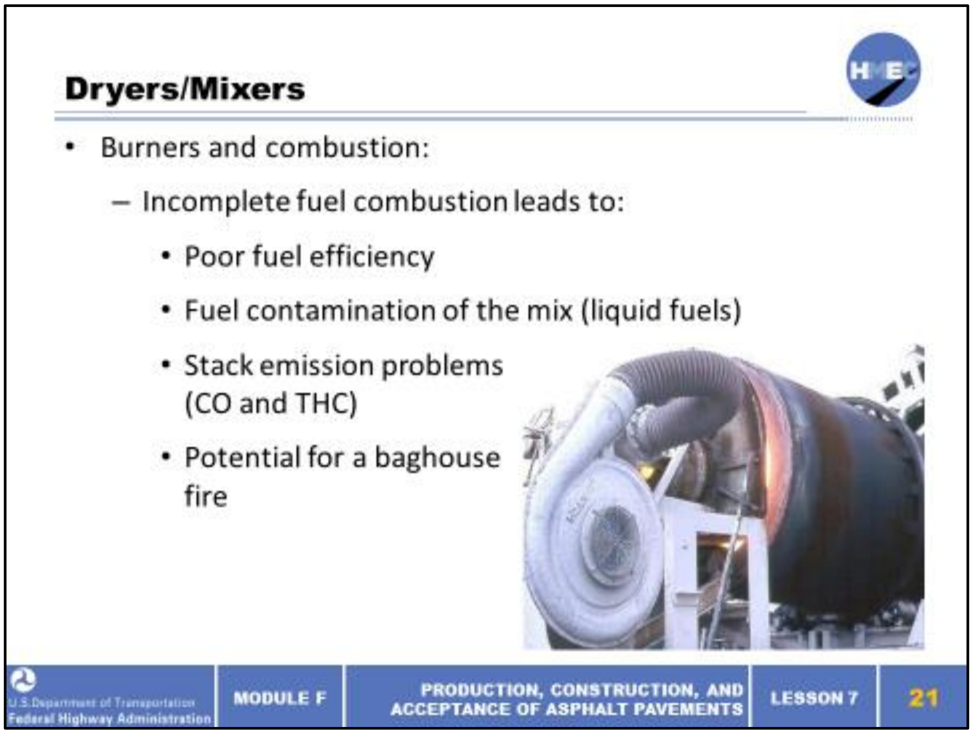


The burners used in asphalt mix production plants use a combination of forced air burners and induced draft burners: about 30% is forced air and about 70% is through induced air.

Drum dryer burners typically use natural gas, fuel oil, propane, waste oil, or some combination of these fuels. A few burners have used pulverized coal or wood, but these are very uncommon. It is not unusual for burners to be fitted with a dual-fuel manifold to allow more than one fuel to be used with the burner.

As the price and availability of fuels change, the plants can easily change their burner operations. The use of some waste oil can result in incomplete combustion of the fuel and contaminate the aggregate surface if the waste oil is not properly preheated.

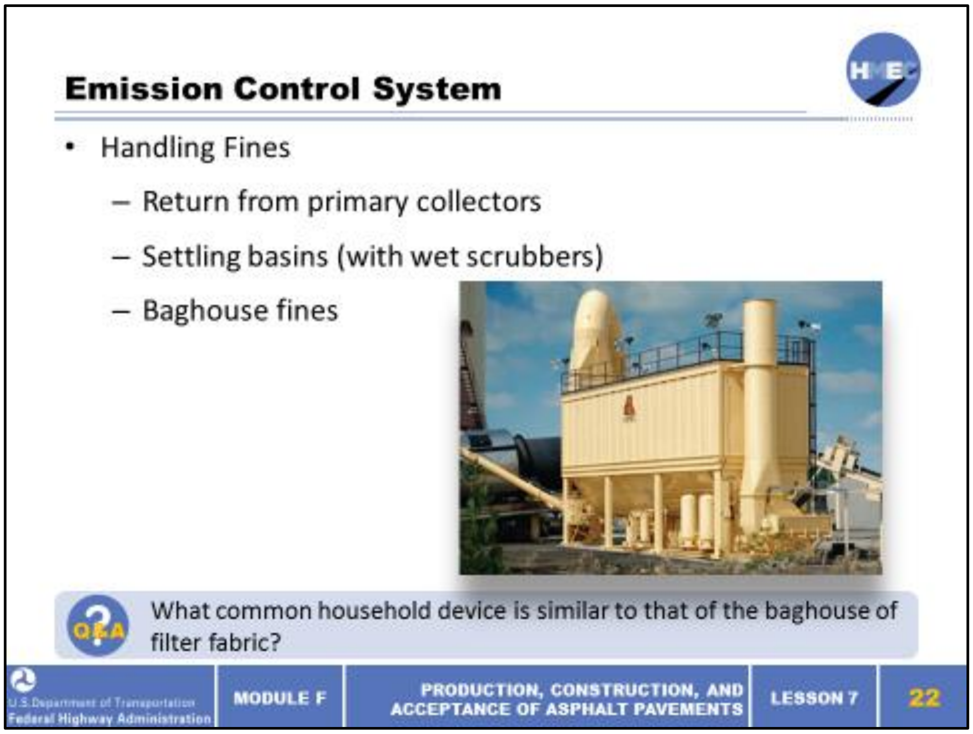
Slide 21



The purpose of the burner inside the drum mixer is to provide the heat necessary to heat and dry the aggregates used in the final mix. Burner tuning is done periodically to maximize the combustion efficiency of burners. An improperly tuned or untuned burner can cause problems including poor fuel efficiency, incomplete fuel combustion, additional cost, lower production, coating of hydrocarbons in aggregates and mix, excessive smoke and emissions, poor performance of baghouses, and potential fire.

Some producers have reported difficulties adjusting burners to operate efficiently at WMA production temperatures. This has been particularly problematic at the low production rates used in many of the early WMA trials. Improper burner adjustment will result in incomplete combustion. The unburnt fuel will contaminate the mixture and the emissions from the plant will be higher.

Slide 22



See pages 105 to 114 under Part II, Section 12 of the *Hot-Mix Asphalt Paving Handbook*. In addition, see pages 414 to 422 of Chapter 6 of the NCAT textbook, *Hot Mix Asphalt Materials, Mixture Design and Construction*.

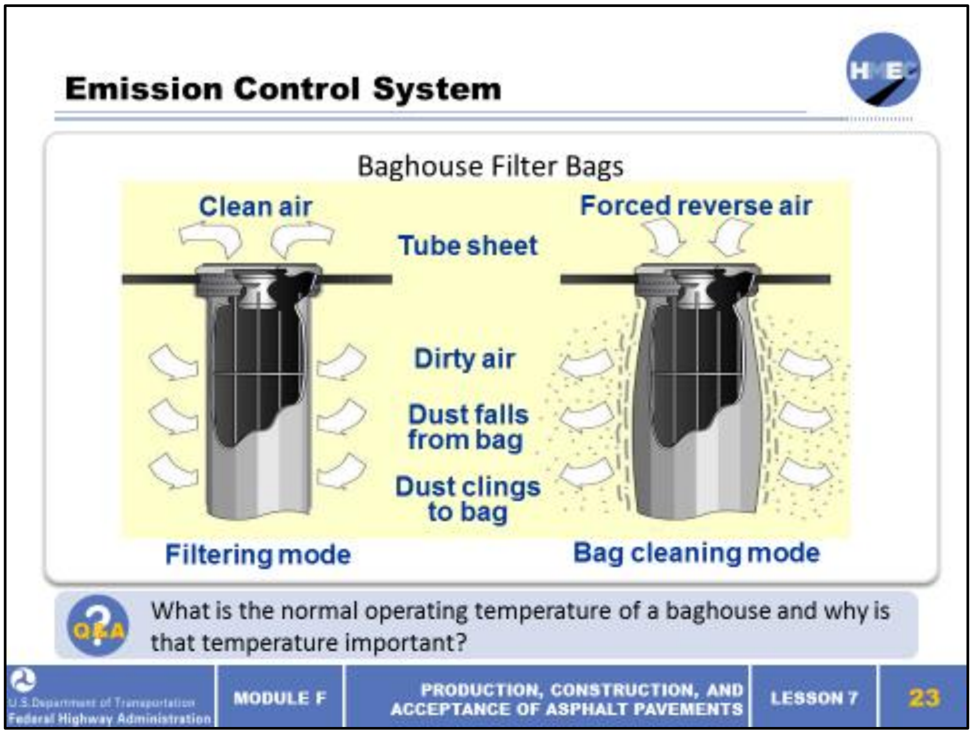
Batch type plants can have both a primary and secondary collection system to collect the fines (collecting the larger and finer dust particles), while most drum mix plants generally utilize one of two systems: wet collectors or scrubbers and baghouse or fabric filters.

The primary collectors used in a batch plant are a knockout box and centrifugal collector. The knockout box has an expansion chamber where the velocity of the heavier particles are reduced, allowing them to drop out of the exhaust gases. The centrifugal collector forces the larger or heavier dust particles to the outside wall, which then drop to the bottom as their velocity decreases. The larger dust particles collected by both systems can be wasted or returned to the HMA mix. The secondary collector used in most batch plants is a baghouse or fabric filters, which are discussed below.

The wet collectors or scrubbers operate by using water to increase the weight of the dust particles so they are not released into the atmosphere. Settling ponds are used with this method or system to separate the dust from the water with time. If mineral filler is being added to the HMA mix, wet collectors or scrubbers are usually not used because most of the mineral filler can be lost and not returned to the HMA mix.

The baghouse or fabric filers are very efficient and can remove almost 99% of the dust from the exhaust gases. The system is simple in that the dust in the exhaust gas are pulled through a filter fabric that traps the dust on one side. As the dust builds up, the efficiency of the baghouse increases. The dust can then be collected and returned to the HMA mix or wasted. The next few slides provide additional discussion on the baghouse emission control system.

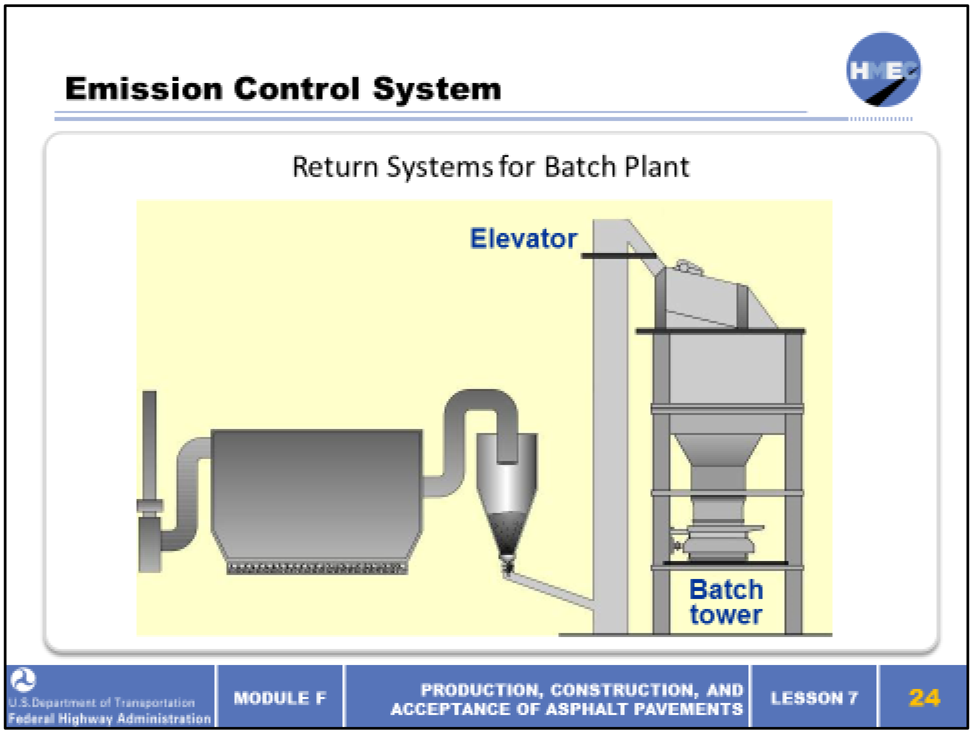
Slide 23



As the dust cake builds up, more fine material is collected in the cake. In order to maintain efficiency of the baghouse, the cake must be periodically removed on each bag.

The most efficient cycle time for removing the cake for gas flow efficiency versus filtering capability (smaller particle sized) will vary depending on the dust loading in the exhaust gas and the size and shape of the particulate matter.

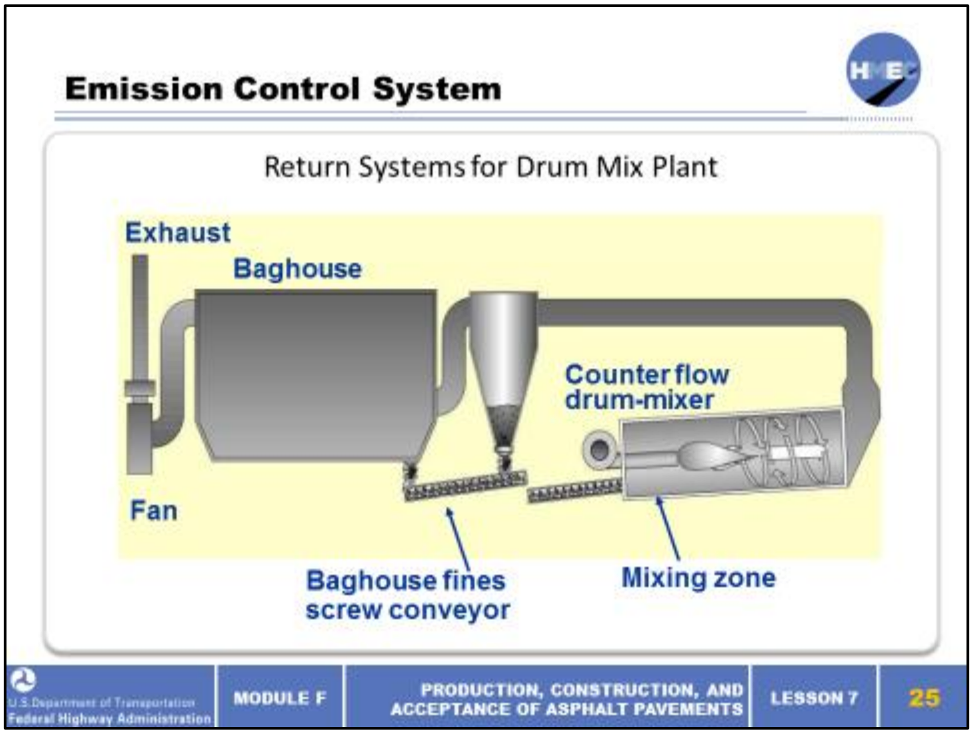
Slide 24



Typically a screw conveyor is located below the primary collector and the fines are conveyed to the boot of the hot elevator. The hot elevator transports all of the dried aggregate size fractions to the hot bucket and hot screens.

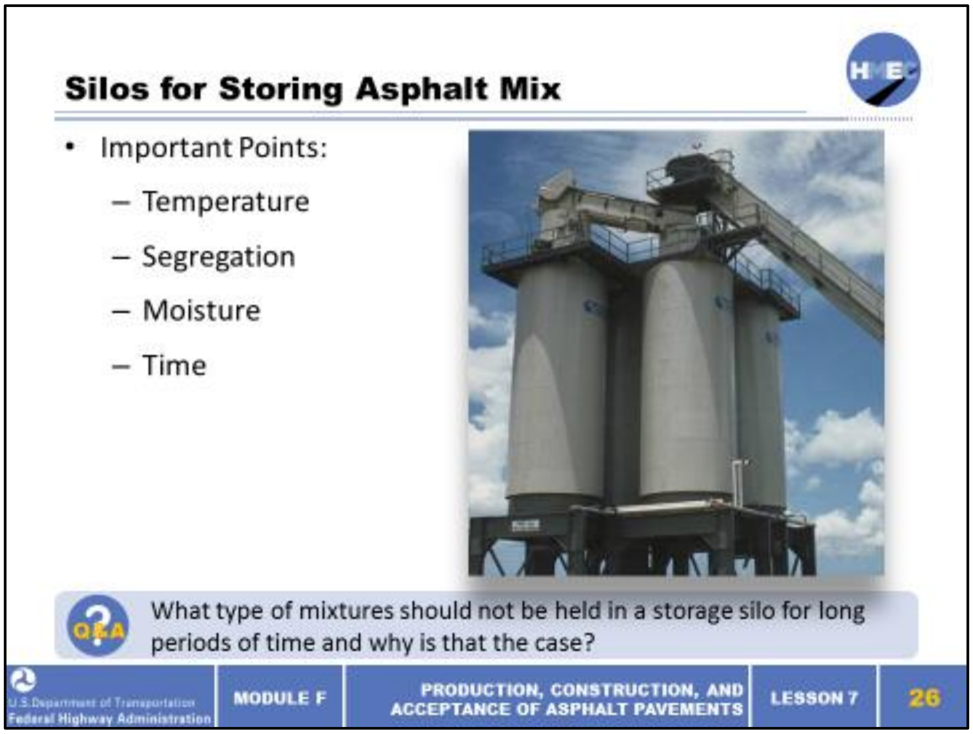
A gravity valve or a rotary valve is usually installed below the primary collector to prevent air from being pulled back up through the primary collector by the exhaust fan and thereby hindering the fines' flow.

Slide 25



This slide shows the screw conveyer in a drum plant.

Slide 26



See pages 96 to 104 under Part II, Section 11 of the *Hot-Mix Asphalt Paving Handbook*.

Because HMA is stored in silos at elevated temperatures, the asphalt in the HMA has the potential to oxidize while in storage. The oxidation reaction requires exposure to oxygen. Oxygen exposure can be reduced with the maintenance of seals on silo discharge gates.

There are two types of silos: surge and storage. Surge silos are usually insulated but unheated because they are designed to hold the HMA for short periods of time, between truck arrivals.

Conversely, storage silos are well insulated, heated, and nearly air-tight because they are designed to hold HMA for long periods of time (up to several days). Coarse-graded mixes (friction courses and SMA) should not be stored for longer periods of time because the asphalt may drain and collect at the bottom of the silo.

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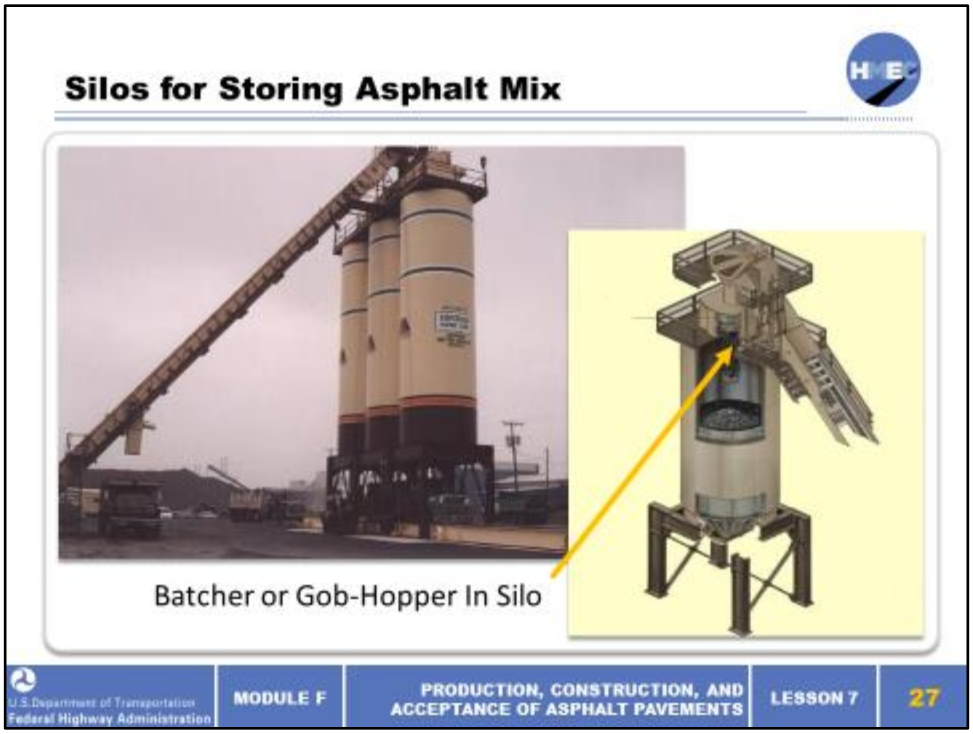
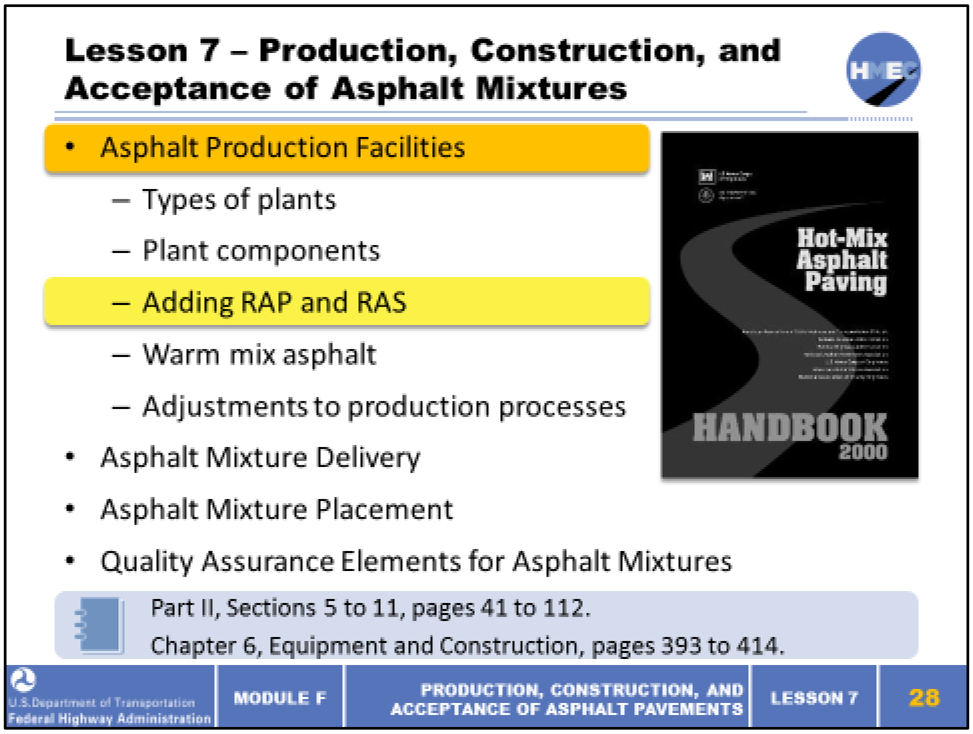
Slide 27



HMA is elevated from the drum output to a silo with slat conveyor. With multiple silos, the conveyor may physically move to fill the desired silo, or a system of chutes and gates may be used to divert mix to the desired silo if only two silos are used.

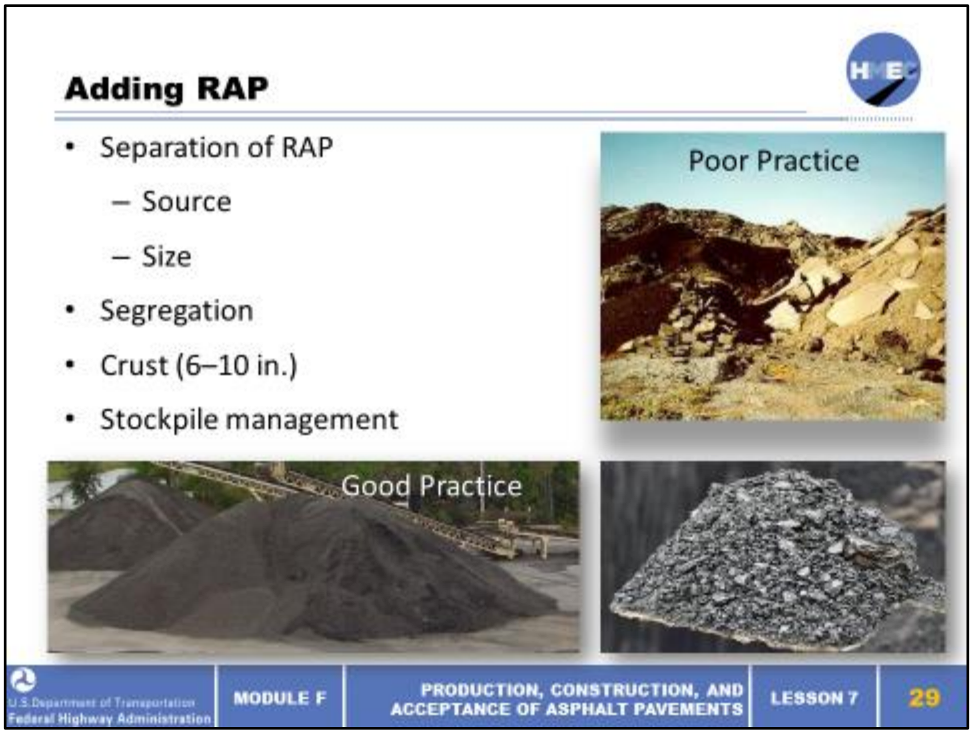
Many silos are equipped with an alarm that notifies the plant operator that the silo is too full or becoming empty. The plant operator should not overfill or under-fill the silo because the batcher will not operate as designed to prevent or minimize the possibility of segregation.

Slide 28



Most of the information covered within this section pertains to RAP in the production process because it is commonly used in asphalt mixtures—both HMA and WMA. More importantly, some agencies are allowing the use of higher percentages of RAP, even in the wearing course. RAS is covered briefly at the end of this section.

Slide 29



RAP is a very valuable resource; it contains both aggregate and liquid asphalt. When RAP is used in HMA and other mixes, it replaces both of these valuable resources, saving money and materials. How much RAP can be used in an HMA mixture depends on the following factors:

• Specification requirements, which vary with layer and road type;
• RAP properties, including:
  - Asphalt content;
  - Asphalt properties;
  - Aggregate gradation; and
  - Aggregate properties.
• RAP uniformity; and
• Plant type and equipment.

To maximize the percentage of RAP, it is important that the material be uniform. Proper processing and stockpiling will help, but it may be necessary to fractionate the RAP into more than one size.

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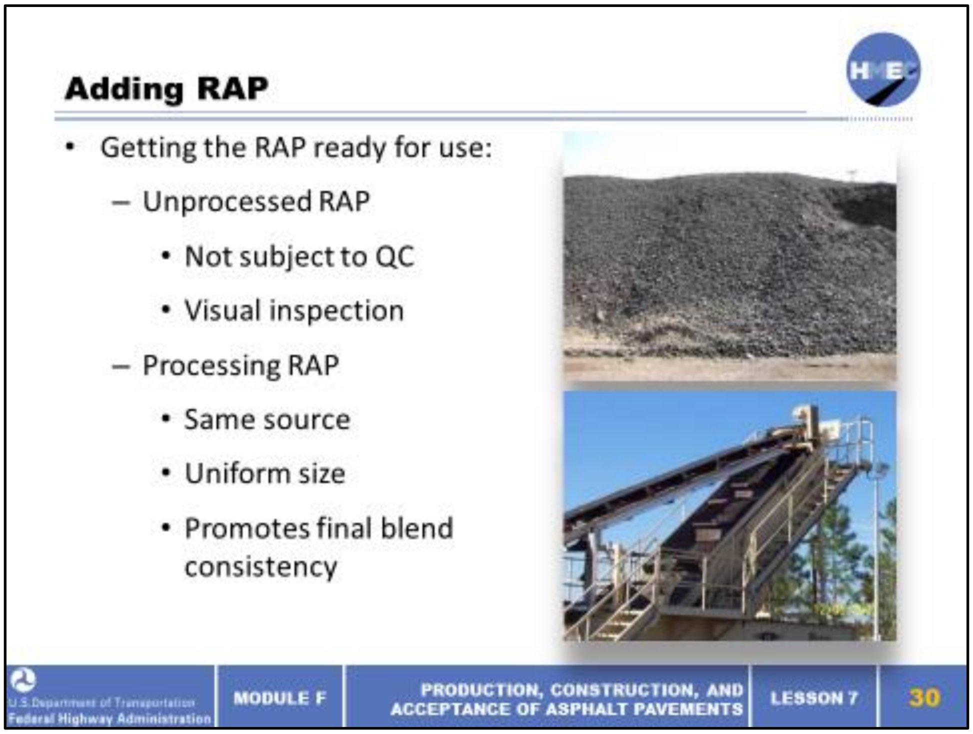
Slide 30



The key to producing a homogenous RAP product from a "composite" pile is to first blend the RAP thoroughly with a front-end loader or bulldozer and then to down-size the top stone size in the RAP in the crushing operation to one smaller than the top size in the hot mix being produced (e.g., ⅝ in. for a ¾ in. top-size mix). This ensures that the asphalt-aggregate bond is broken as much as possible and no oversize stone appears in the mix.

The actual crushing and testing of a given tonnage will generally prove this situation to be true. This crushing efficiency is important for those interested in conserving landfill space and increasing the percentages of full-depth RAP materials that come from several sources.

Slide 31

When large quantities of RAP from different sources are available, it is advisable to keep stockpiles separated and identified by source— good stockpile management program. This is especially true for high-volume surfacing projects or commercial airfield surfaces that are more likely to contain a high percentage of low-polishing, crushed aggregates and polymer-modified asphalt. A premium was paid for these materials when they were first used, and their value in skid resistance and binding capability should not have diminished.

On the other hand, materials taken from lower pavement layers, or low-volume roads, retain their value as well. These pavements, however, are not likely to have polymer-modified binders. They may contain aggregates with a higher polish value and may be more rounded. As such, it is important that the aggregate properties of the RAP be evaluated. The tests to be used are those commonly used by the individual agency for mixture design. NCHRP Report 452, however, provides the recommended use of RAP in the Superpave Mix Design method and the test standards or procedures to be followed. The use of RAP was discussed under Lesson 5 from a design standpoint.

The other important component for using RAP is the management of the RAP stockpiles. The NCAT report, "Best Practices for RAP Management," is a document that can be followed for improving on the performance on the use of RAP materials.

Slide 32

RAP should never be covered with a tarp or plastic in humid climates because of condensation that will increase the water content in these materials. Covering the RAP in humid areas is logical, but it must be done through a system that will prevent condensation. While increases in moisture below tarps is less than allowing precipitation to fall directly on the RAP stockpile, it still has a detrimental effect on the moisture content in the stockpile and should be avoided.

For this reason, most RAP stockpiles are either left uncovered, or RAP is stored under the roof of an open-sided building. In such a structure, free air can pass over the RAP, yet the RAP is protected from precipitation. Such structures are relatively economical and, with enough RAP production, can be justified based on reduced fuel consumption and the possible savings from using higher RAP percentages in the facility.

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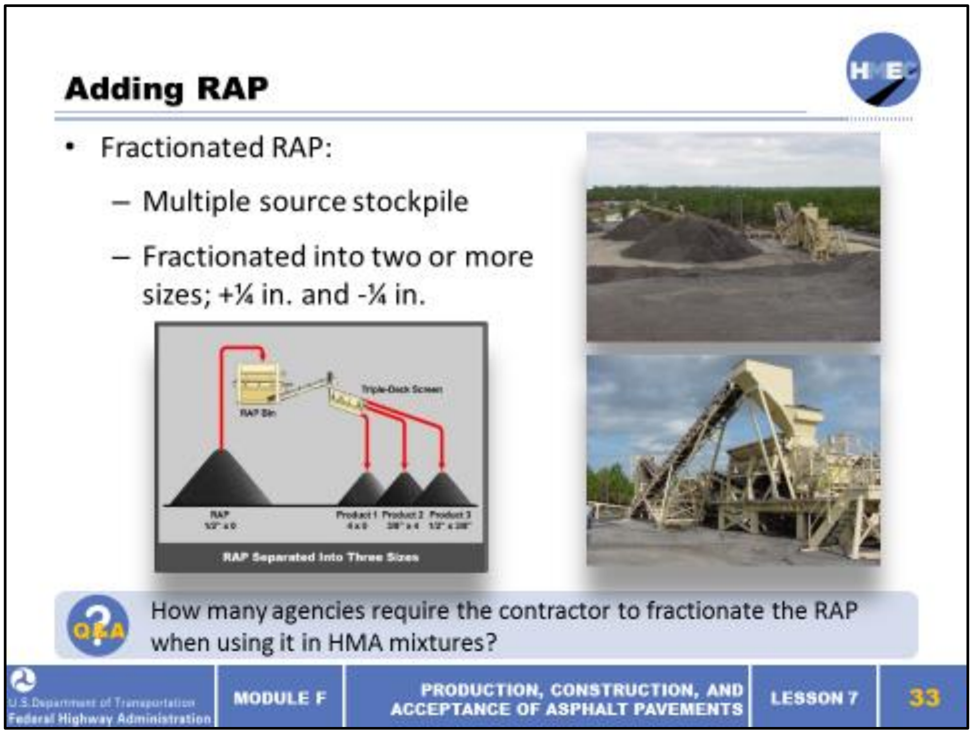
Slide 33



Whether the recycled materials are all from the same project or different projects, constructing separate coarse and fine RAP stockpiles will minimize segregation of RAP particles and allow greater flexibility in adjusting RAP content for the final aggregate gradation. The fine RAP material will have a greater asphalt content than coarse RAP stockpiles due to the higher surface area of fine material.

The asphalt binder content in both the fine and coarse RAP stockpiles can be expected to be more uniform than the asphalt content of a single RAP stockpile. It may be possible to introduce more of the fine RAP in mixtures having a small nominal maximum aggregate size, such as surface or leveling mixtures. Conversely, in large stone mixes, a larger portion of the coarse RAP can be used to help meet the gradation requirements.

- 4 is #4 since -4.75 mm; and
- 4 x 0 is #4 and smaller or (< #4).

Any oversize particles larger than ⅝ in. are discarded through a scalping screen.

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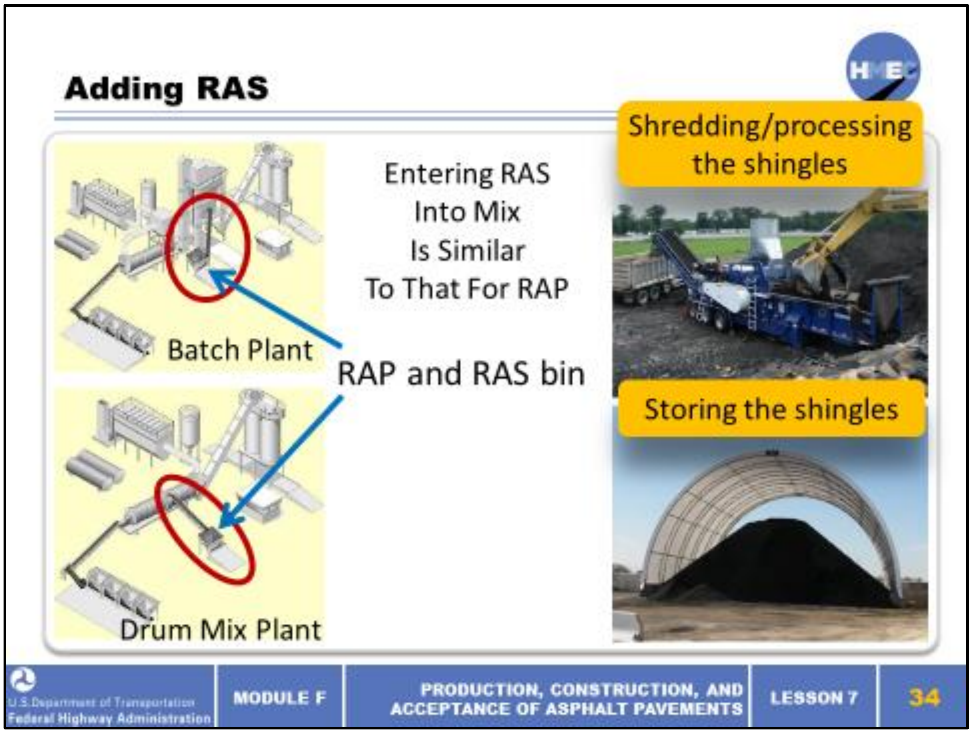
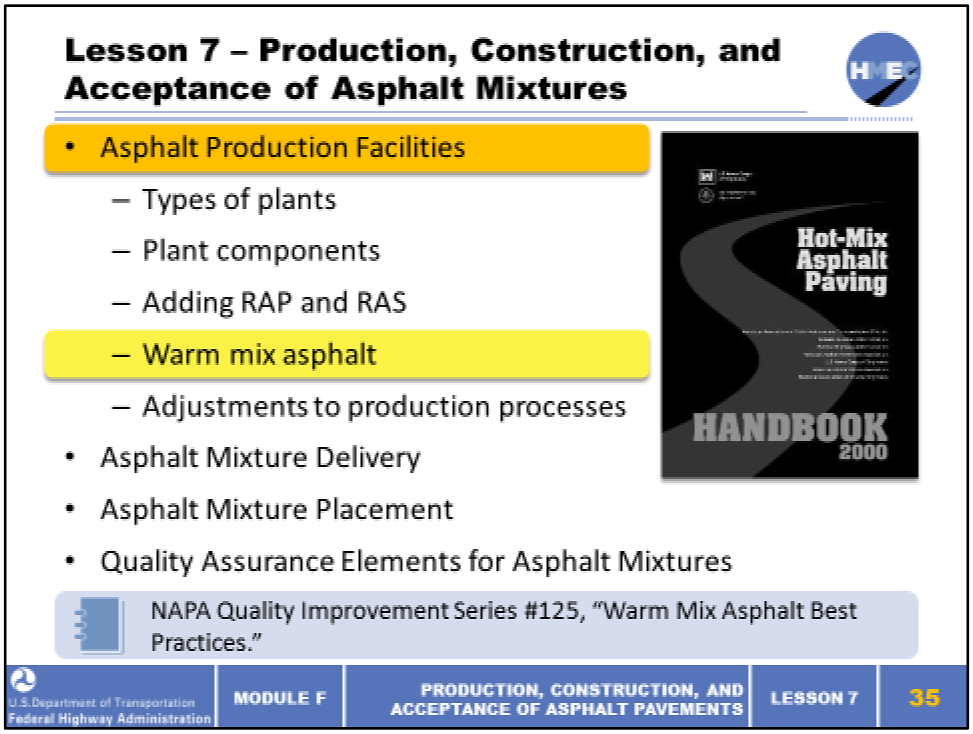
Slide 34



The use of RAS has been increasing in the US, especially in urban areas where there is usually a larger supply of material because of higher populations. The use of RAS in asphalt mixtures is usually grouped into two categories: manufactured waste and tear-offs. Manufactured waste is more uniform, but both can be used effectively.

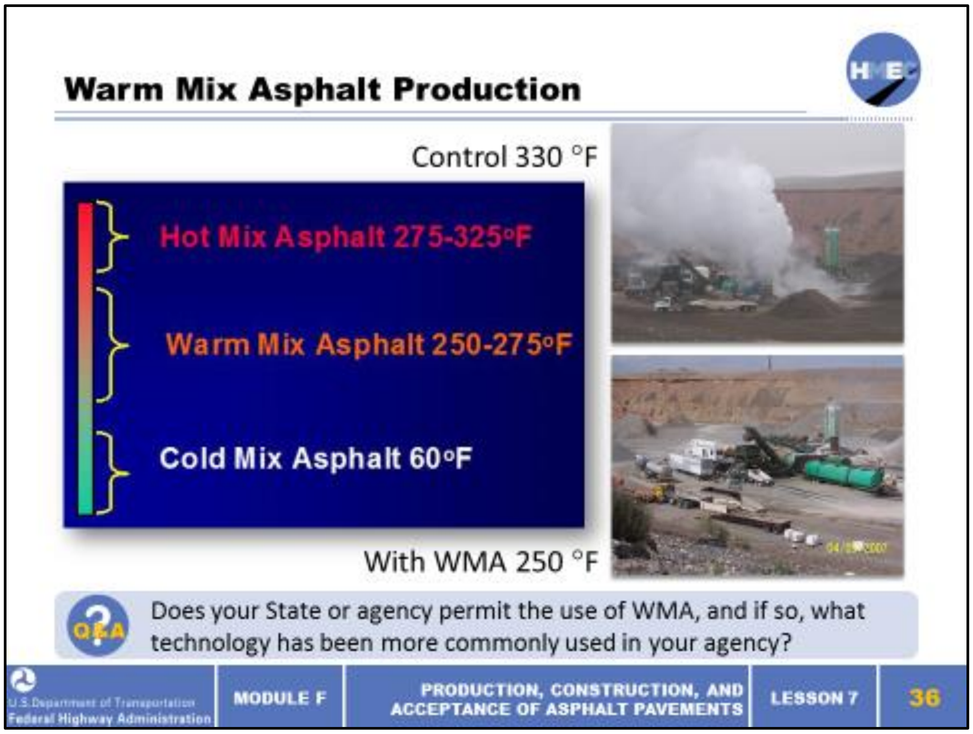
Slide 35



Two documents provide a lot of information on WMA in terms of design, construction, and performance of WMA, which are listed below:

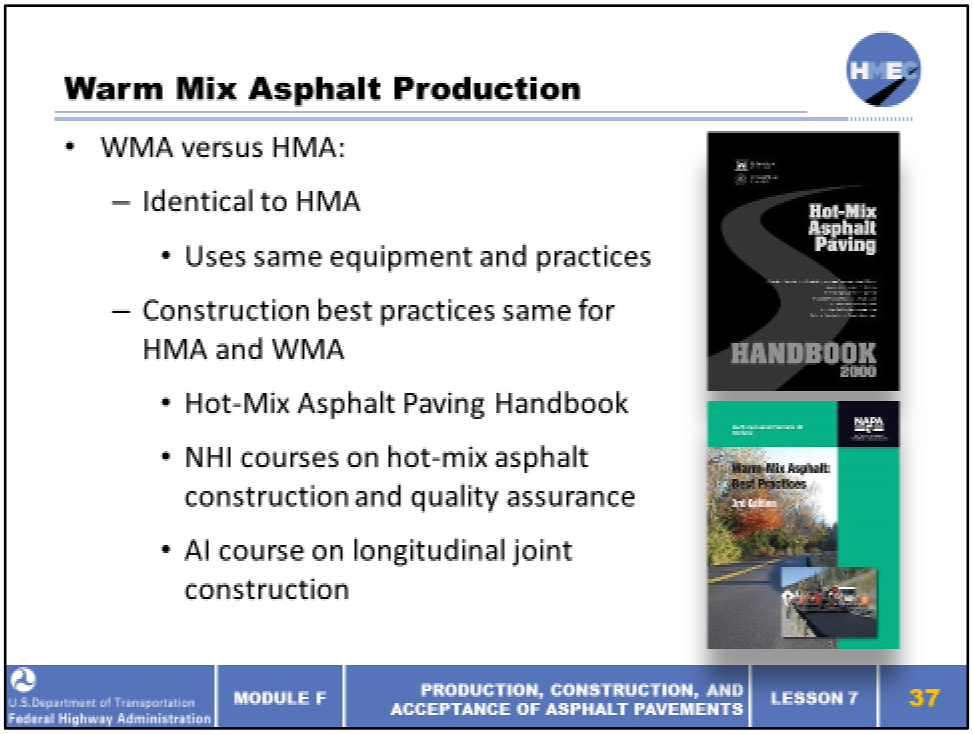
• A course developed by FHWA under the "Every Day Counts" program entitled "Mix Design, Construction, and Performance of Warm Mix Asphalt Mixtures." This document or course provides detailed information on the use, properties, and performance of WMA mixtures.
• NAPA Quality Improvement Series #125, "Warm Mix Asphalt Best Practices."

Slide 36



WMA technology has developed into more than just a reduction of emissions and fumes, even though that still is a major driving force. Warm mix technology is being used to improve the workability of stiff mixes, extend the paving window, extend haul times, allow paving in colder temperatures, and to increase the percentage of RAP used—all while enhancing the quality and performance of the HMA.

Slide 37



The WMA mix is delivered, placed, and compacted in the same way as HMA with no or minor differences between WMA and HMA. The same construction practices and equipment are used. The key is to follow the same best practices used in HMA construction for the mat, as well as for the longitudinal joints. The best practices focused on WMA are documented in the NAPA Quality Improvement Series #125 entitled “Warm Mix Asphalt Best Practice,” as shown in this slide.

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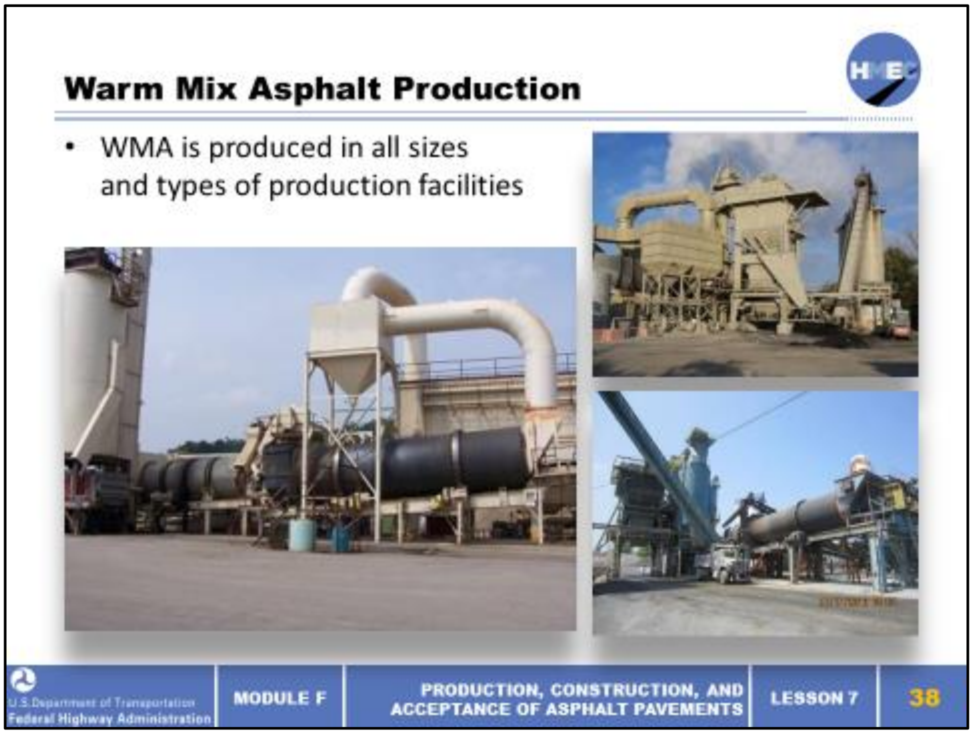
Slide 38


Warm Mix Asphalt Production
• WMA is produced in all sizes and types of production facilities
U.S. Department of Transportation
Federal Highway Administration
MODULE F
PRODUCTION, CONSTRUCTION, AND ACCEPTANCE OF ASPHALT PAVEMENTS
LESSON 7
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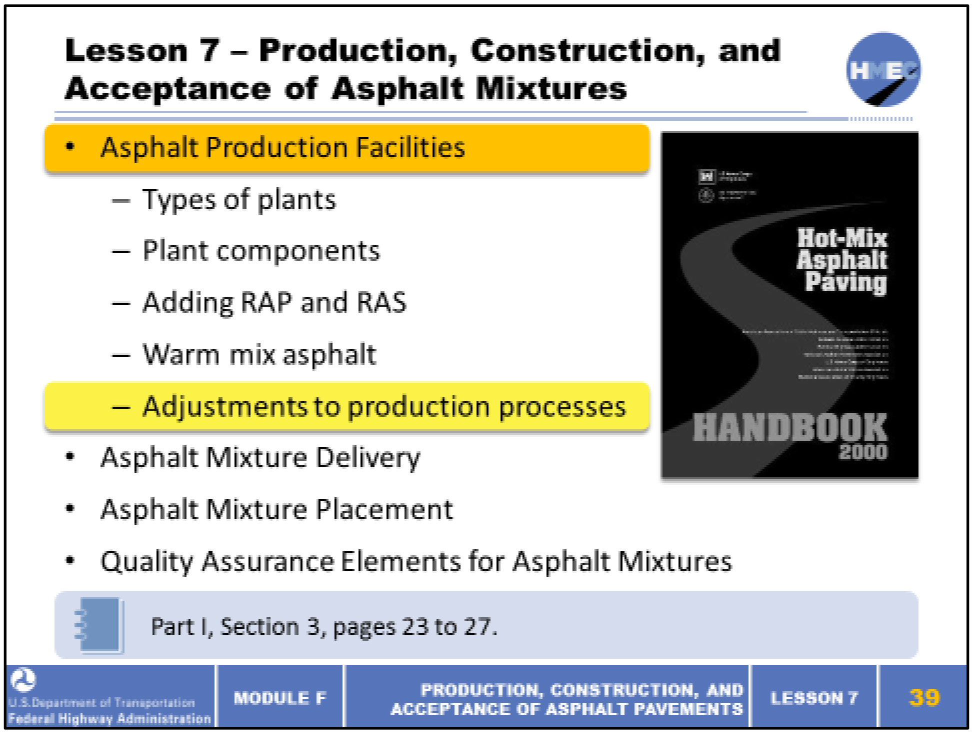
Slide 39



It is important that the mix properties used to formulate the job mix formula from the laboratory (refer to Lesson 5) be compared to the properties measured on the plant-produced mix. Slight adjustments will usually be required to the job mix formula for the plant-produced mixes.

See pages 23 to 27 under Section 3 of Part I in the *Hot-Mix Asphalt Paving Handbook* for a listing and discussion of the factors that differ between the lab and plant.

Slide 40

As listed on the previous slide; Section 3 of Part I in the *Hot-Mix Asphalt Paving Handbook* includes a listing and discussion of the factors that differ between the lab and the plant.

What are some of the major differences between the laboratory produced job mix formulas and the plant-produced mixtures, and how will those differences affect the properties of the plant-produced mixture?

Record your answers on the blank computer paper provided in your binder.

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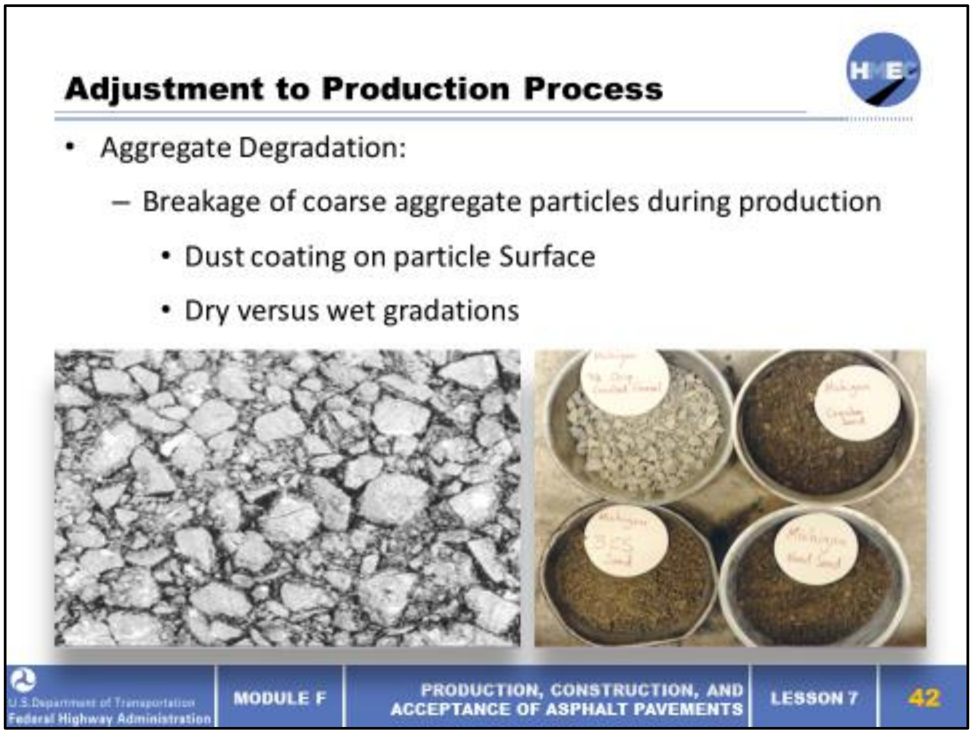
Slide 41

The point is that many different operating features can be used under different material conditions to produce HMA mixtures. All of these factors affect the hardening or viscosity of the asphalt and coating of the aggregate. The viscosity and material condition will have an impact on the compaction characteristics of the mix. A specific short-term aging procedure is used in the lab to simulate these different operating features and conditions. Using the same laboratory compactive (Ndesign) effort to determine the design or target asphalt content, however, will result in different air voids and other mix volumetric properties.

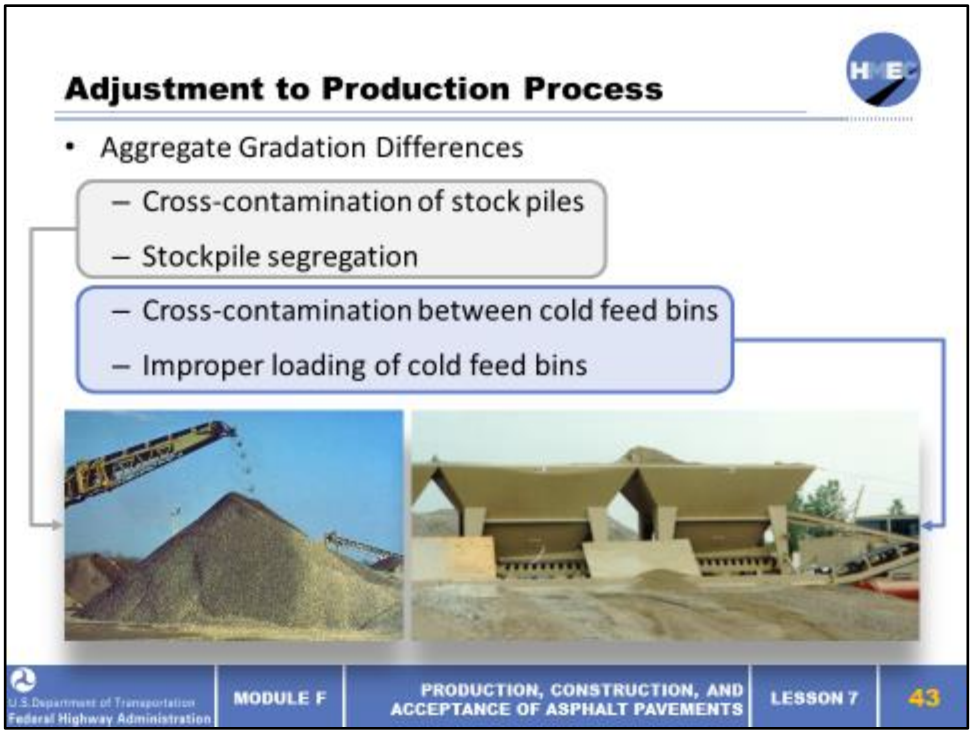
Slide 42



The degradation of the aggregate can and does create additional fines that can coat the coarse aggregate and change the adhesion properties of the aggregate. In many cases, the VMA decreases or is lower in the plant-produced mix in comparison to the lab-produced mixtures because of the additional fines that are created through the production process.

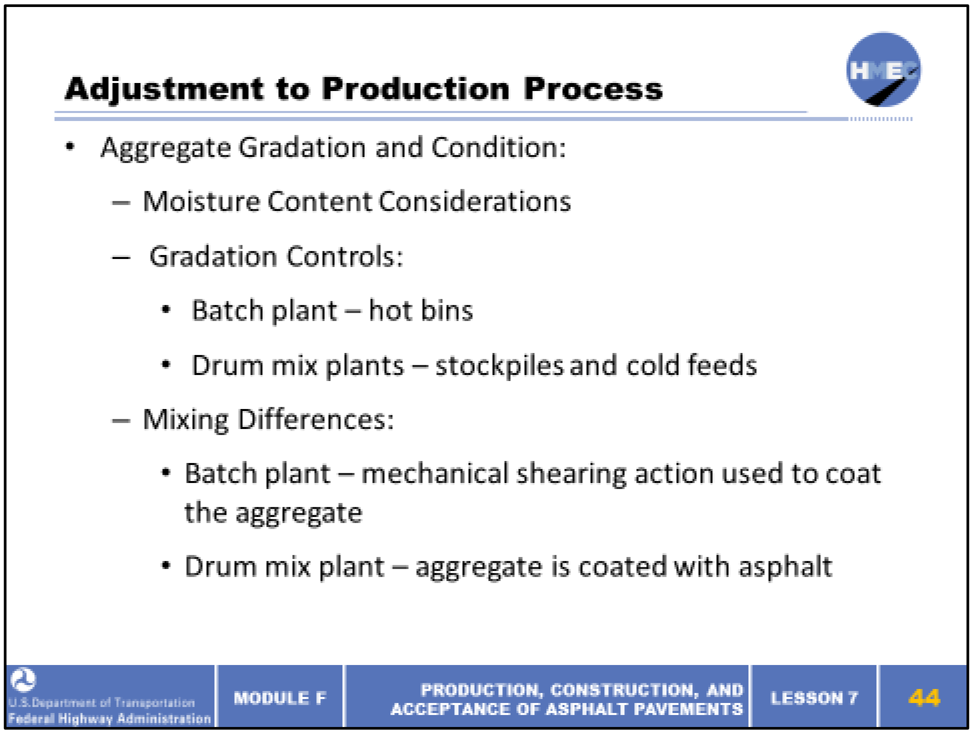
Higher water contents in the plant-produced mix can definitely have an impact on the volumetric properties of both lab-compacted and field-compacted mixtures.

Slide 43



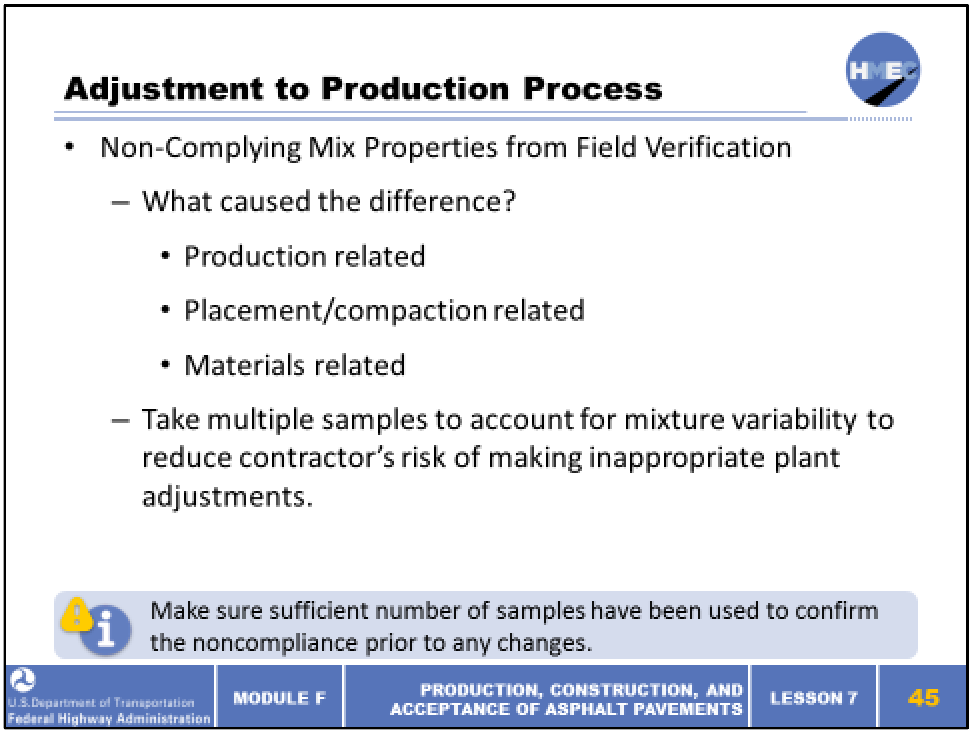
During actual production, however, the gradation can deviate as a result of variations in the stockpiles, and moving the aggregates from the stockpiles to the cold feed bins. The issue with these potential differences is that they are not constant during the production process.

Slide 44



In most cases, the field-produced mixtures will require slight revisions to the target asphalt content to verify and match the properties of the laboratory-produced mixtures.

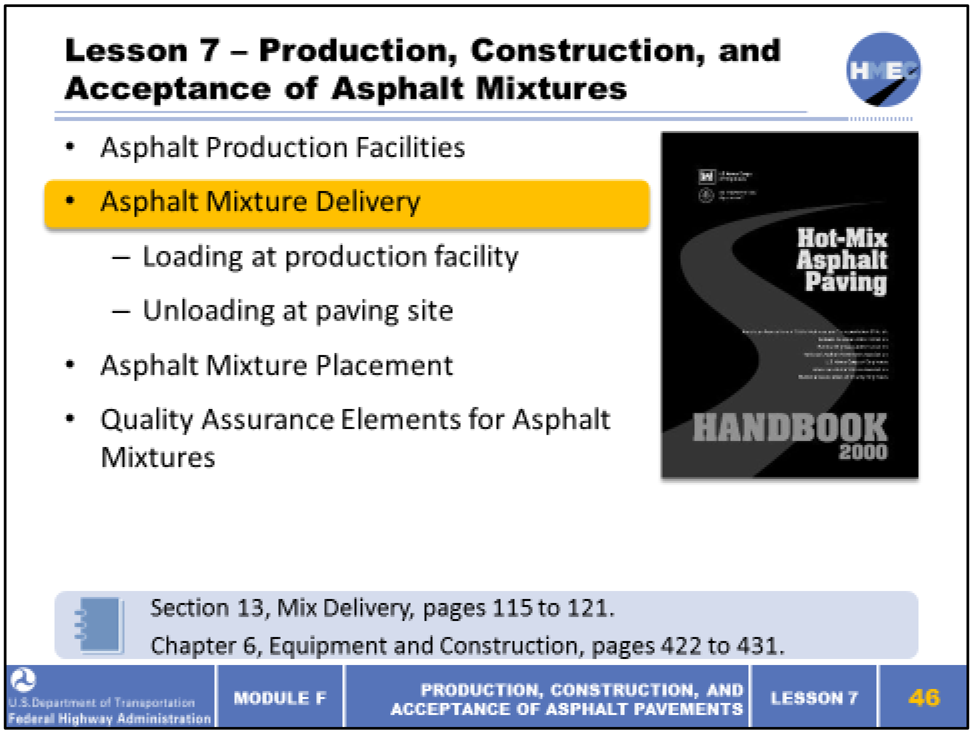
Slide 45



An important point is to make sure that a sufficient number of samples have been used to confirm the noncompliance (refer to Module A on determining the proper number of samples for making changes to the job mix formula) prior to making any changes.

The other point is determining whether the mix needs to be redesigned rather than just adjusting the job mix formula. If the target asphalt content changes by more than 0.2%, the mixture should be redesigned; however, this is agency dependent. This deviation also suggests that significant differences exist between the lab and field and those differences should be identified before moving forward with production.

Slide 46



Pages 115 to 121 of Section 13 of Part III in the *Hot-Mix Asphalt Paving Handbook* includes the proper loading and unloading of the trucks. In addition, pages 422 to 431 in Chapter 6 of the NCAT textbook provide additional information on transportation operations.

Slide 47


HMA Delivery – Haul Units
End dump trucks
Belly dump trucks
Live bottom trucks
U.S. Department of Transportation
Federal Highway Administration
MODULE F
PRODUCTION, CONSTRUCTION, AND ACCEPTANCE OF ASPHALT PAVEMENTS
LESSON 7
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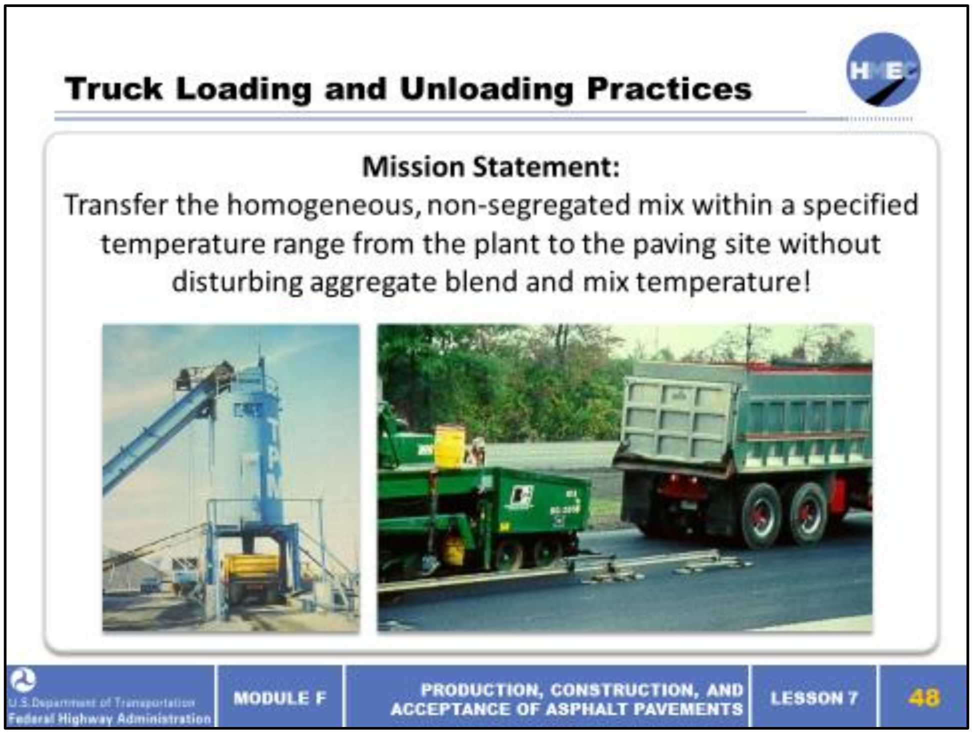
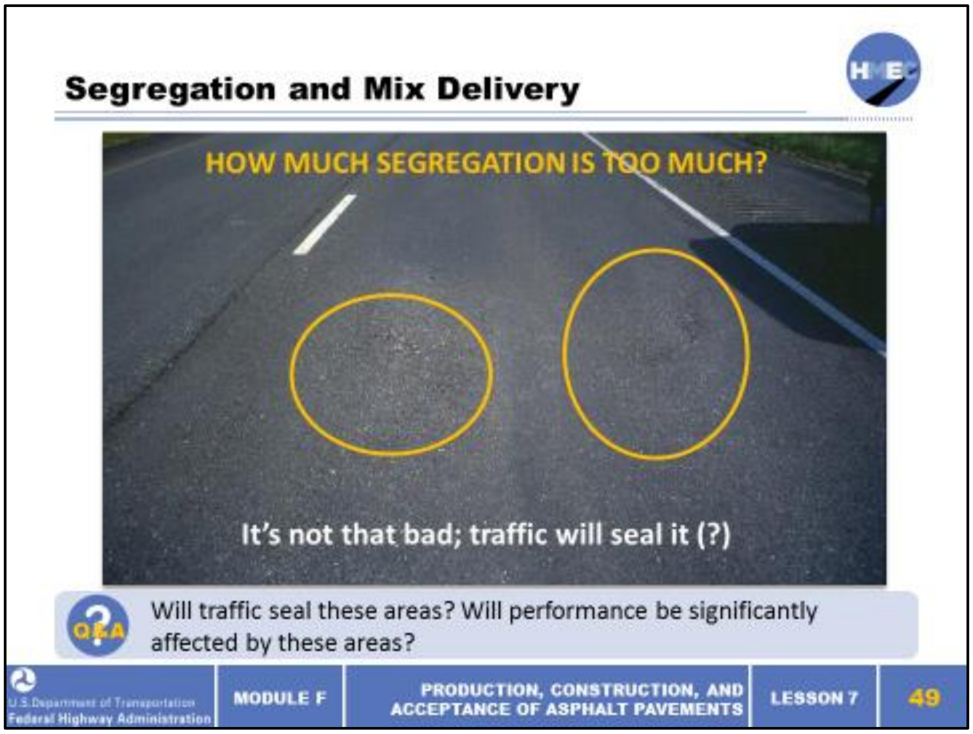
Slide 48



Upon receiving the non-segregated, homogeneous blend of HMA, within a specified temperature range from the silo or batch tower, it is the responsibility of the plant operator and truck driver to load and deliver that unsegregated mix to the paving site.

Slide 49



This slide shows two areas of segregated mix at the surface, which was caused by the truck loading practice. These segregated areas have high air voids and are susceptible to raveling and cracking.

Slide 50

This condition extends for miles along this project.

Slide 51

It is important that truck to truck segregation be identified and eliminated along the project.

Slide 52


Truck to Truck Segregation Pattern
Single Gate Silo
Paving Direction
Truck to truck segregation is primarily on left side of paver. Why?
MODULE F
PRODUCTION, CONSTRUCTION, AND ACCEPTANCE OF ASPHALT PAVEMENTS
LESSON 7
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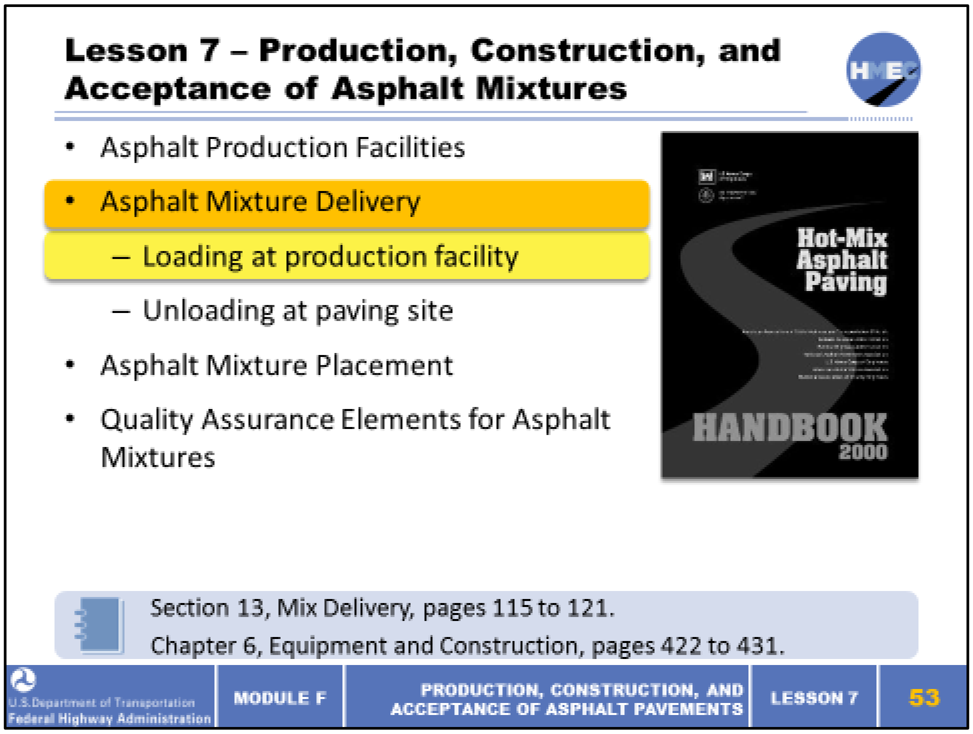
Slide 53

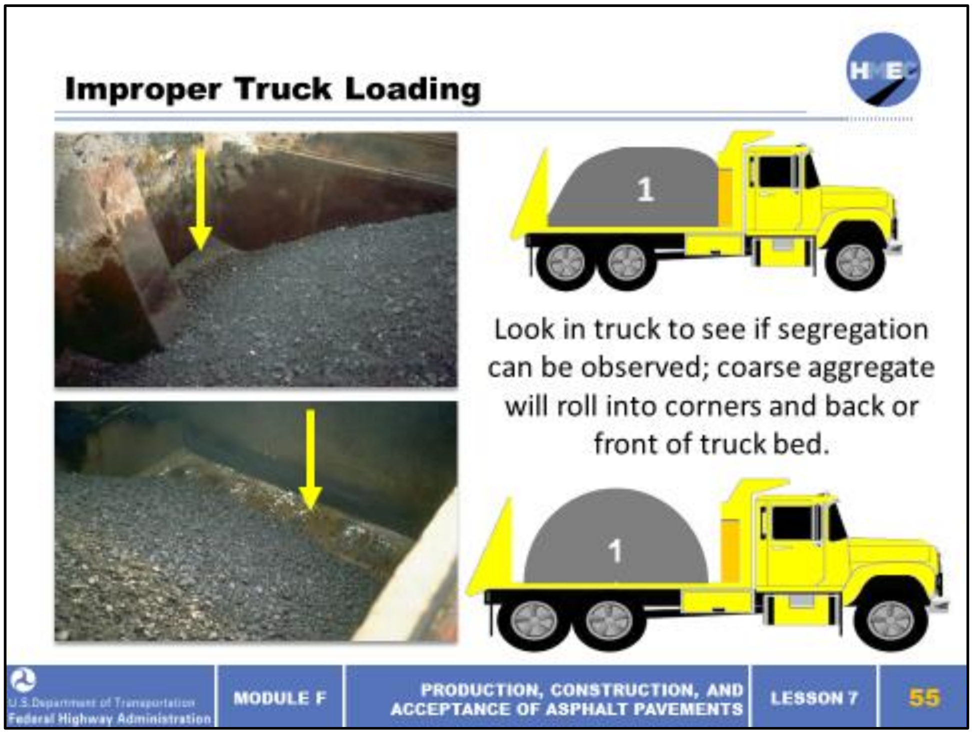


Pages 115 to 121 of Section 13 of Part III in the *Hot-Mix Asphalt Paving Handbook* includes the proper loading and unloading of the trucks. In addition, pages 422 to 431 in Chapter 6 of the NCAT textbook provide additional information on transportation operations.

Slide 54

As shown in this slide, three dumps are used to load the front end and rear end of the truck. The focus is to ensure that mix is placed against the tailgate and front part of the bed, and then fill in the center of the truck bed with mix. The key is to eliminate the probability of the coarse aggregate rolling down the slope of the mix in the bed as much as possible. In other words, eliminate coarse aggregate separating from the fine aggregate.

Slide 55



This slide shows a prime example of HMA of one dump being used to load the end dump truck—either at the center of the truck or at the front or rear end of the bed. This allows the coarse aggregate to "break and run." As the HMA cascades from the discharge point to the front and rear of the truck bed, the larger aggregate rolls down the surface and accumulates at the bottom of the bed. The coarse aggregate will stay in this segregated mass unless it is corrected and will show up in the pavement as load to load or truck to truck segregation.

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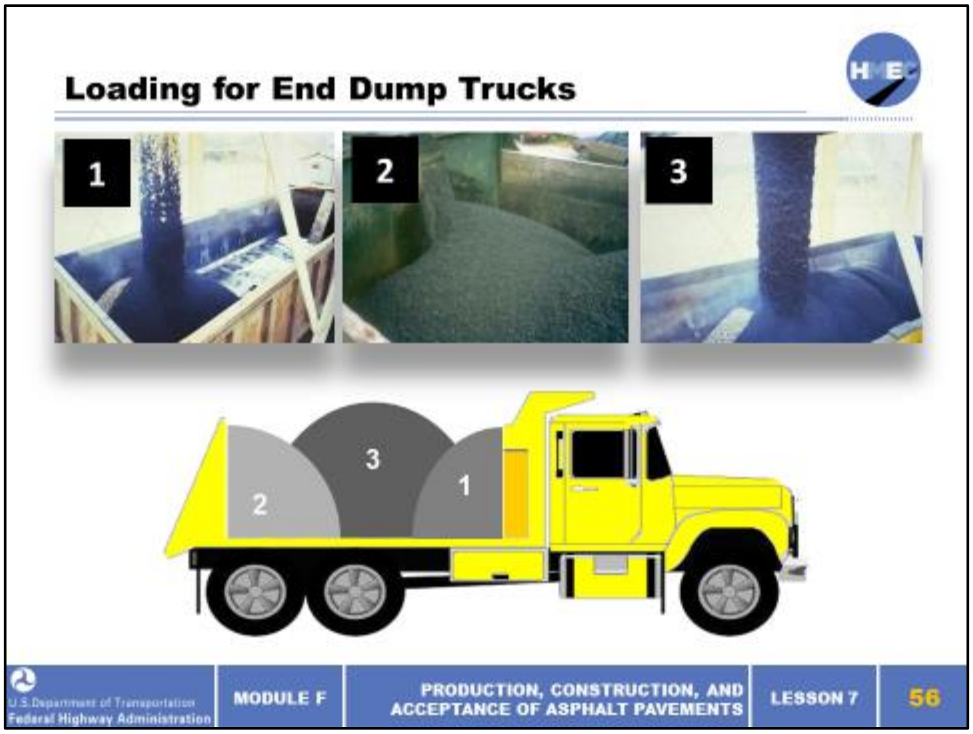
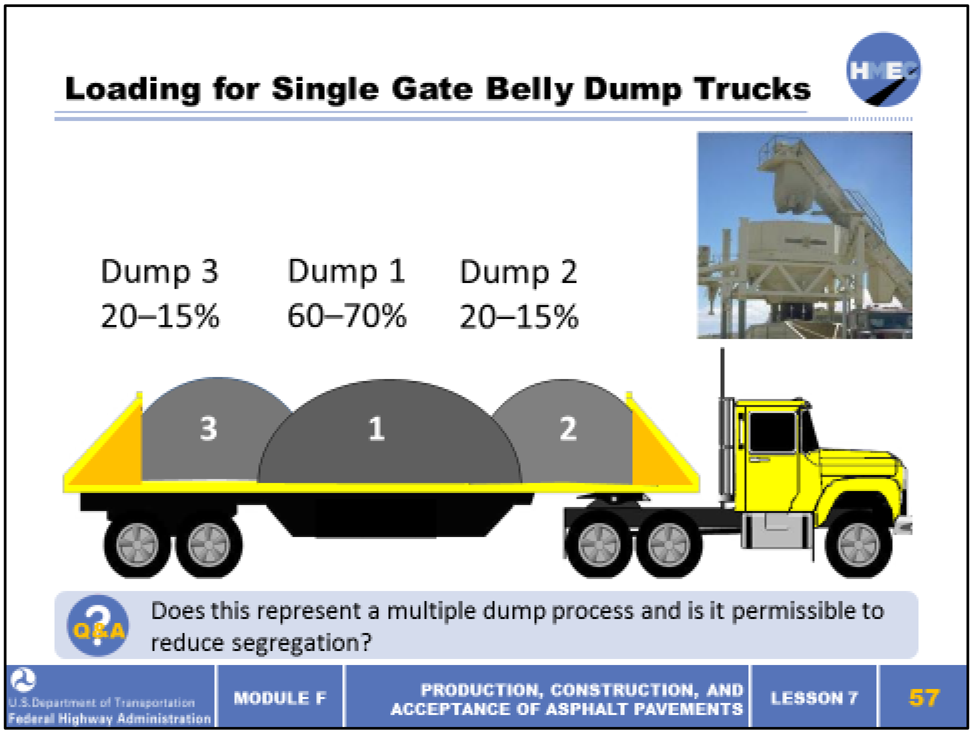
Slide 56



This slide shows the first dump at the rear of the bed—near the tailgate, the second dump at the front end of the bed, and the third and/or fourth dumps in the middle of the bed.

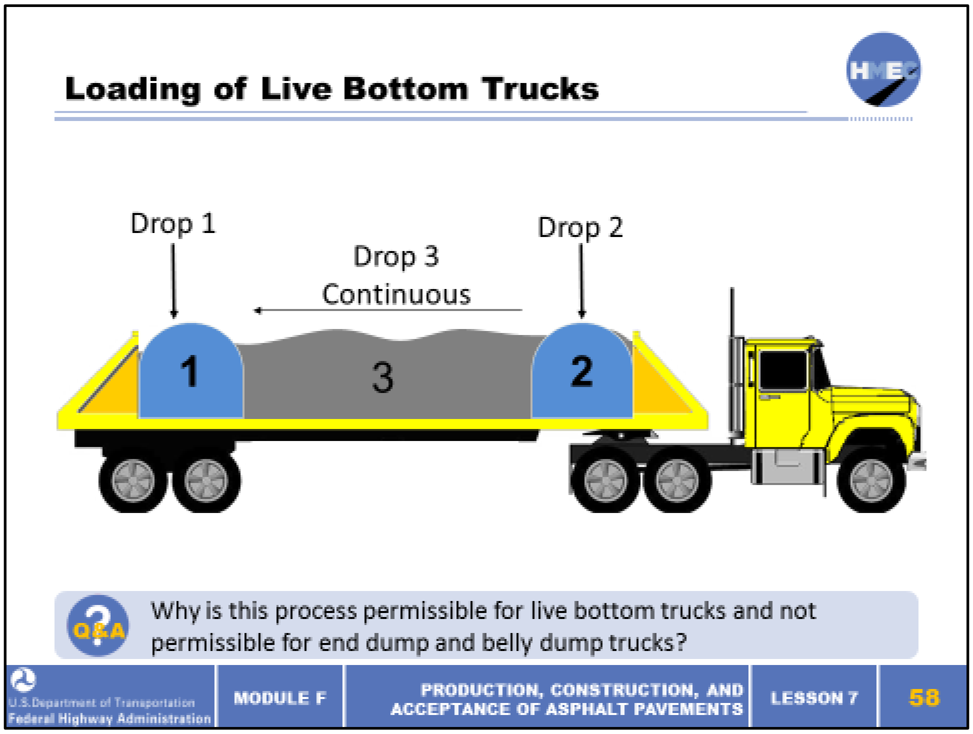
Slide 57



As with smaller end dumps, the front and tailgate are loaded first; then the space between is loaded with small, separate piles. This slide shows a schematic for loading a single gate belly dump (or bottom dump) truck. When single gate belly dumps are used, the first dump should be directly over the gate. The second and third dumps should be at the front and then at the tailgate getting as close to the front and rear ends of the bed.

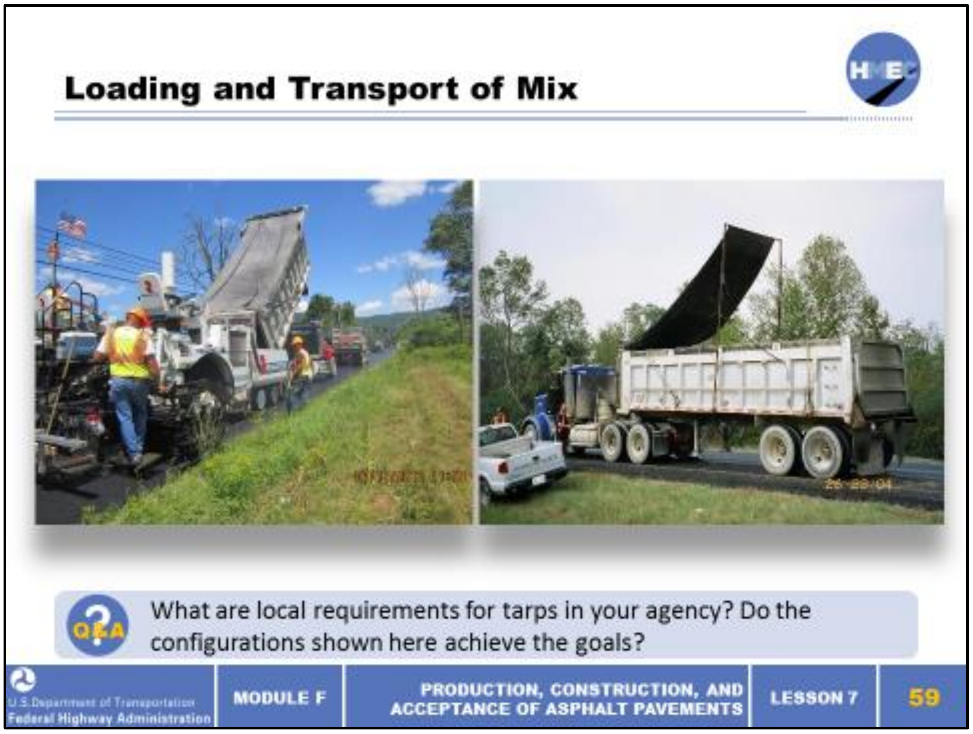
For larger capacity belly dump (or bottom dump) trucks with two gates, even more drops are necessary. The first two dumps should be located over the bottom gates. The final dumps should be located near the font and end of the truck bed.

Slide 58



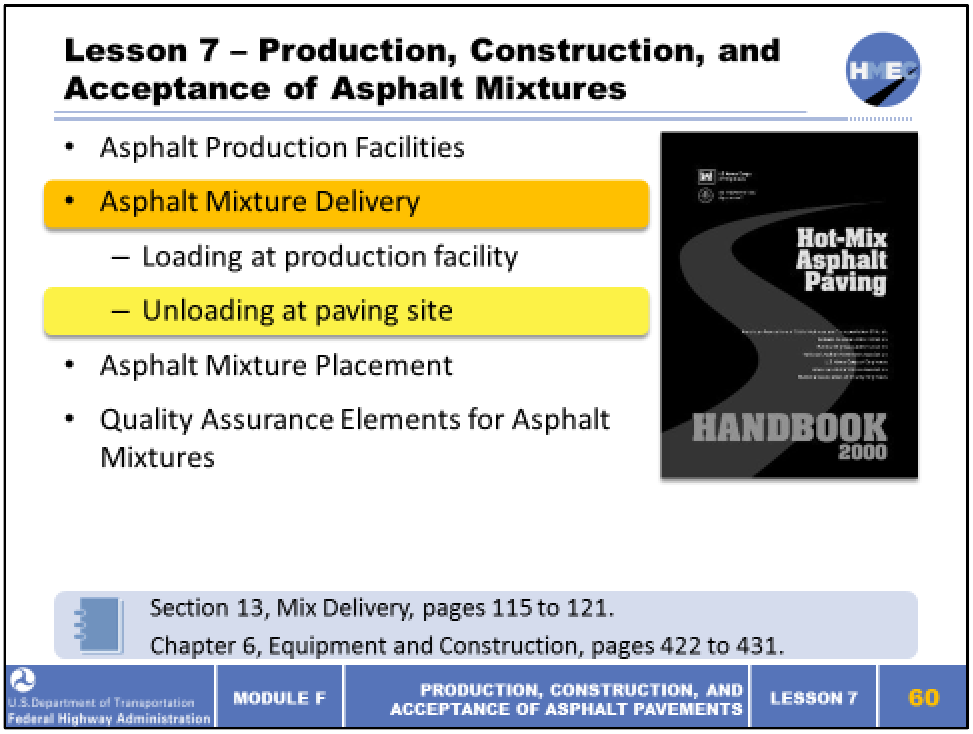
For the larger live bottom (horizontal discharge trucks or flow boys) trucks, the first drop of mix or dump should be located at the rear end gate. The truck driver then backs up the truck so the second dump can be located at the front end of the bed. For large live bottom trucks, it is permissible for the truck driver to slowly move forward as mix is continuously dumped to fill in the middle of the bed, as shown in the schematic.

Slide 59



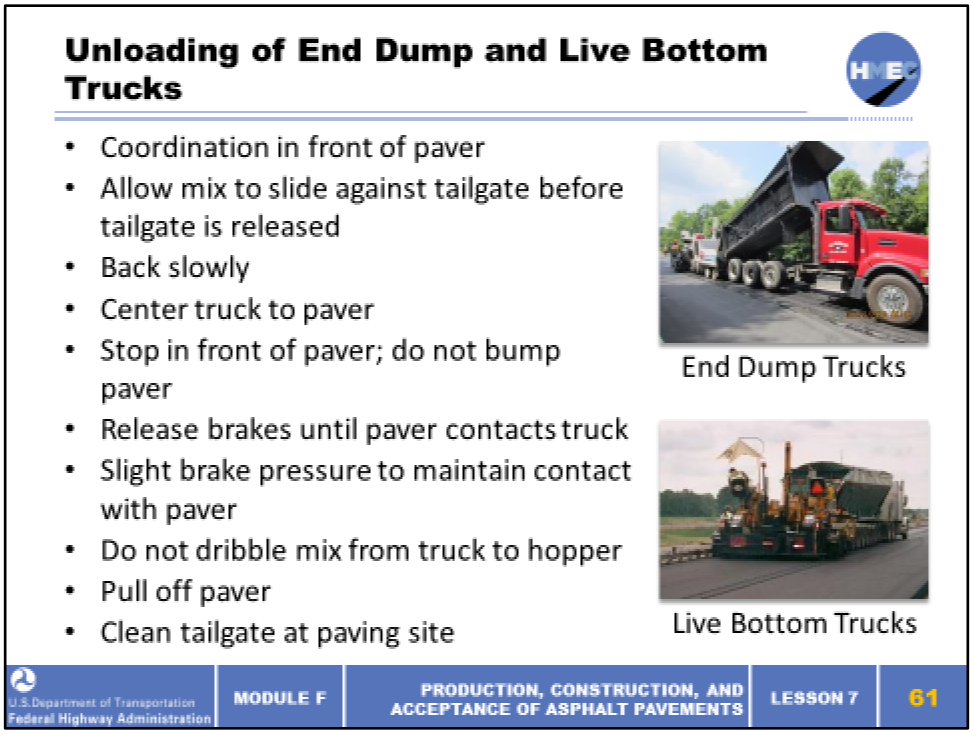
Most agencies require the use of tarps to keep rain and other debris out of the bed in protecting the mix, as well as trying to insulate the mix.

Slide 60



Pages 115 to 121 of Section 13 of Part III in the *Hot-Mix Asphalt Paving Handbook* includes the proper loading and unloading of the trucks. In addition, pages 422 to 431 in Chapter 6 of the NCAT textbook provide additional information on transportation operations.

Slide 61



Once a truck is loaded, it is important that the truck proceed immediately to the project site to limit temperature loss. Once on site, the mix should be delivered to the paver in the most rapid time possible. The truck driver should coordinate with the paver operator or dump operator and deliver the weight ticket to the on-site person.

Before the other truck leaves the paver, the truck driver should raise the bed if an end dump truck or engage the conveyor if a live bottom truck is used to break or charge the load against the tailgate. It obviously is important for the truck driver to ensure the tailgate is still closed and secured.

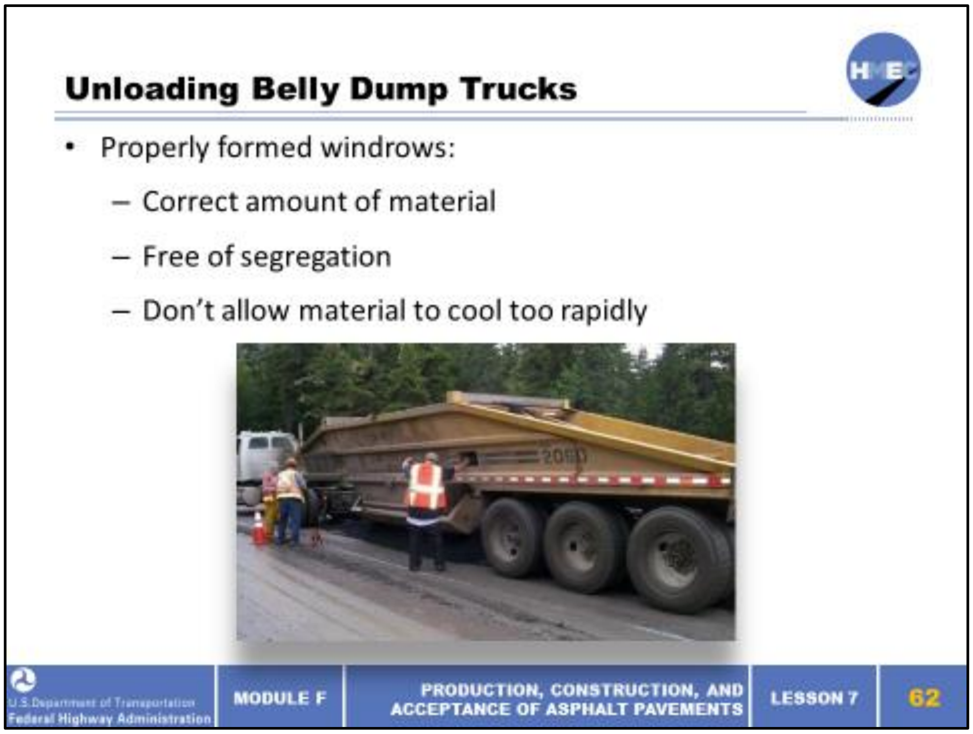
After the previous truck leaves the paver, the truck driver starts to back up slowly to center the truck with the paver. The truck driver should stop the truck just in front of the paver so that the truck does not bump the paver and release the brakes, if not on an incline.

When the paver makes contact with the truck, slight brake pressure can be used by the truck driver to maintain paver-truck contact if the paver operator does not engage the truck. It is better if the truck is attached to the paver, because different truck drivers apply different slight brake pressures which causes the paver to operate differently with different brake pressures being applied between drivers. After contact, the truck driver continually raises the bed and releases the tailgate to move the material against the tailgate in a single mass into the paver hopper. The tailgate should then be opened an adequate amount to feed the paver without dribbling materials.

Once the truck is empty, truck driver lowers the bed all the way down and pulls away from the paver. The truck driver should not bang the tailgate in front of the paver. The truck bed and tailgate should be cleaned, if necessary, in the designated clean-up area.

If draindown or other unusual conditions are observed in the truck bed, they should be reported.

Slide 62



It is important that windrows be properly formed with the correct amount of material, free of segregation, and only placed as far as necessary to feed the paver without allowing for the material in the windrow to cool too rapidly.

The next set of slides provide more detail for unloading the different types of trucks.

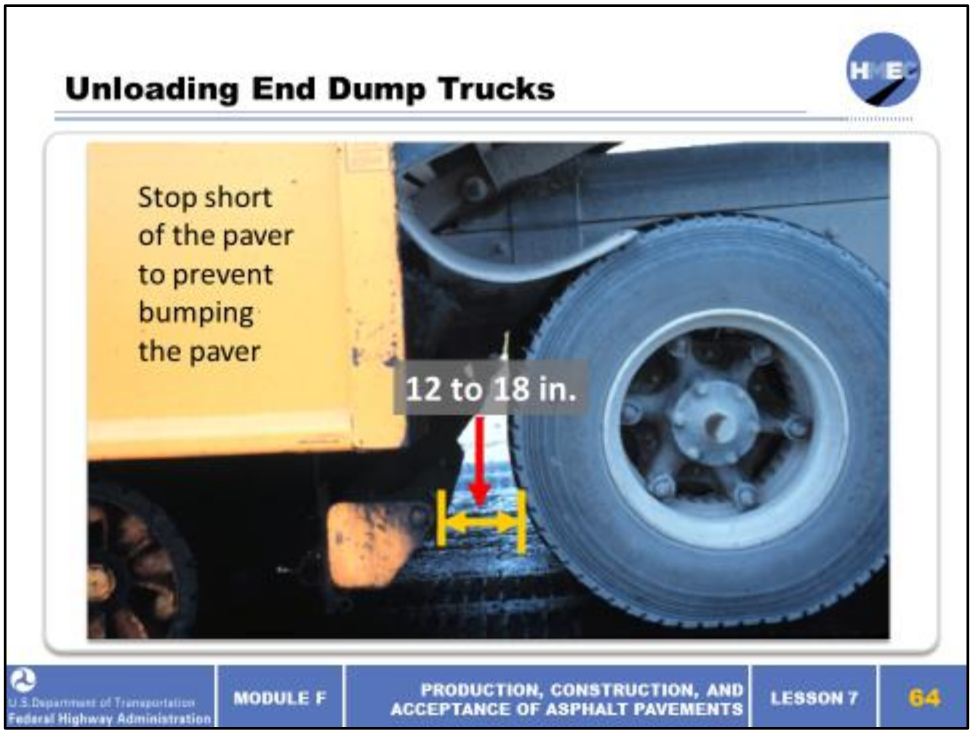
Slide 63

Here you see “breaking the load” and how it can improve quality and production if done correctly.

1. With end dump trucks, the proper procedure for dumping the mix into the hopper is to raise the truck bed slightly and allow the mix to slide against the tailgate before it is released. The mix should already be against the tailgate in loading the truck at the plant.
2. This procedure will allow the mix to flood the hopper—not allowing mix to dribble from the truck into the hopper before the bed is raised.

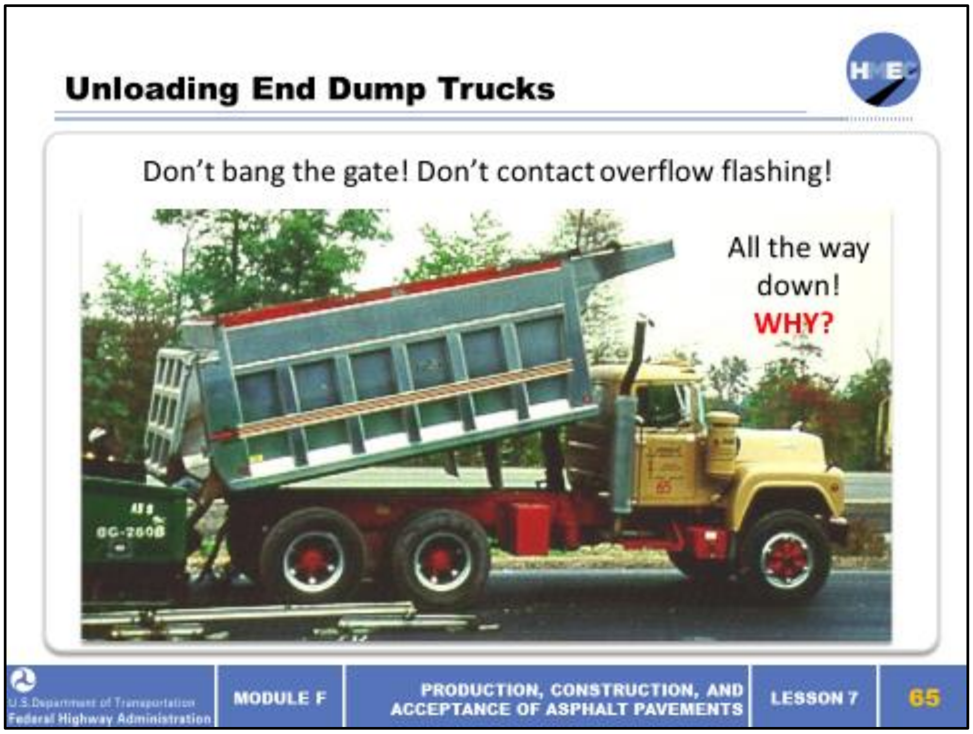
The bed of the end dump truck should not be raised too high, so the apron of the end gate does not come in contact with the paver hopper flashing that prevents mix from spilling from the paver hopper on the surface in front of the paver.

Slide 64



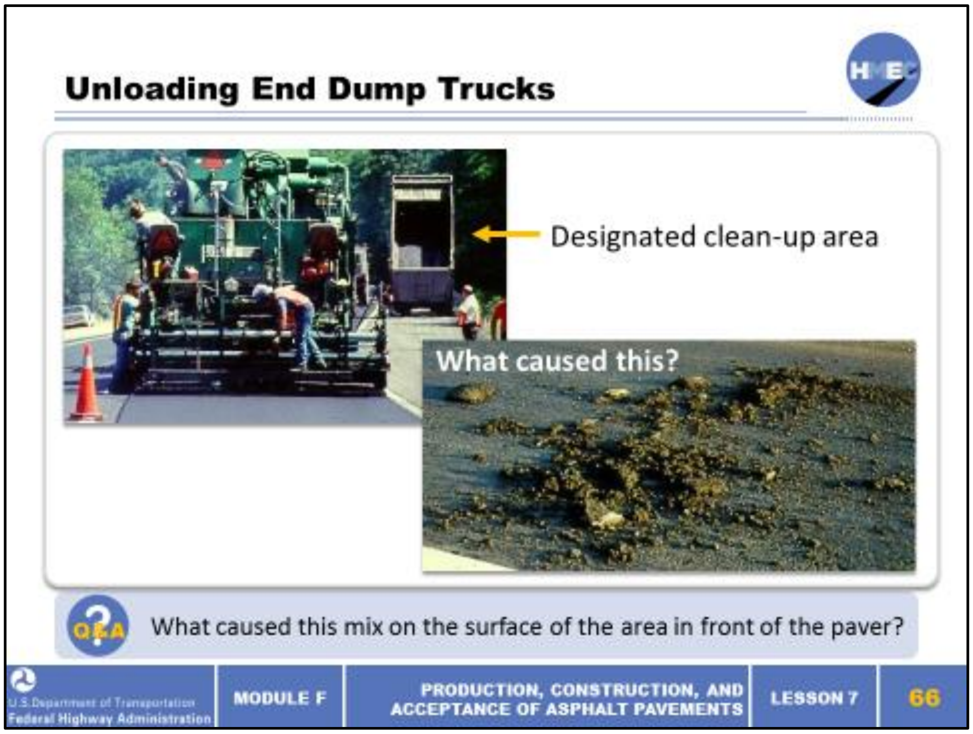
The paver should not be too far from the paver to ensure the hopper will retain a sufficient supply of mix before the truck driver opens or releases the tailgate and delivers the mix into the paver hopper.

Slide 65



Lowering the bed allows the truck apron to clear the hopper guards and avoid 'bumping" or dragging the flashing of the paver hopper forward.

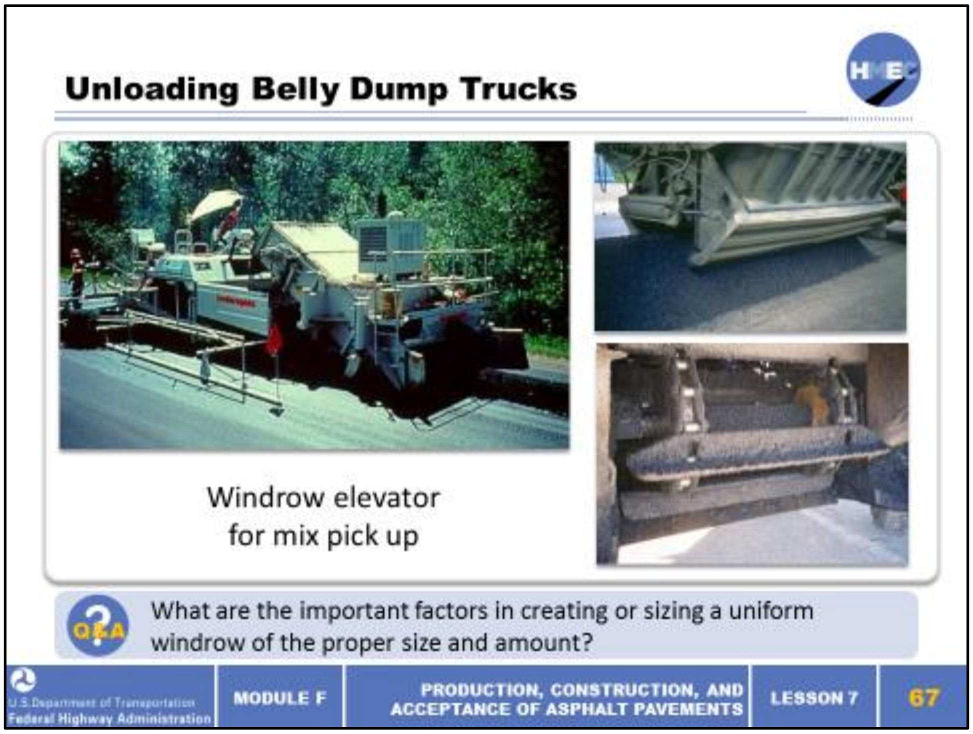
Slide 66



This process should be prevented because a paver passing over this mix from the bed will create rough spots and the mix will cool rapidly leaving an areas with potentially much higher air voids at the bottom of the lift.

After discharging the mix in the paver hopper, the truck bed should be lowered all the way down and the truck proceed to the designated clean up area.

Slide 67



In some cases, horizontal discharge trucks are used to create windrows. However, windrows created from live bottom (or horizontal discharge) trucks is not recommended because the uniformity in terms of amount of material is difficult to place on a consistent basis.

The slat conveyor on the windrow elevator must be adjusted to pick up all of the mix from the pavement. The windrow should be sized to keep the paver hopper 25 to 75% full. Don't overfill the hopper.

Slide 68

The windrow has been sized with too much mixture, so the paver hopper has become too full. When this occurs a portion of the windrow must be removed and the windrow rebalanced to start over. This will affect the finished mat so caution must be exercised to not let this happen! It is better to slightly undersize the windrow, because it is easier and takes less time to add mix to the windrow, rather than remove mix from the windrow.

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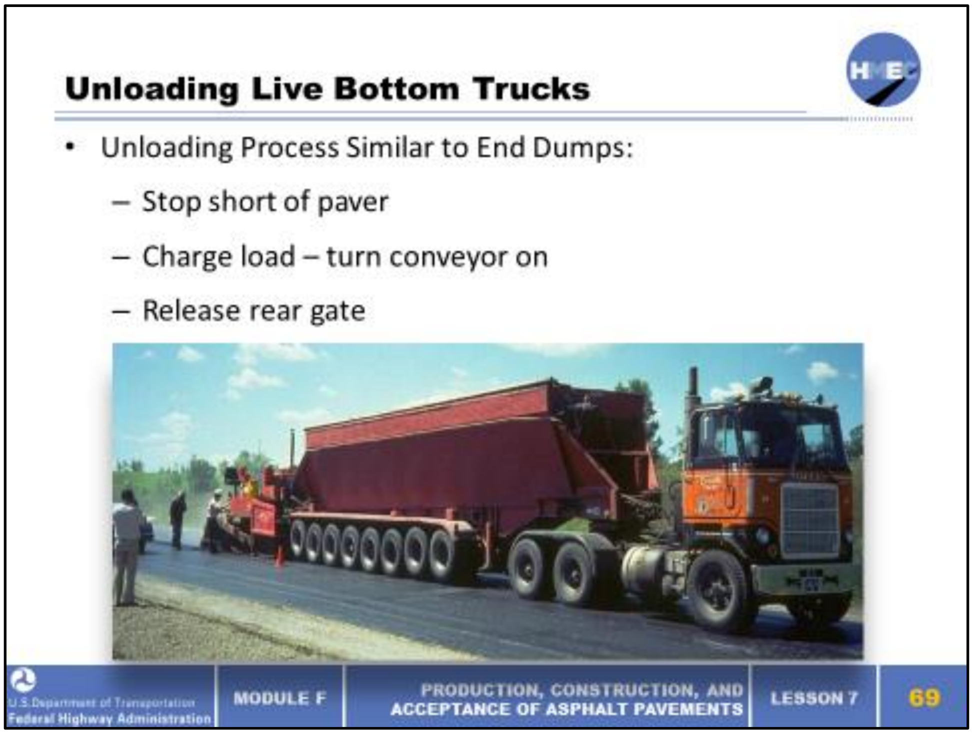
Slide 69



In summary:

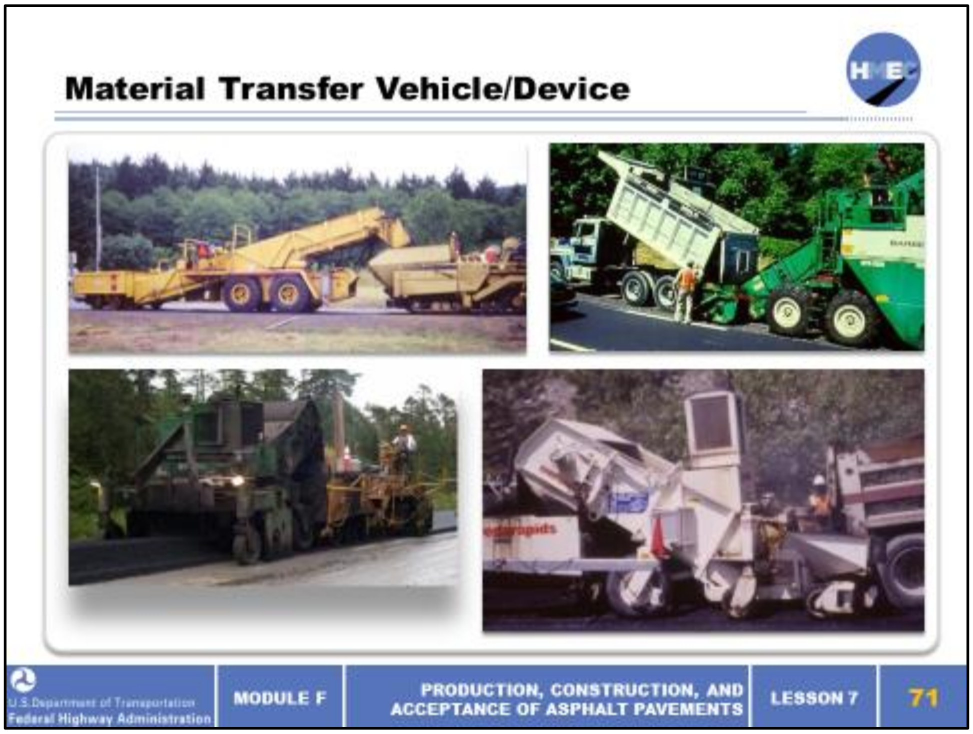
• Truck driver should turn on the conveyor belt to “charge the mix” at the rear gate, making sure the gate is closed.
• Stop short of the paver.
• After the paver contacts the truck, the rear gate is released or opened.
• Mix is delivered into the hopper at a continuous rate, until the truck is emptied.
• The rear gate is closed and the truck departs the paving site.

Slide 70

MTVs are required by many agencies in placing HMA mixtures on high volume routes like the interstate system. They have proven very effective in eliminating segregation, minimizing temperature differences between truck loads, and creating a smoother mat surface because trucks do not impact the paver and the paver has a greater possibility of paving at a constant rate or speed and does not stop.

The windrow elevator (discussed under the section of belly dump trucks), which is required to pick up the asphalt mixture dumped on the roadway by belly dump trucks to the paver hopper, acts like a MTV in concept.

Slide 71



The devices have very large capacities and multiple augers are used to move the material through the material transfer device.

The net effect of transferring material through a MTV is remixing in bulk that results in uniform mixture temperature and the reduction of variation in gradation. The devices can essentially eliminate segregation generated in the plant production, storage, transportation, and delivery processes.

Due to the large capacity of the MTV equipment, a constant feed of material can be delivered to a paver even if small gaps in trucking exist. This will assist with generating a uniform finished mat by eliminating the ability to cycle hopper wings between loads; providing a uniform feed to the screed; and by eliminating the need for starting and stopping that can lead to poor ride quality.

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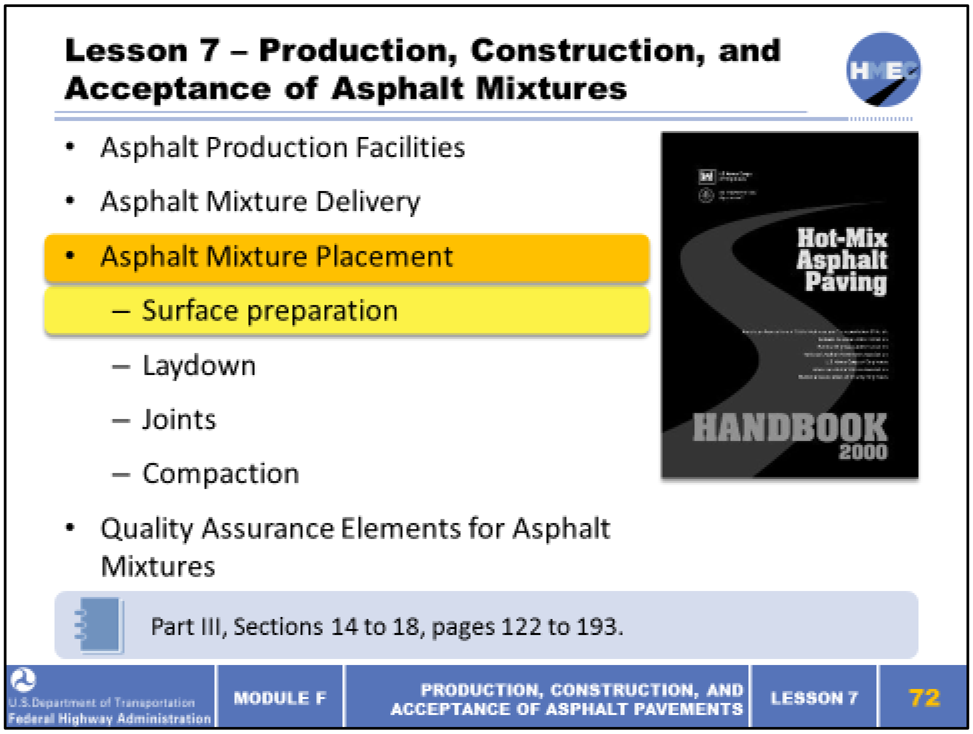
Slide 72



Pages 122 to 193 of Part III, Sections 14 to 18 in the *Hot-Mix Asphalt Paving Handbook* is focused on mixture placement through compaction.

Slide 73

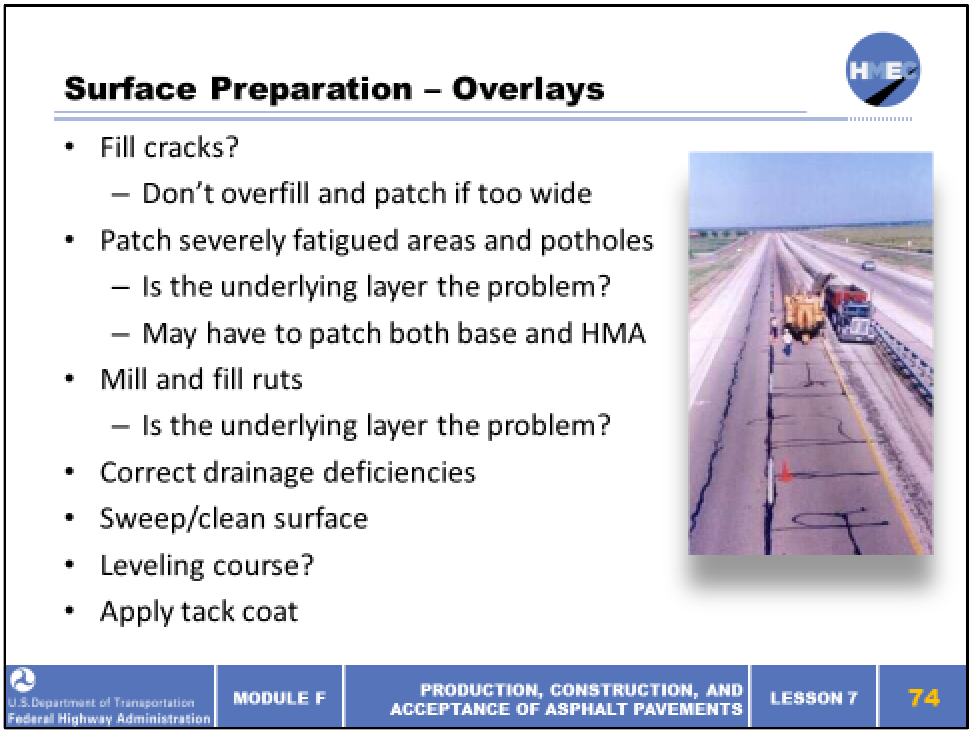
Adequate compaction of underlying materials requires development of moisture density relationships for the materials such that optimum moisture and density targets can be established. The proper equipment for the process must be secured and operated correctly.

During and post-compaction, in-place density must be monitored by QC personnel to ensure proper compaction. In addition to QC operations, compaction can be proof rolled (or tested) with a full water truck.

Finally, if the tires on a paver are leaving significant indentations in a base course, the paving should be terminated and the base compaction improved. This is particularly important when a base course is prepared several days before HMA placement is initiated.

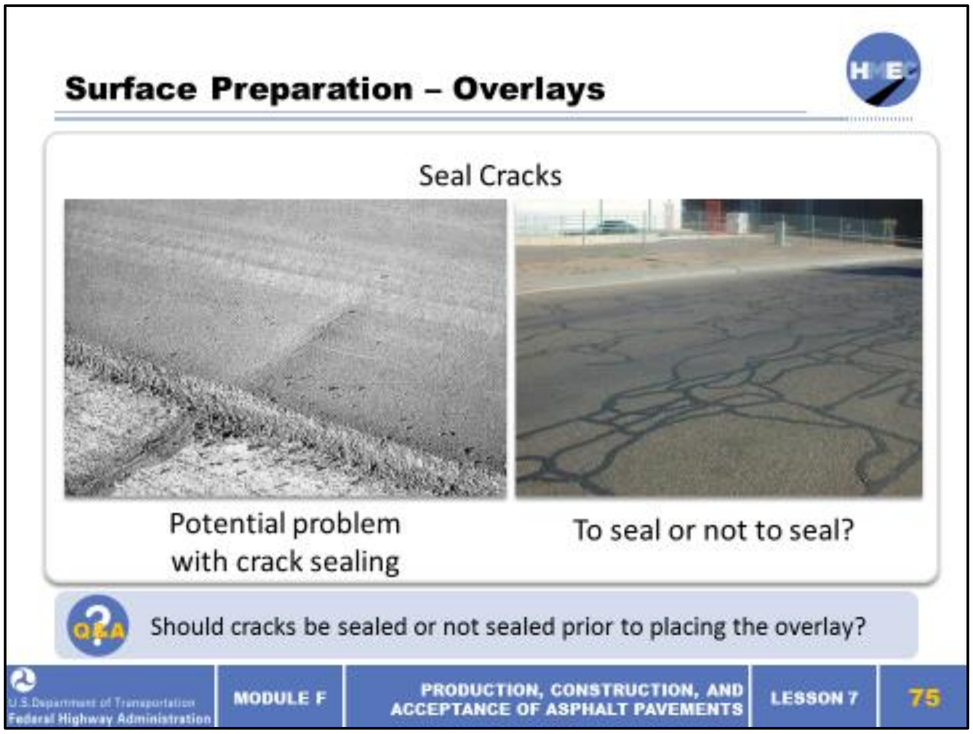
Prime coats serve to seal the base course and facilitate a bond between the base course and HMA. It is very important to apply a uniform prime coat using the correct material at the correct application rate.

Slide 74



Proper preparation is project-specific and dependent on the condition of the existing pavement, which are overviewed in the next set of slides.

Slide 75



If extensive cracking exists on the pavement surface, sealing all of the cracks is usually uneconomical compared to other techniques. In addition, the benefit of sealing cracks just prior to placing the overlay is questionable. Other surface treatments that have been used to seal cracks (especially when they are narrow) and reduce the potential for reflection cracking include chip seals and slurry seals. These methods are believed to be more beneficial than just sealing the cracks. The effectiveness of crack sealing depends upon the cause and amount of cracking in the existing surface. Depending on the extent of cracking, they may be less expensive methods compared to crack sealing. The photograph on the right side of this screen is an example where other methods are probably better or more economical than crack sealing.

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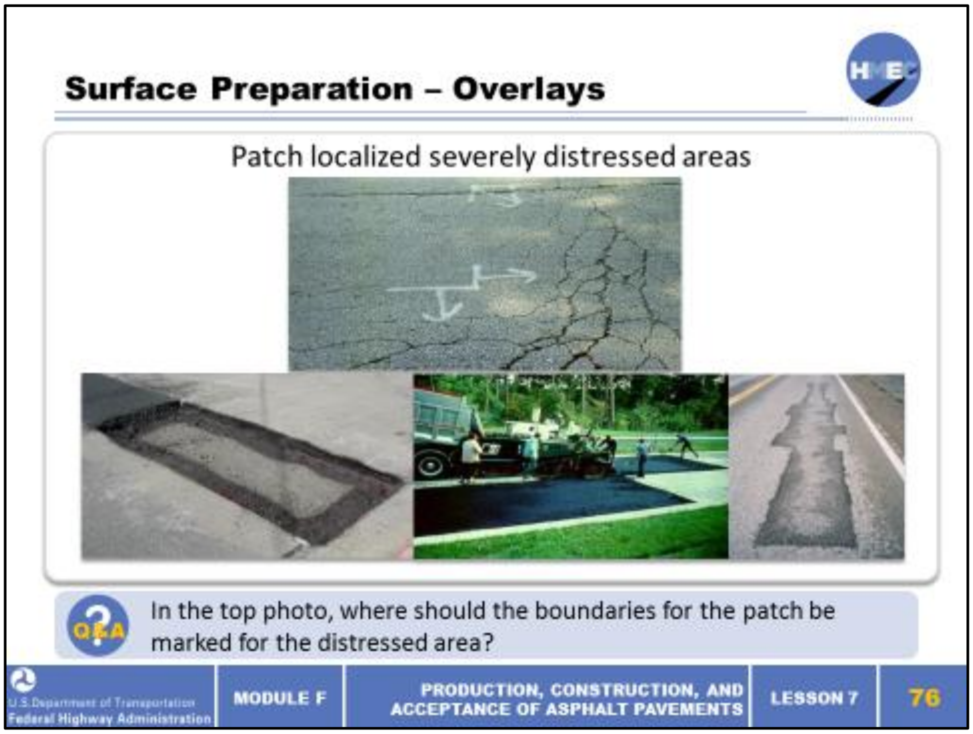
Slide 76



The first step for making these repairs is to mark the boundaries of the repair area. Apply the following criteria:

- Identify limits that exceed the distressed area (recalling that the condition of the HMA is usually worse at the bottom than at the top).
- Mark straight boundary lines with areas as rectangular as possible.
- Consider width of compaction equipment.

For small patches, however, the patch material should be placed in the hole using shovels, taking care to avoid segregation of the large aggregate. (Rakes are not recommended for material placement because of their propensity for segregation). For large patches, the patch material can be placed using a paver. Large patches should be placed considering the following:

- A tack coat is necessary to ensure a proper bond between the patch and the existing pavement. It also helps to prevent moisture infiltration.
- The patch area should be cleaned of rubble and debris prior to tacking.
- Tack coats are applied uniformly using either a spray wand or tack brush.
- Care should be taken to avoid over-application and puddling.

Slide 77

Milling is used extensively in many rehabilitation projects. Milling can re-level or improve the condition of the surface at minimal costs. An important point regarding the milling operation is not leaving a scab of mix from a layer, as shown in the photograph located on the right side of the screen. The depth of milling should be determined to remove the entire lift of the asphalt mat being milled to just below the interface with the next lift.

The surface should be adequately swept after any milling operation and/or before placing the tack coat or HMA overlay.

Slide 78

A key point is determining the correct application rate. The photograph in the lower right-hand side of the screen shows a single and double application rate of a paving grade asphalt used for the tack coat. It is not necessary to cover 100% of the existing surface with a tack coat—enough tack coat needs to be placed so that a good bond exists between the existing layer and overlay. In general, at least 75% of the surface should be covered uniformly with the tack coat. The application rate depends on the condition of the existing pavement surface.

Slide 79

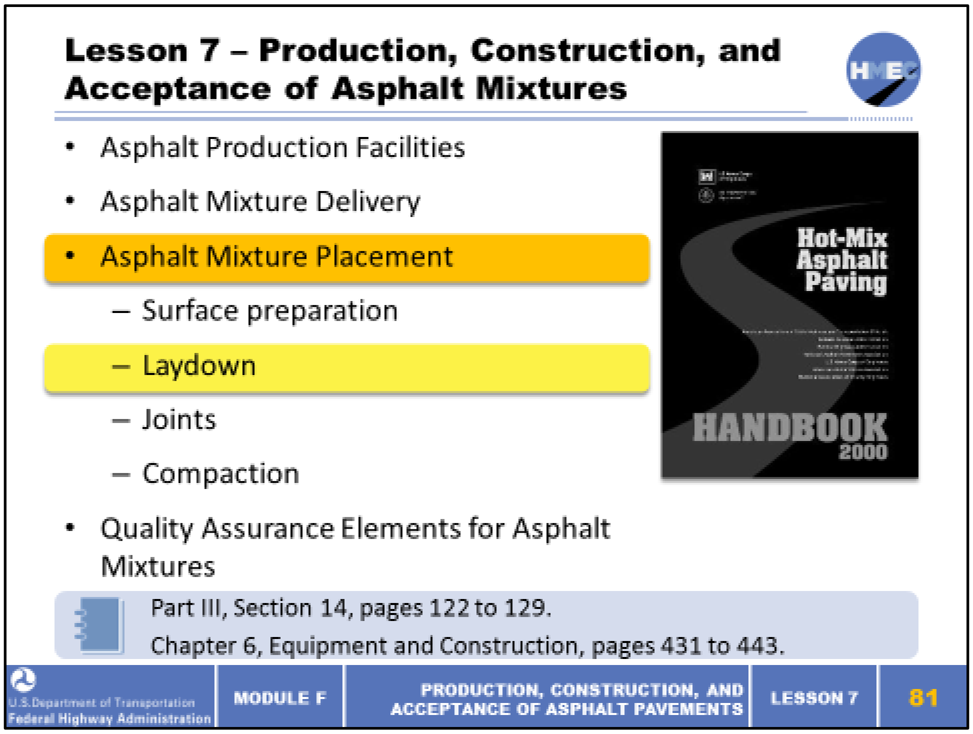
To date, there is no AASHTO test procedure and no consensus on any test to measure the bond between two asphalt lifts or layers to determine the effectiveness of the tack coat. However, some individual State agencies do have their own test procedures for measuring the bond between two asphalt layers. In addition, the National Cooperative Highway Research Program (NCHRP) has sponsored research studies to develop a test for measuring the bond between two HMA layers. The Louisiana Transportation Research Center (LTRC) is the prime contractor for one of these studies that should be completed in 2015. Results of the study will be reviewed by AASHTO prior to becoming a standardized test procedure.

Slide 80

Significant tracking of the tack coat can occur if the emulsion did not properly break before trucks are allowed on the tacked surface, placing a tack on a dirty surface, etc. Three options can be used to ensure the tack remains in place where it belongs to provide adequate bond between the two lifts or layers: (1) using material transfer devices which are located outside the paving lane (not shown in the slide), (2) allowing the emulsion to fully break prior to the delivery trucks driving over the tack – which can be impractical for the trucks delivering the mix to the paver, (3) use of a special paver where the tack (normally an emulsion) is placed directly under the paver just prior to the mixture being delivered to the screed, and (4) use of a trackless tack material.

An example of a paver that has the capability to apply the tack coat under the paver is shown in the slide (photograph on the right). There are relatively few pavers being used in the U.S., however, that have the capability to apply the tack under the paver so the delivery trucks do not track the material during paving operations. The use of trackless tack coats are becoming more commonly used because of the tracking issue. The trackless material is more expensive than traditional paving grade asphalt and emulsions that have been used for tack coats.

Slide 81

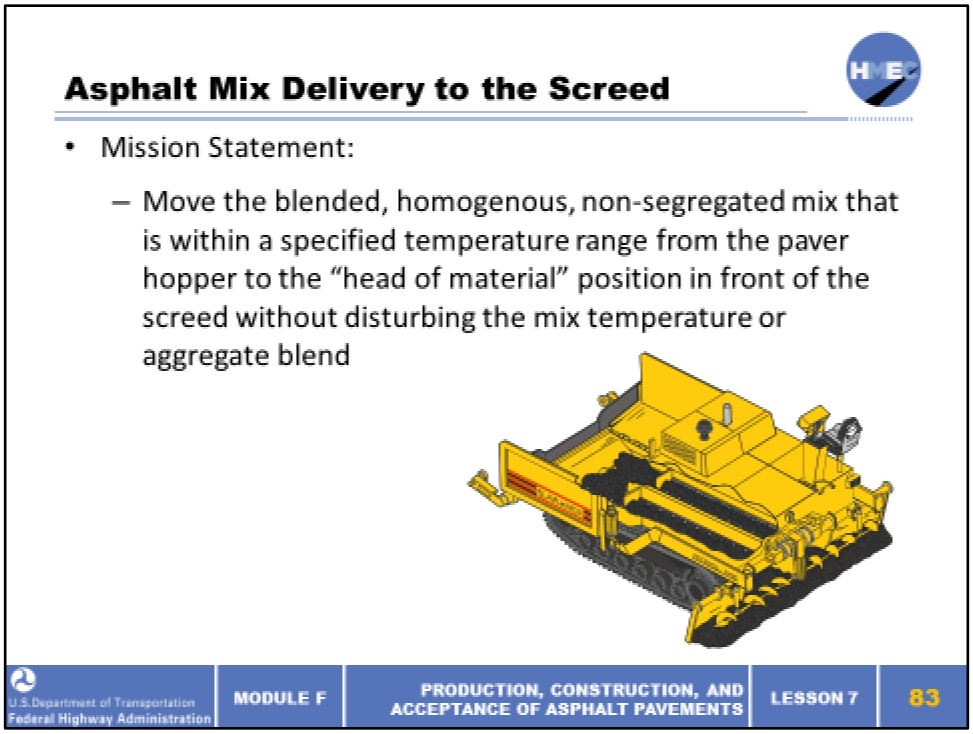


Pages 122 to 129 of Part III, Section 14 in the *Hot-Mix Asphalt Paving Handbook* is focused on mixture placement through compaction. In addition, pages 431 to 443 in Chapter 6 of the NCAT textbook provide additional information on asphalt pavers.

Slide 82

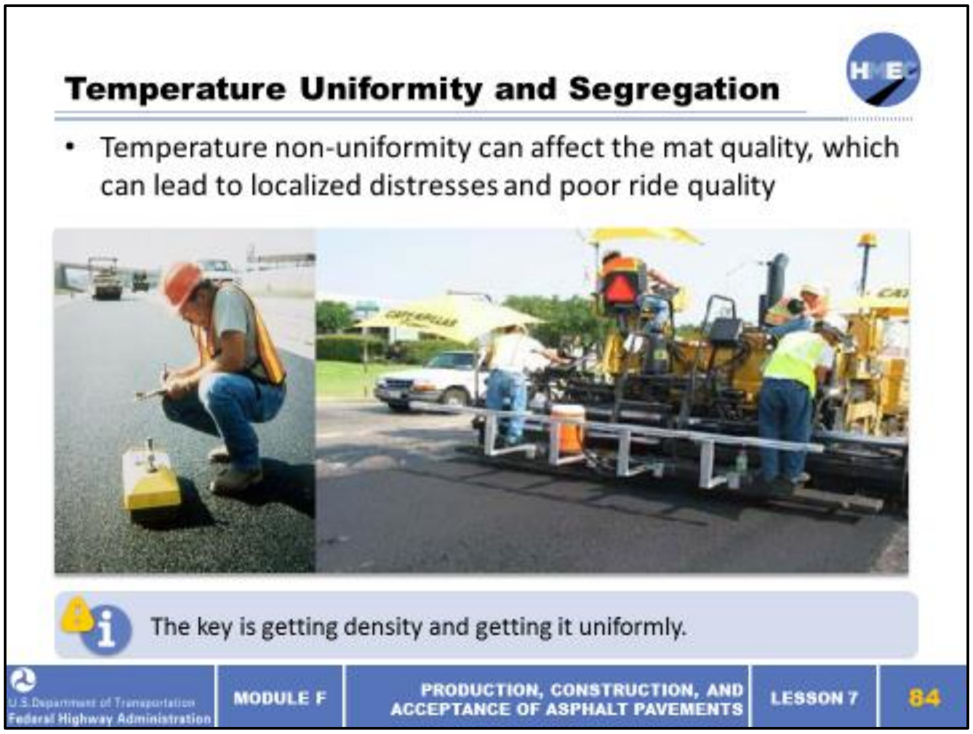
This slide shows two photographs: the upper photograph is of a paver being used in the 1930s and the lower one of a paver being used in the modern day time period, 2000s.

Slide 83



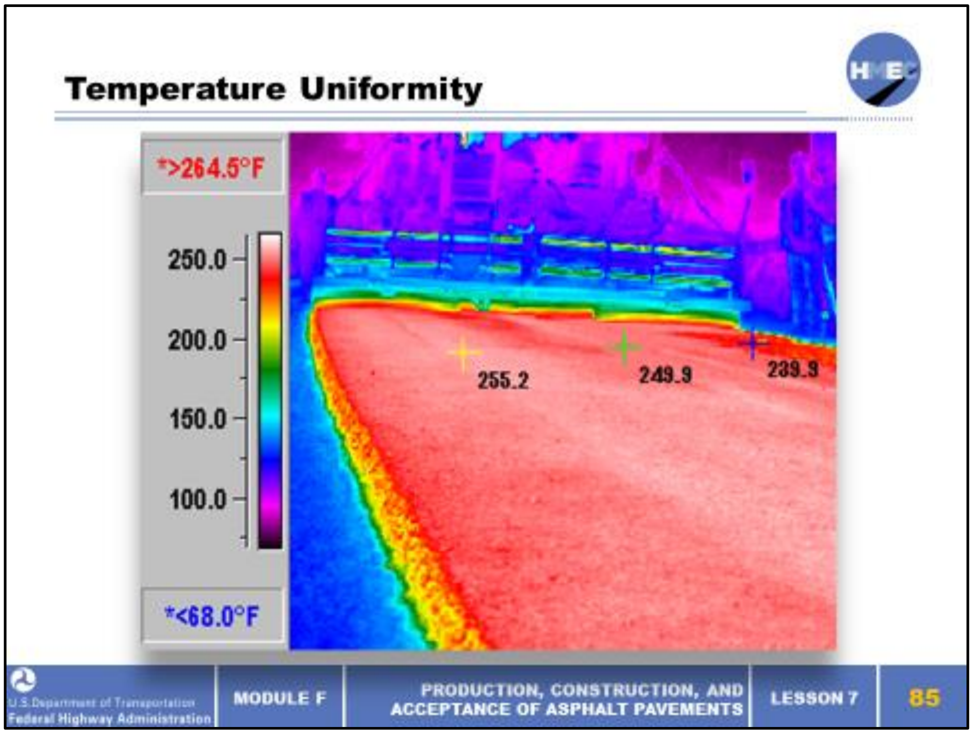
When delivering the asphalt mix to the screed, move the non-segregated, homogeneous blend of HMA within a specified temperature range in the paver hopper to the "head of material" position in front of the screed without allowing a drastic change temperature or aggregate blend.

Slide 84



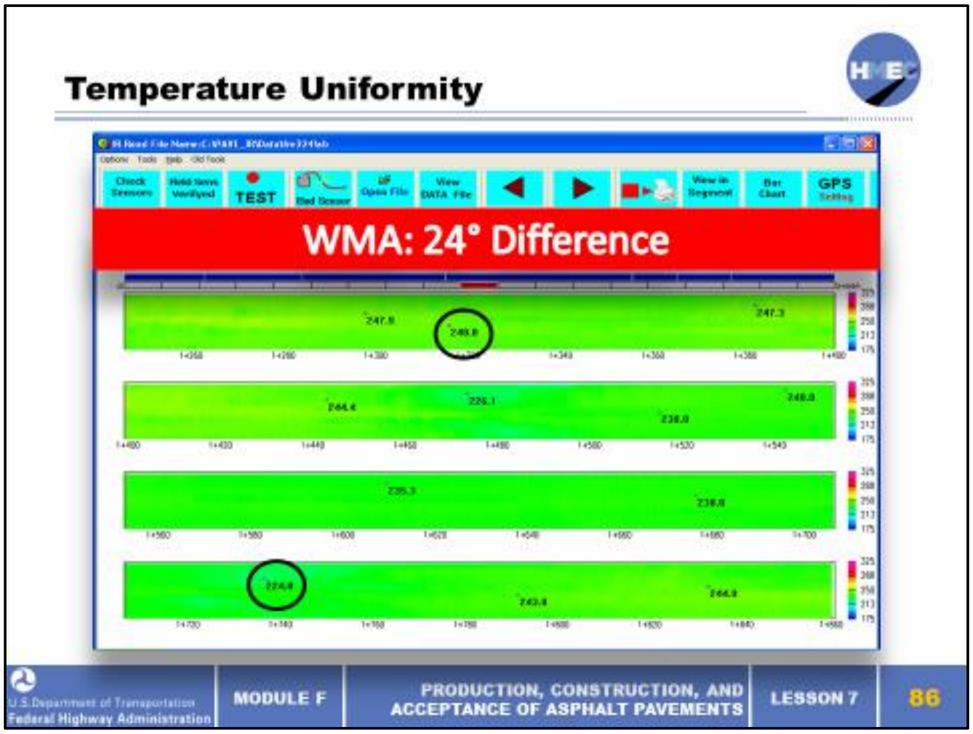
Temperature non-uniformity can and does affect mat quality, and can lead to localized distresses and poor ride quality. The importance of density and air voids or percent compaction based on the maximum theoretical specific gravity was discussed in Lessons 5 and 6.

Slide 85



The temperature ranged from 240 to 255 °F and is considered to be uniform. In other words, this is the goal.

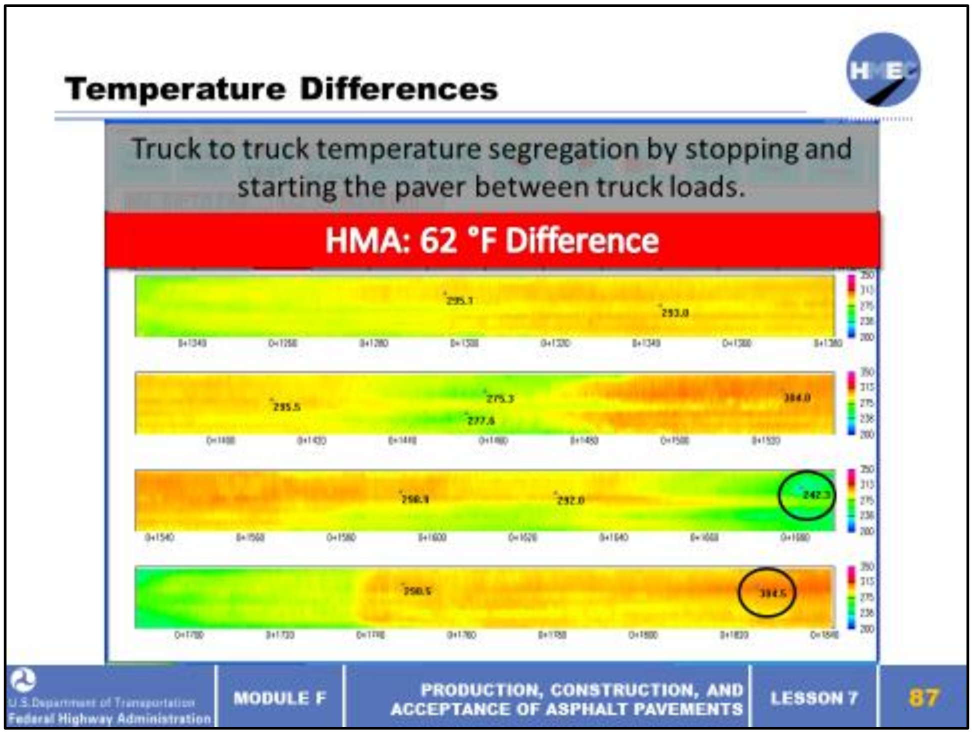
Slide 86



These thermal images were captured during WMA placement. The WMA placement was carried out during a field trial project at Lufkin, Texas in February through March, 2008. A PG 64-22 binder was used in this study for a mat thickness of 1.5 in.

Each image represents about 150 ft. The difference between the maximum and minimum temperatures is about 24 °F. Compared to HMA, the temperatures are relatively milder and more uniform. The temperature differential is lesser for WMA because the WMA is closer to ambient temperatures during construction.

Slide 87



Some agencies (for example, Texas and Washington) use the infrared camera or infrared bar for determining where to take the cores for density measurements; i.e., a biased sampling plan that is permissible to identify localized defects in the mat.

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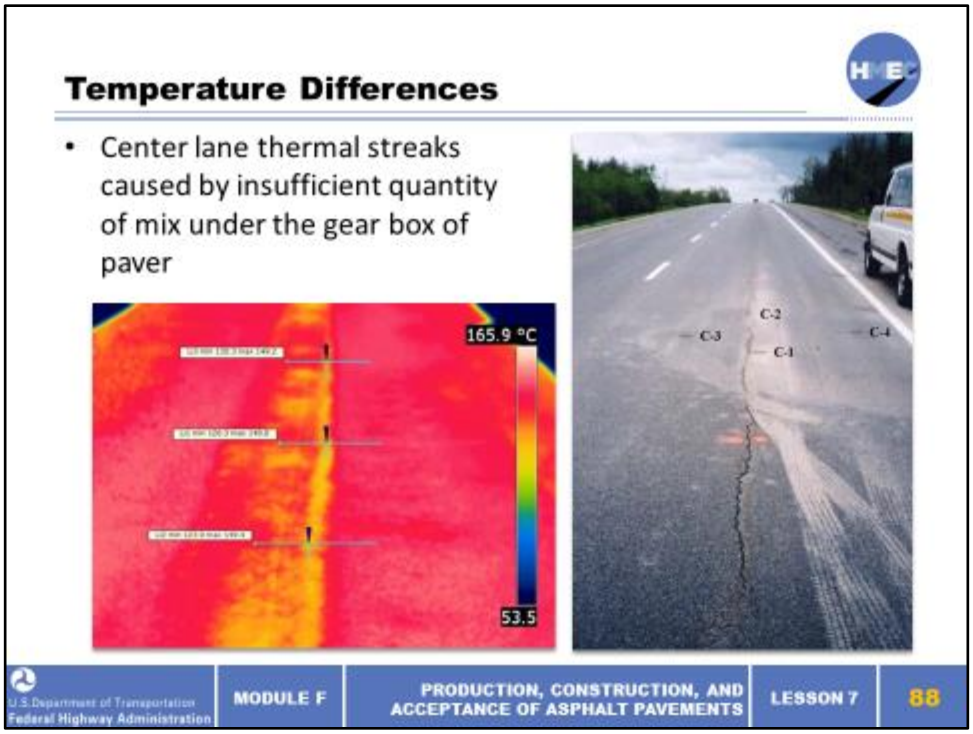
Slide 88



The cooler area along the longitudinal streak will result in a cracks along that thermal streak because the density is lower and more susceptible to cracking. No agency in the US is known to identify and use these cold thermal streaks as part of their acceptance plan. However, thermal streaks commonly result in cracks and crack deterioration causing the pavement or HMA mat to be rehabilitated prior to reaching its design life or expected service life.

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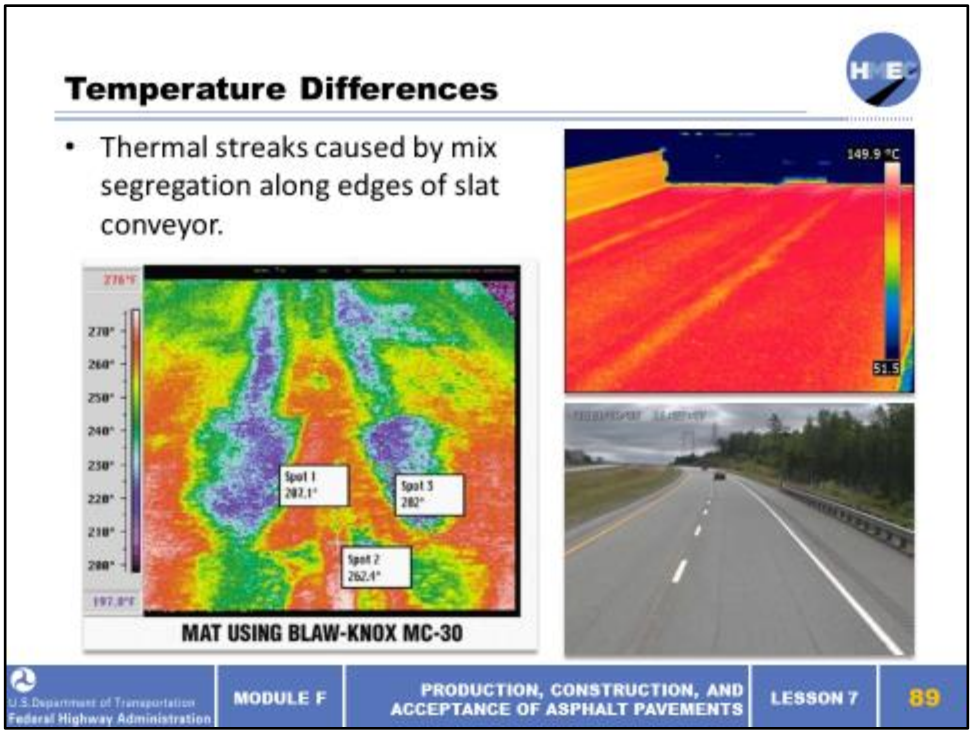
Slide 89



The two photographs shown on the right side of the screen show two much less severe thermal streaks and two longitudinal cracks in the same location as the thermal streaks. Eventually these cracks will have to be maintained depending on the severity of the thermal streak in terms of temperature differences and whether the streaks are caused by longitudinal segregation at the surface of bottom of the lift.

Slide 90

The dark lines or areas along the centerline include water that has yet to dry from a rainstorm. These longitudinal areas have high air voids that will trap and retain water.

Slide 91

The location of these two longitudinal lines with segregation are along the outside of the augers and along the outside edge of the slat conveyor of the paver.

Slide 92

The insufficient amount of mix in this area allows the mix to cool more rapidly than in other areas with a sufficient amount of mix and will exhibit density because of the lower amount of mix and more rapid cooling of the mix.

Slide 93

Further information on this topic is presented in the *Hot-Mix Asphalt Paving Handbook*, 2000, Section 15 (pages 130 through 137).

Pavers come with different track drive systems. The tracks spread the weight of the paver over a larger area and are beneficial when placing lifts over softer materials or layers. However, remember under an earlier section of this lesson: in order for the pavement or HMA mat to provide adequate performance, the foundation layers must be stiff and resistant to deformation.

Track drive systems are typically used or more effective when paving on steep grades. The tracks can be all steel, steel with rubber pads, or flexible bands with steel shoes and rubber pads. Track drive systems are also used more effectively and preferred when laying wide and thick lifts.

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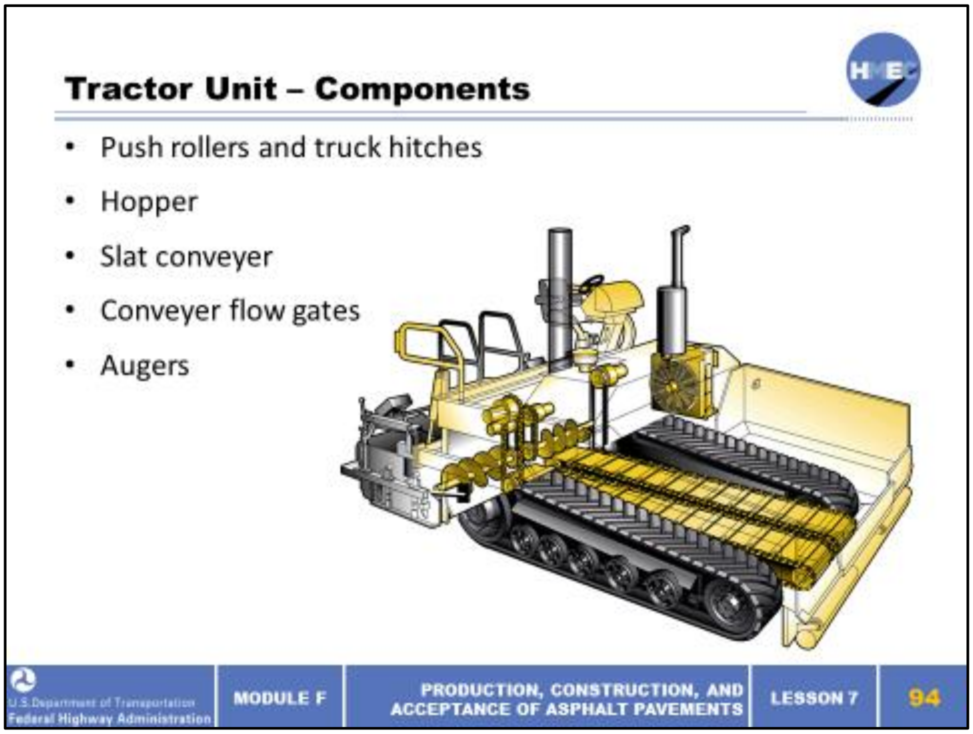
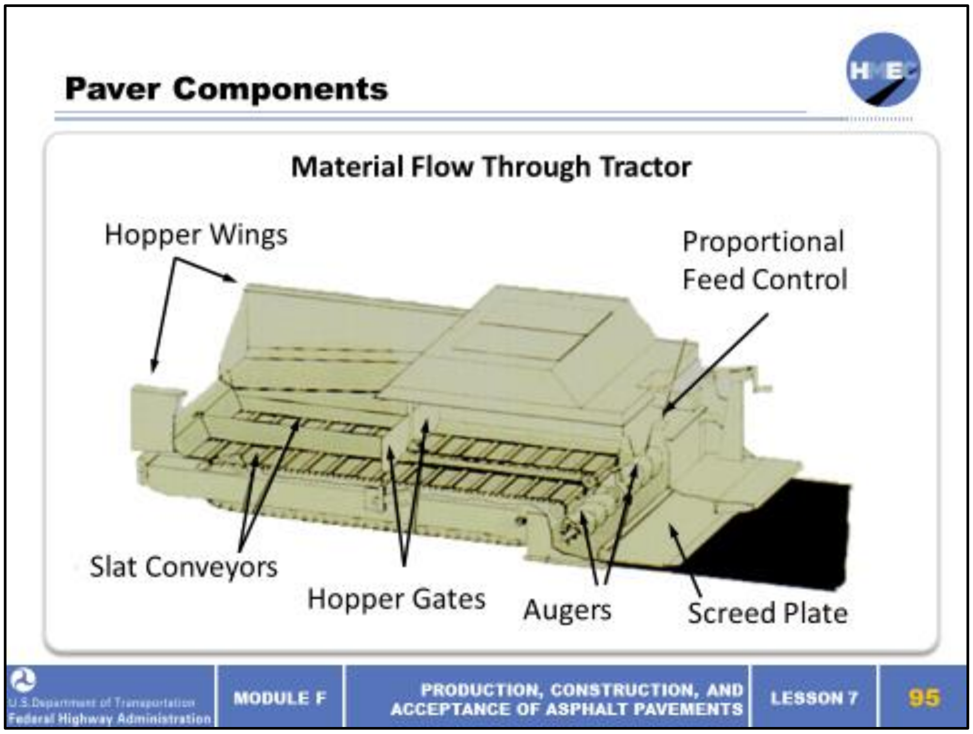
Slide 94



The tractor unit provides mobility and supports the hopper where material is delivered and conveyed back to the screed.

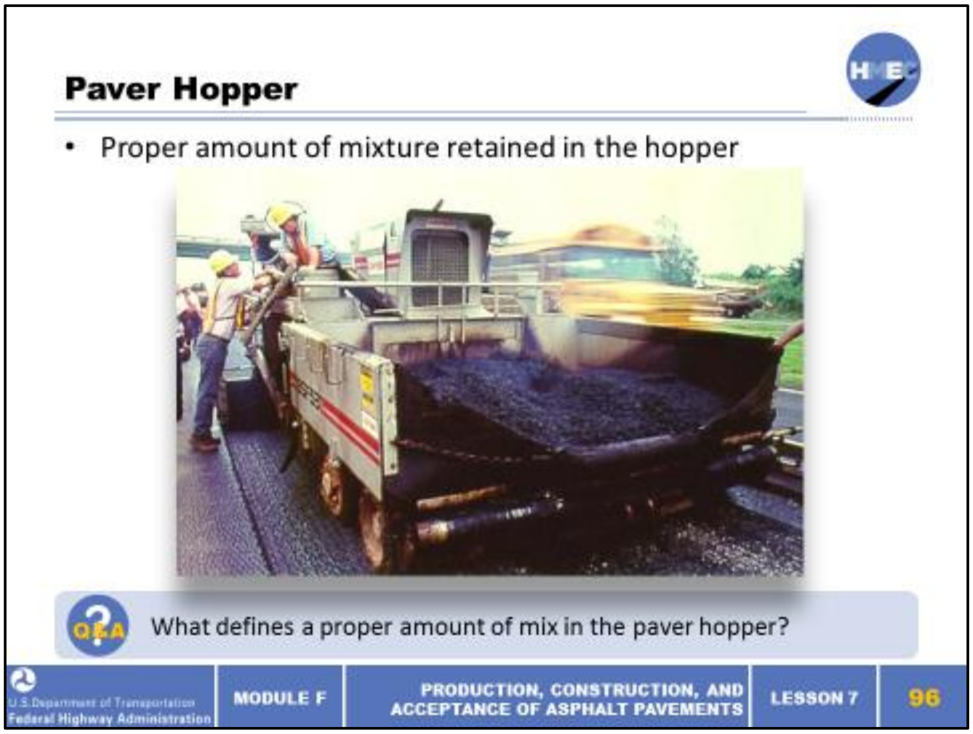
Slide 95



The goal of the material feed system is to get a constant head of material in front of the screed.

If the material feed system is set and operating properly, the slat conveyors and augers on each side of the paver will rarely shut off. This is accomplished by setting the proper position on the hopper flow gates and using the correct speed on the slat conveyors and augers. If the conveyors and augers are set at a constant speed (only on or off), the flow gate settings control the flow. For variable speed augers and conveyors, material flow is controlled by all three.

Slide 96



A proper amount of mixture should be retained in the hopper.

Slide 97

Two important observations for this paver are the torn and ripped flashing and the blades placed in front of the drive system.

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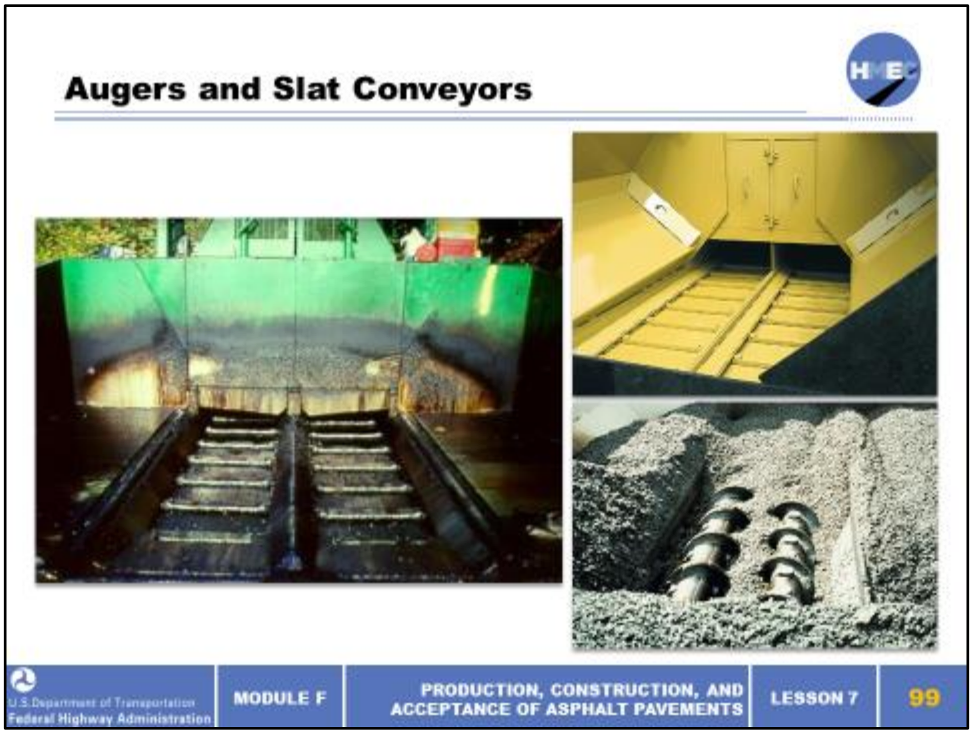
Slide 98

When this process is followed, a clear pattern of segregation is visible on the mat defined as truck to truck segregation. It essentially appears at the ends of each truckload. The inspector on the project can simply observe the truck to paver operation and determine whether there will be some segregation behind the paver. More importantly, the inspector should look inside the truck bed to see if it has been loaded properly at the plant—before the truck bed is raised to break the load and make sure it is against the tailgate prior to releasing the gate or discharging the mix into the paver hopper.

It is better practice to balance plant production, trucking, and laydown speed such that a minimum material head is maintained in the paver hopper at all times and never fold the wings until the end of the day in creating a transverse joint where that mix will be removed and wasted.

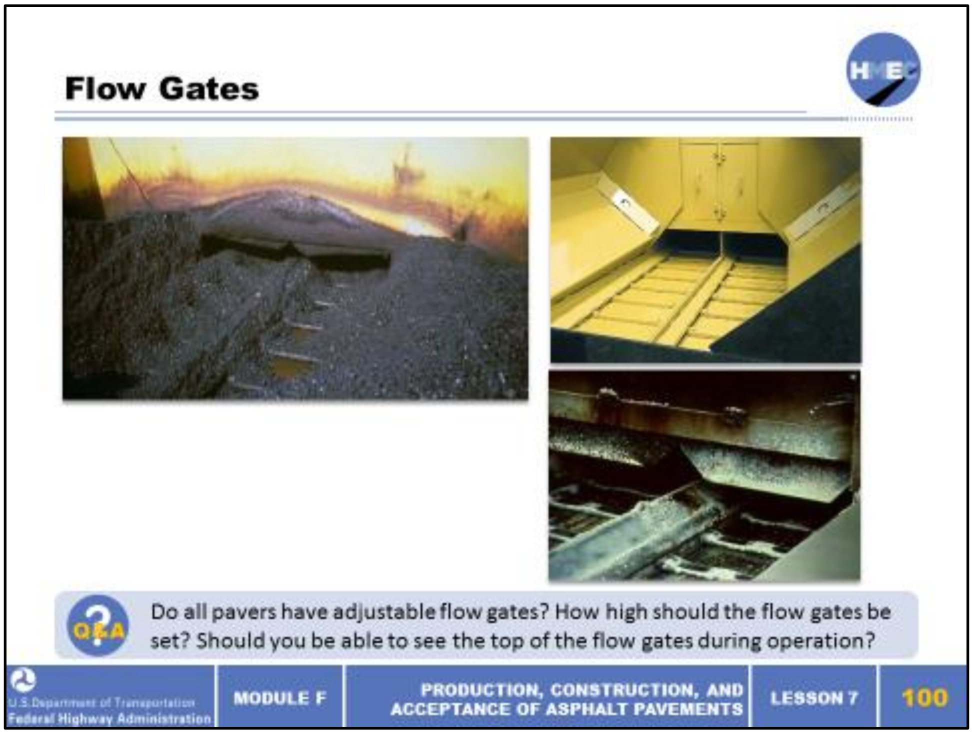
In some cases, it will be necessary to fold the wings between the truck loads to keep the paver moving at a constant speed as much as possible. In this case, proper folding of the hopper wings is important. When the truck exits or leaves the paver and is clear of the machine, begin raising the hopper wings slowly (if allowed by the inspecting body) while the paver is still moving forward. Continue raising the hopper wings until lift cylinders are fully extended. At that point, stop the paver and lower the wings and wait for the next load. Hoppers should only be raised often enough to keep HMA from cooling below good placement temperatures and so that the top of the flow gates remain embedded in HMA.

Slide 99



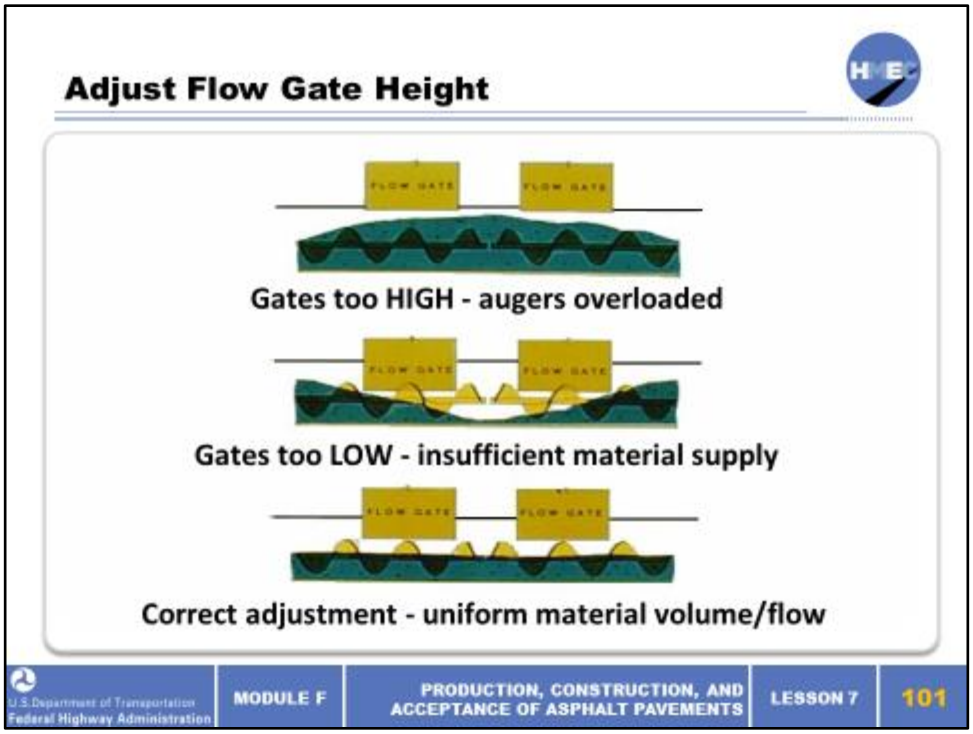
On most newer pavers, the conveyor operates independently of the speed of the paver, and on some pavers, the conveyors are independent of the speed of the auger. On some newer pavers, the slat conveyor has been replaced with a screw conveyor system as shown in one of the photographs in this slide.

Slide 100



The bottom photo on the right: Some paver manufacturers have deleted these adjustable gates altogether from their designs and the flow gates are set at a specific height. The flow of the material is controlled by the speed of the slat conveyor or screw augers in the hopper, which are tied to the speed of the augers in the auger chamber.

Slide 101

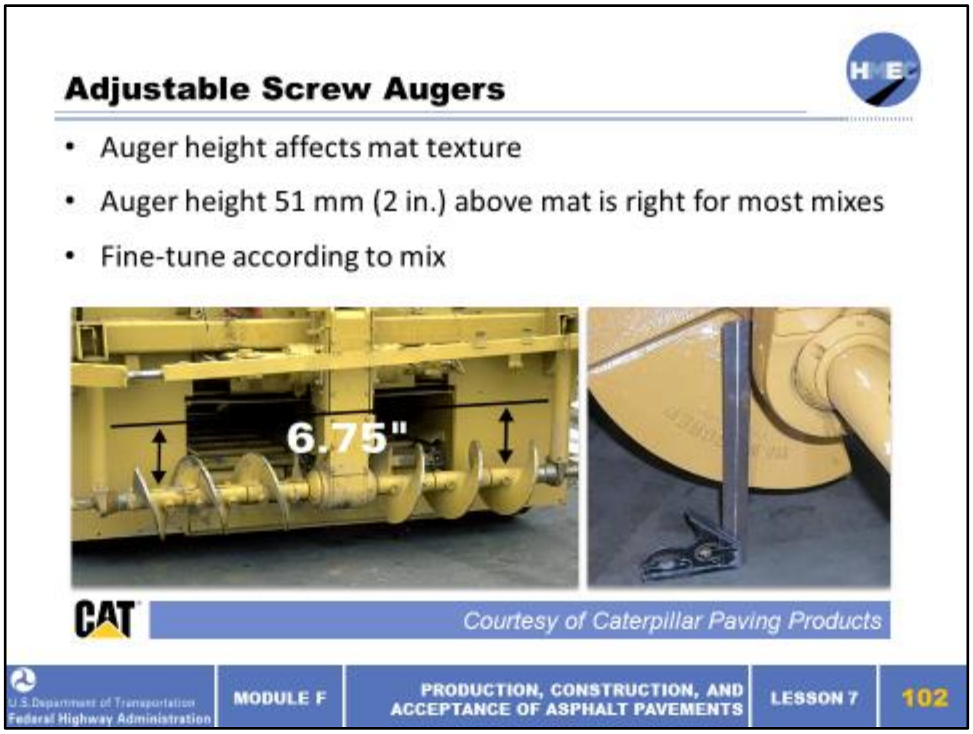


This slide shows a schematic of what happens to the amount of mix in the auger chamber (head of material in front of the screed) if the gates are set too high or too low.

- The goal is to deliver a constant amount of mix into the auger chamber that can be distributed equally throughout the auger chamber via the screw augers—create a uniform head of material and keep it constant. The lower schematic in the slide illustrates this condition.
- If the flow gates are set too high, more mix will be located in the center of the auger chamber which will become over filled, as shown in the upper schematic in the slide. In addition, if extensions are being used, some mix may need to be shoveled to the corners of the end plates.
- If the flow gates are set too low, more mix will be located in the auger chamber along the outside edges of the auger chamber and there will be an insufficient amount of mix in the center of the auger chamber. The middle schematic in the slide illustrates this condition.

Simply watching the paving operating and looking into the auger chamber, an inspector can determine whether the gates are set incorrectly.

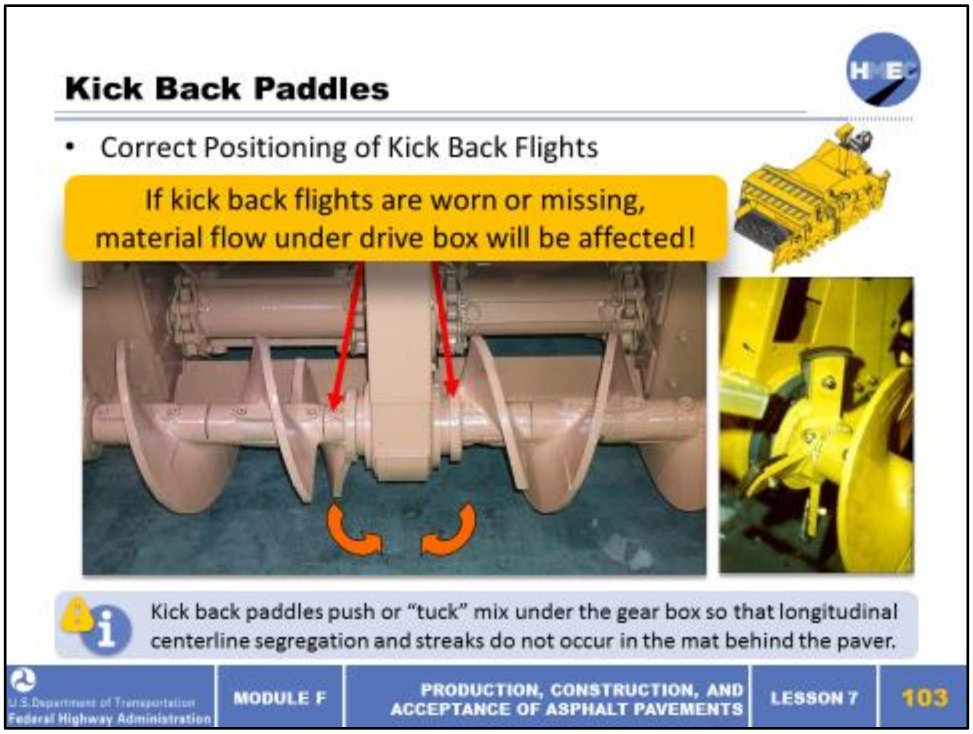
Slide 102



The mix is brought from the hopper by the slat conveyors and discharged into the center of the auger chamber. The augers spread the mix laterally in front of the screed.

As with the slat conveyors, the augers operate independently. The augers can be raised or lowered depending on the type of mix placed. The height of the augers are normally set about 2 in. above the height of the screed. This option helps reduce centerline longitudinal segregation. After paving the height of the augers can be fine-tuned in placing a very uniform mat surface condition.

Slide 103



These kick back paddles can be installed in reverse in which case they will push mix away from the gear box and an insufficient amount of mix will be placed under the gear box. This condition is usually obvious behind the paver. One of the slides included at the beginning of this section illustrated the mat condition when the kick back paddles are installed in reverse. These kick back paddles, however, are a high-wear item and must be replaced periodically. Once they start to wear, the typical centerline streak behind the paver under the gear box will become more and more noticeable as the paddles continue to wear. As the condition worsens, the probability of a crack occurring along the center of the mat increases because density of the mat along this longitudinal streak is much lower than in other areas of the mat.

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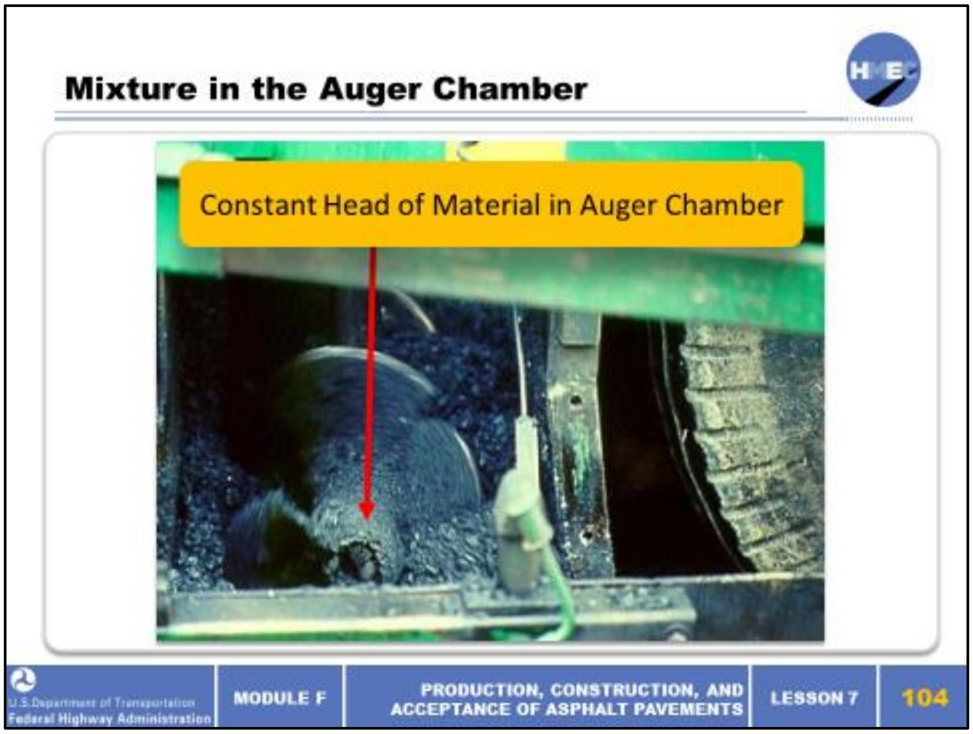
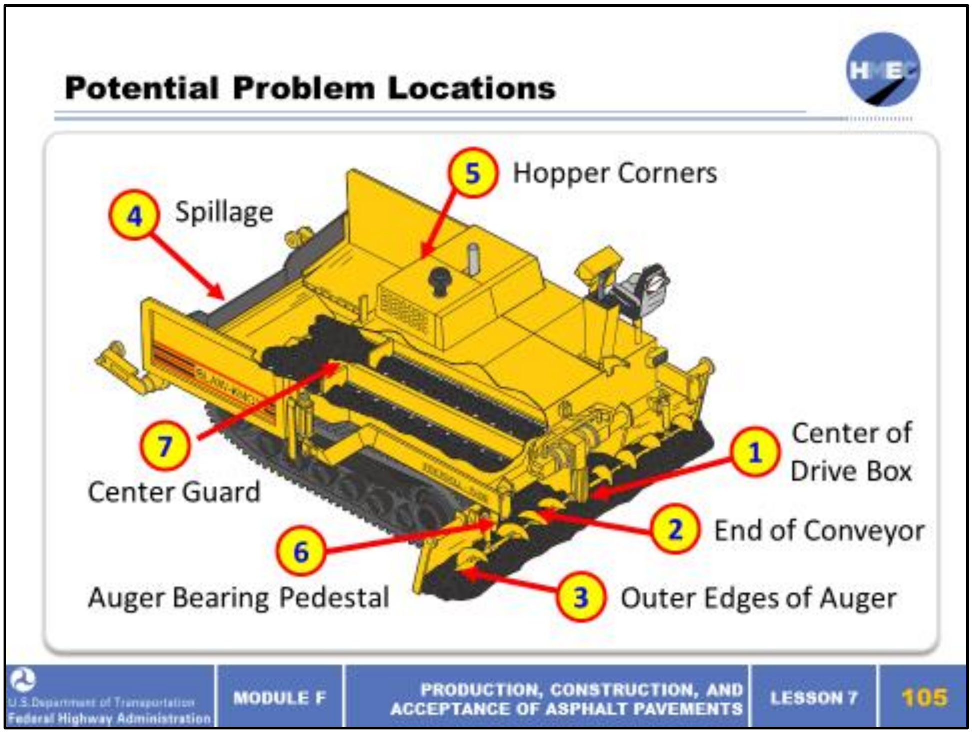
Slide 104



The flow gates may need adjusting to achieve the desired balance and uniformity. The inspector of a project can simply walk along the side of the paver and observed when the augers are running, how the head of material changes throughout inside the auger chamber, and how the head of material changes as the paver moves forward. These observations will indicate if achieving a smooth mat or uniform density will be an issue.

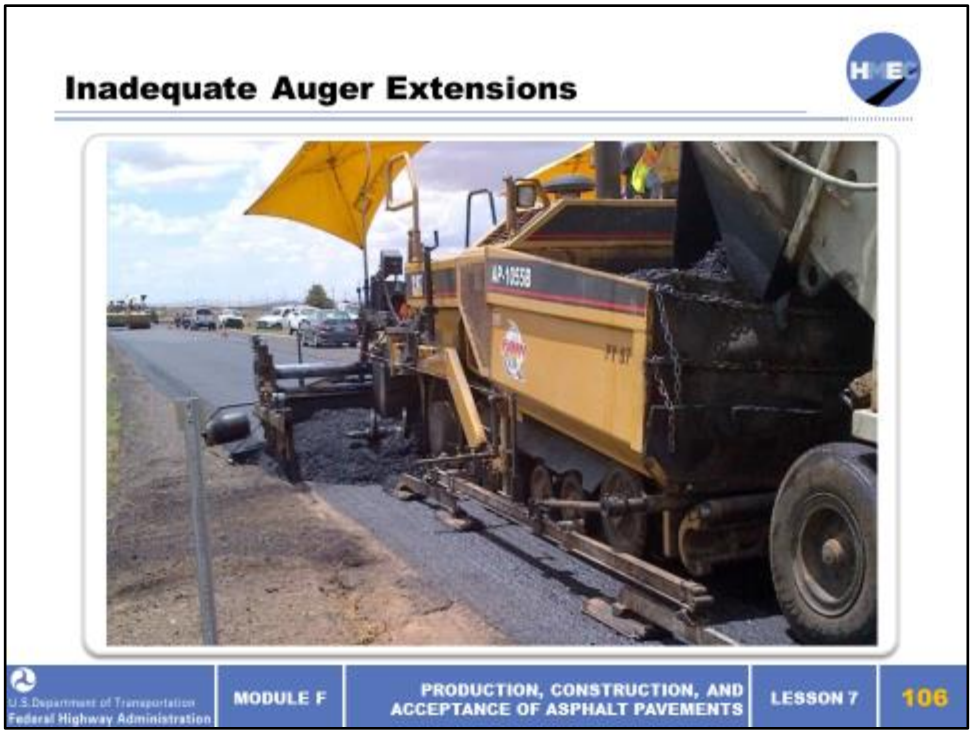
Slide 105



Problems or defects can be generated as early in the production process as the stockpiling and loading operations at the plant, as discussed previously, or as late as in the laydown process. It may also be generated in any of the processes in between these.

The key to reducing segregation is maintaining consistent operations (production, delivery, and laydown).

Slide 106

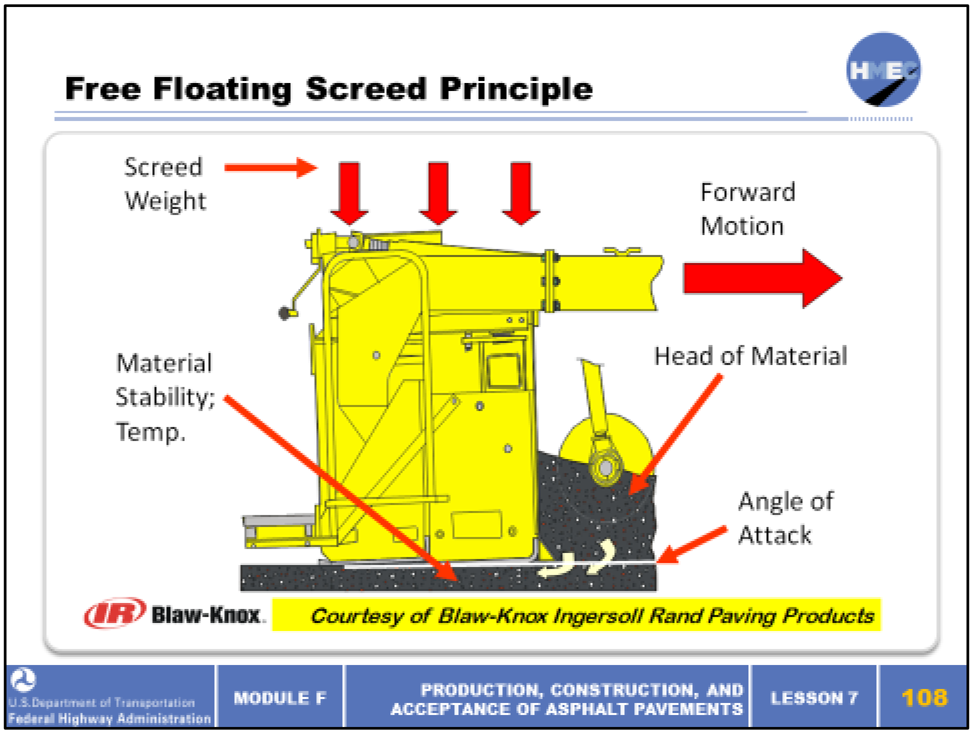


Segregation is difficult to identify from visual observations of the mat behind the screed, especially if the longitudinal segregation occurs at the bottom of the mat. A device that can assist in identifying longitudinal segregation is an infrared camera or infrared bar, which were discussed and illustrated earlier within this lesson.

Slide 107

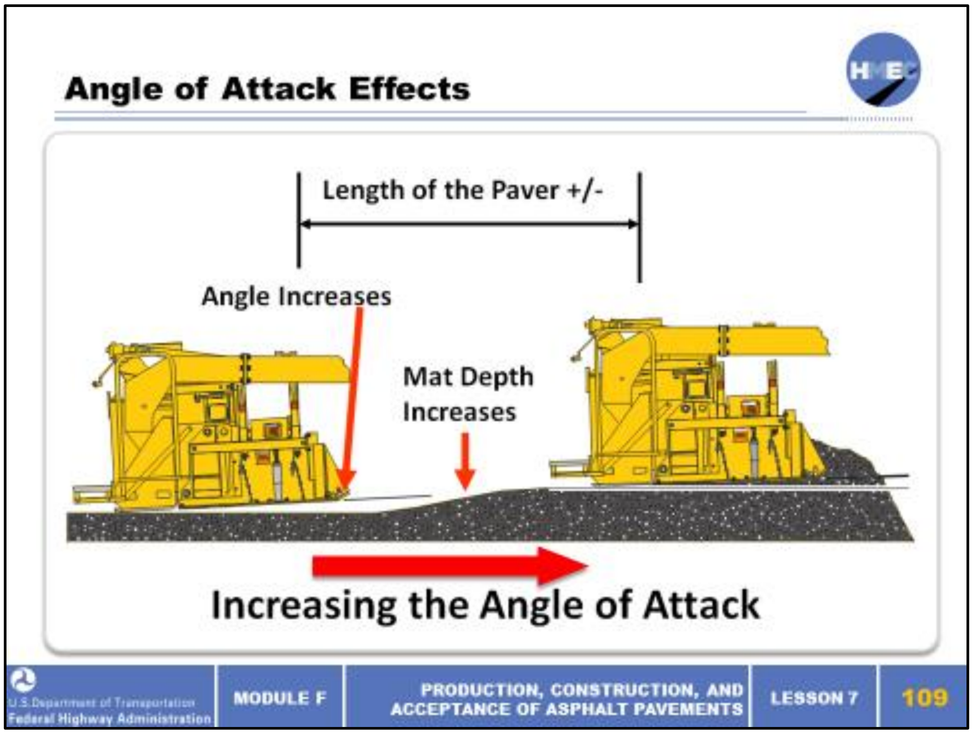
Further information on this topic is presented in the *Hot-Mix Asphalt Paving Handbook,* 2000, Section 15 (pages 137–150).

Slide 108



The forces acting on the screed are mentioned and discussed on pages 139 to 141 in the *Hot-Mix Asphalt Paving Handbook,* 2000.

Slide 109



The angle of attack of the screed is defined as the angle of the screed relative to a horizontal line. As that angle changes, the amount of mix placed changes, which is illustrated in this schematic and in the next schematic.

Increasing the angle of attack increases the thickness of the mat placed by the screed.

Slide 110

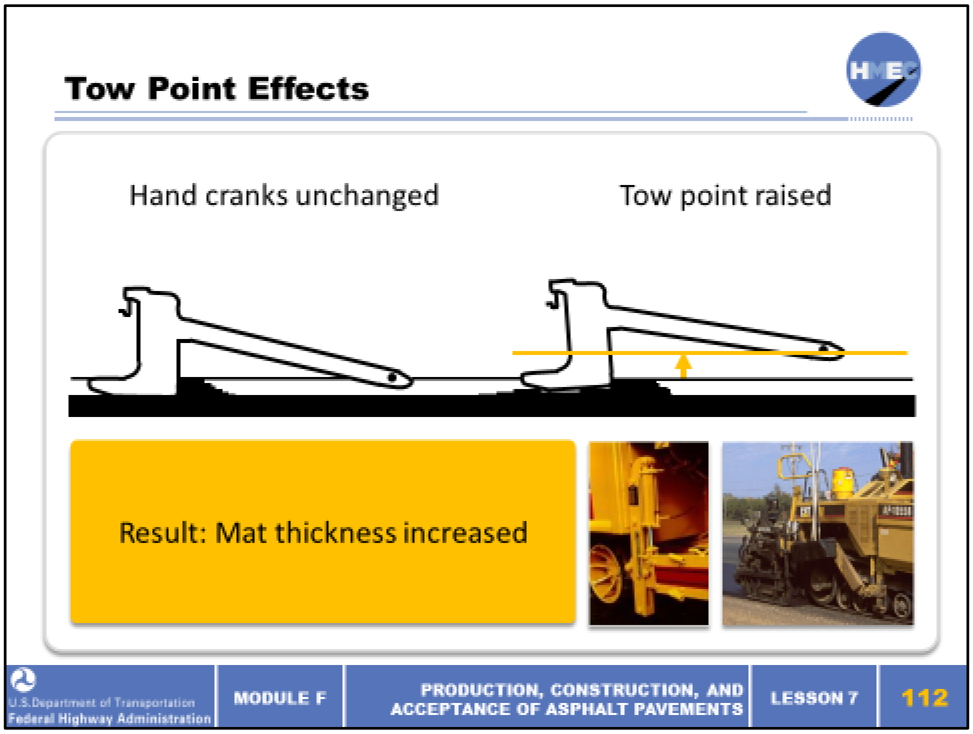
The screed arm (referred to as the tow or leveling arm) attaches the screed to the tow point on the tractor. As shown in the slide, the tow points are a pin connection where the screed is attached to the tractor unit. The tow point can be automatically raised or lowered as necessary as the paver passes over a rough surface with short or long wave lengths. A hydraulic cylinder on both sides of the tractor unit move the position of the tow point. The cylinders can be adjusted manually or automatically with a grade and slope control system.

The tow point is typically centered when paving begins, and then is adjusted as necessary.

Slide 111

Increasing the angle of attack of the screed by turning the thickness control crank increases the mat thickness. Decreasing the angle of attack of the screed by turning the thickness control crack in the opposite direction decreases the mat thickness.

Slide 112



Demonstrate how the tow points affect the mat thickness and the angle of attack of the screed by dragging a ruler or flat edge device along an uneven surface using a stringline or wireline.

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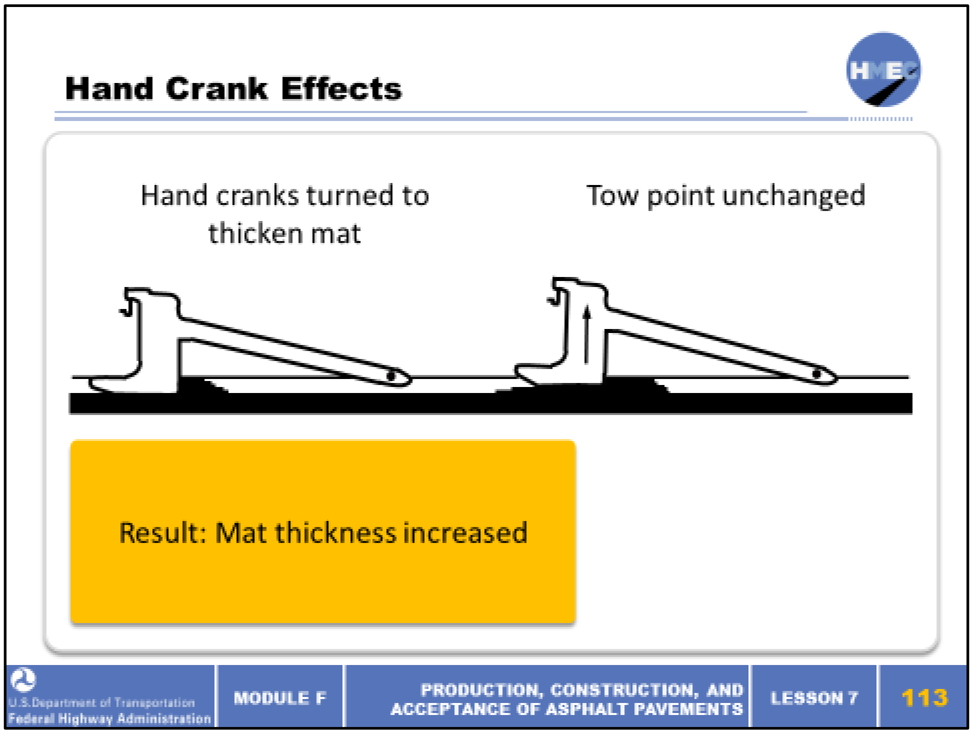
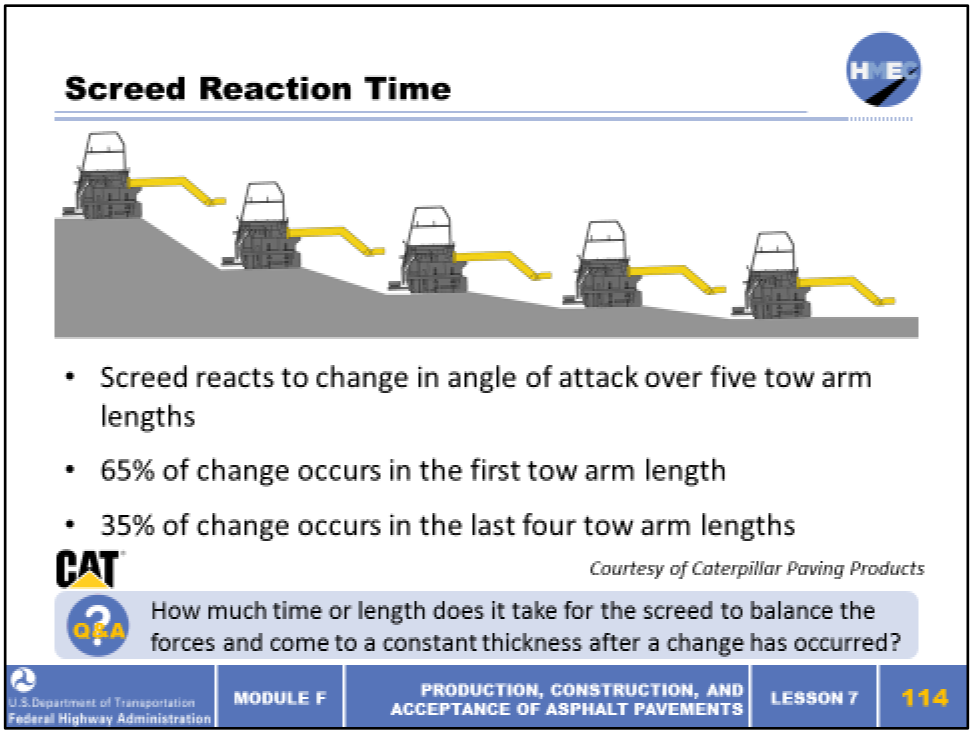
Slide 113



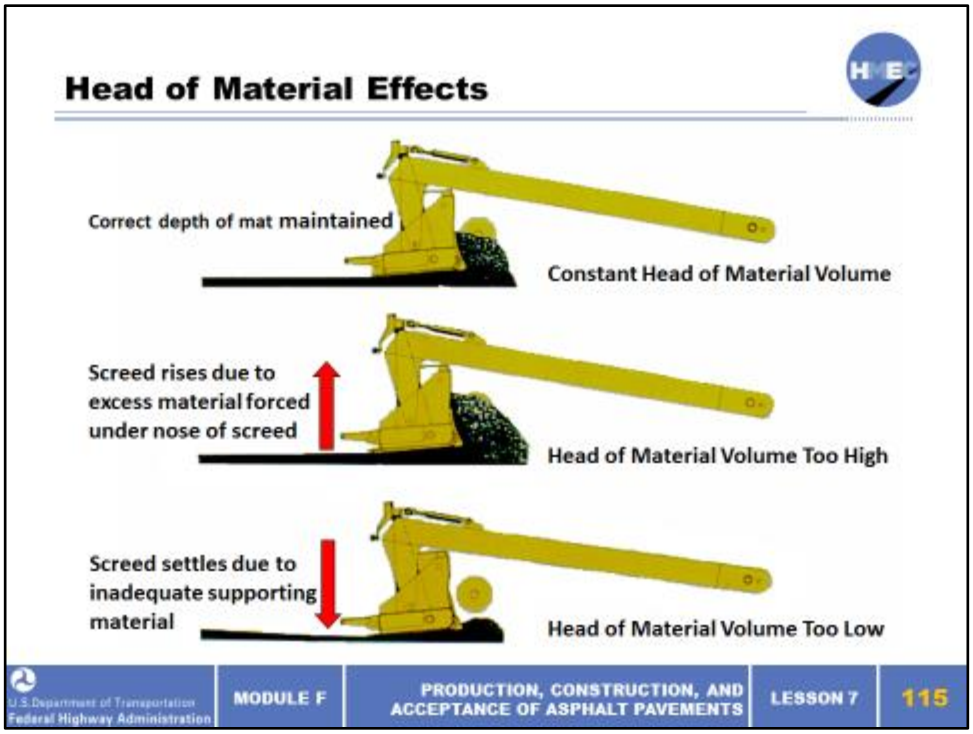
To increase depth, the front of the screed is pivoted up, and the screed rises by turning the thickness control crank. The change in thickness is not instantaneous, but instead happens over a length of paving. (This is discussed in detail later in this module.) This change occurs with no change in the tow point.

Slide 114



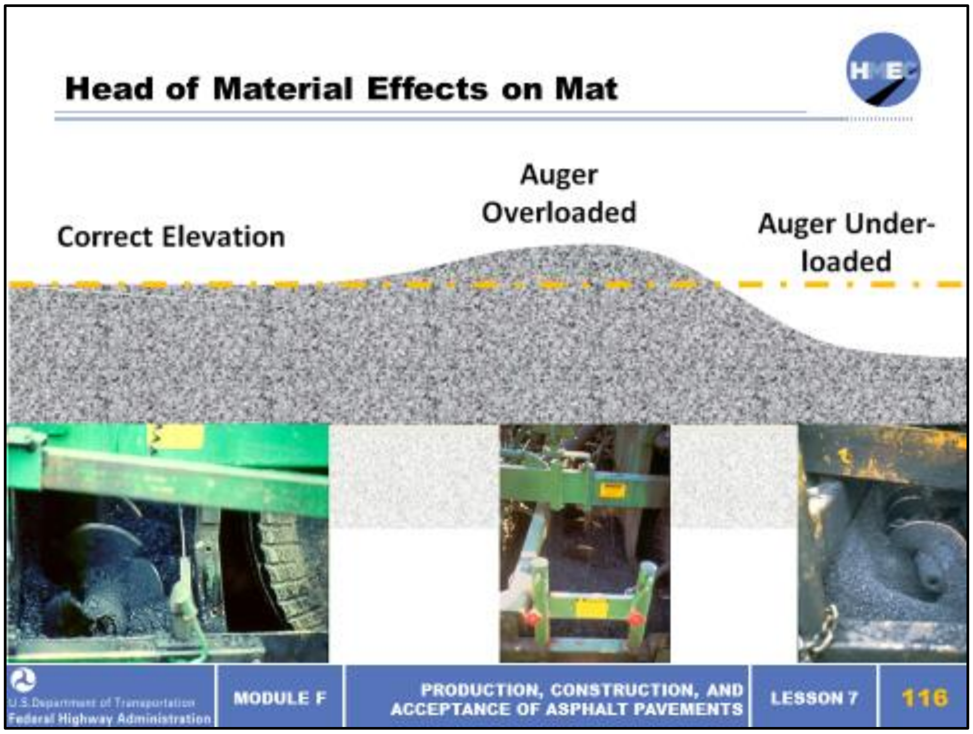
If a second change is made before the first one is accomplished, the first change will not be completed before the screed starts to react to the second adjustment. If the screed operator is continually changing or turning the thickness control crank, there is no way the final mat will be smooth. A good screed operator "sticks" the mat infrequently with a device that has been set to the planned mat, uncompacted thickness.

Slide 115



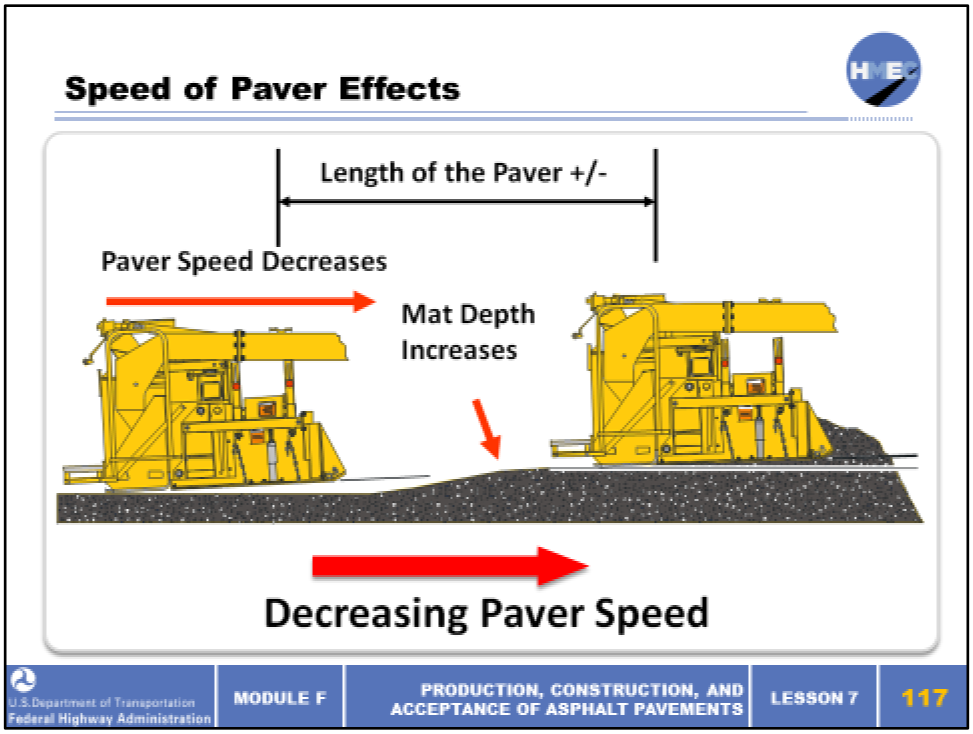
As discussed earlier, the ideal head of material covers the center of the screw augers to about the centerline of the augers. If head of material increases, then the force on the front of the screed increases, and the screed compensates by moving upward. Conversely, if head of material decreases, then the force on the screed decreases, and the screed compensates by moving downward. A varying constant head of material as the paver moves down the roadway builds in roughness or long wave lengths. As noted, walking along the side of the paver and observing the head of material can explain a rough mat at the end of the paving day.

Slide 116



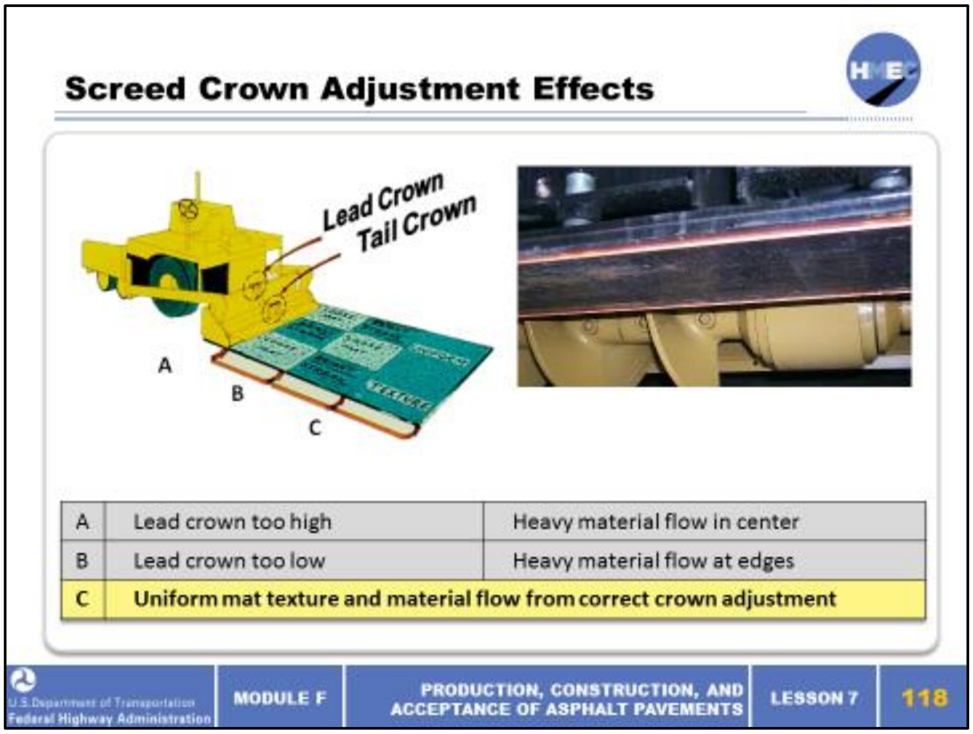
The head of material is the most important force acting on the screed. Some paving experts feel that 90–95% of paver-related problems can be solved by maintaining a uniform head of material during paving.

Slide 117



As paver speed decreases, the thickness of the mat increases, which is very similar to a water skier.

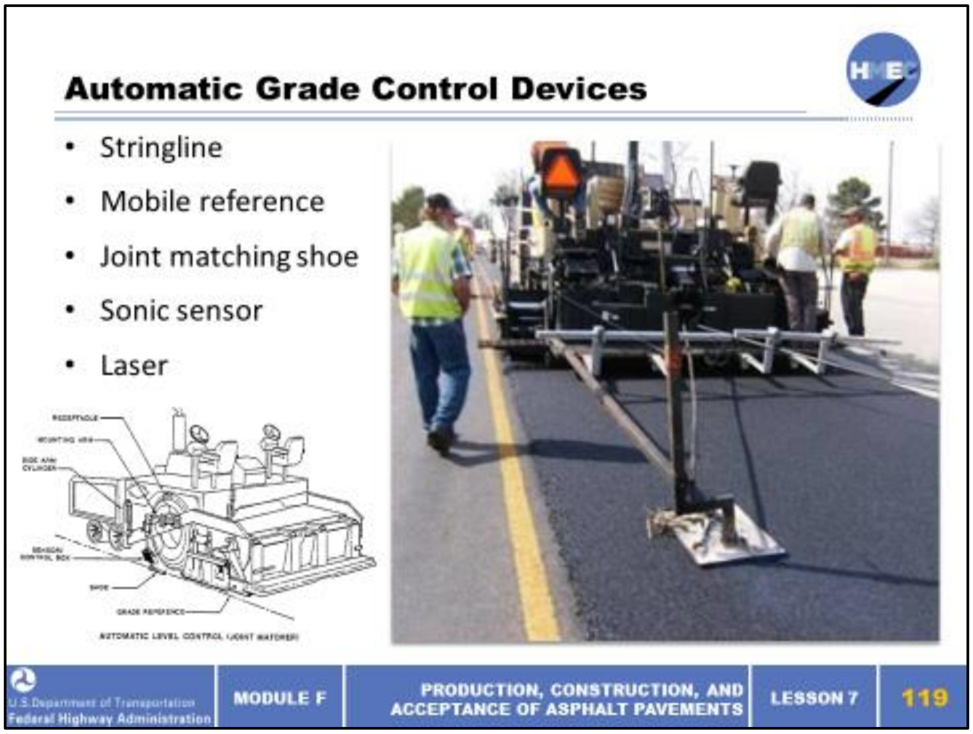
Slide 118



All pavers or screeds have a lead and tail crown and the combination of these two affects the texture of the mat. Typically, the lead crown is 1 to 5 mm ($^{1}/_{20}$ to ⅕ in.) more than the tail crown to achieve the desired results. Observing the mat texture behind the screed can be used to determine if the lead and/or tail crown have been incorrectly set, as shown in the schematic in this slide. A difference in texture will result from either–the loose mat texture is caused by the lower amount of material in that location.

• If the lead crown is too high, more material will be forced under the center of the screed, and the mat will have a shinier surface or mat texture there and the outside edges will have a loose mat texture surface.
• If the tail crown is too high, more mix will be forced under the screed at the edges. In this case, the shinier surface or mat texture will be located along the outside edges of the screed and the loose mat texture will be located in the center of the mat.

Slide 119



The grade control devices and automatic screed controls are mentioned and discussed on pages 152 to 161 in the *Hot-Mix Asphalt Paving Handbook,* 2000.

Slide 120

These floating beam systems come in various lengths and most contractors build their own. This type of mobile reference typically results in a very smooth pavement. It is more likely to ignore isolated changes in grade, such as a rock on the pavement, or localized small depression.

One manufacturer developed a mobile reference that bridges over the paver and refers to this system as a bridge ski. The lead beam senses the grade of the existing surface in front of the paver, while the trailing beam rides on wheels behind the screed, sensing the elevation of the newly placed mat. Intermediate beams connect the two reference beams, and the grade sensor rides on one of them. The bridge ski system is shown in the lower photograph in the slide.

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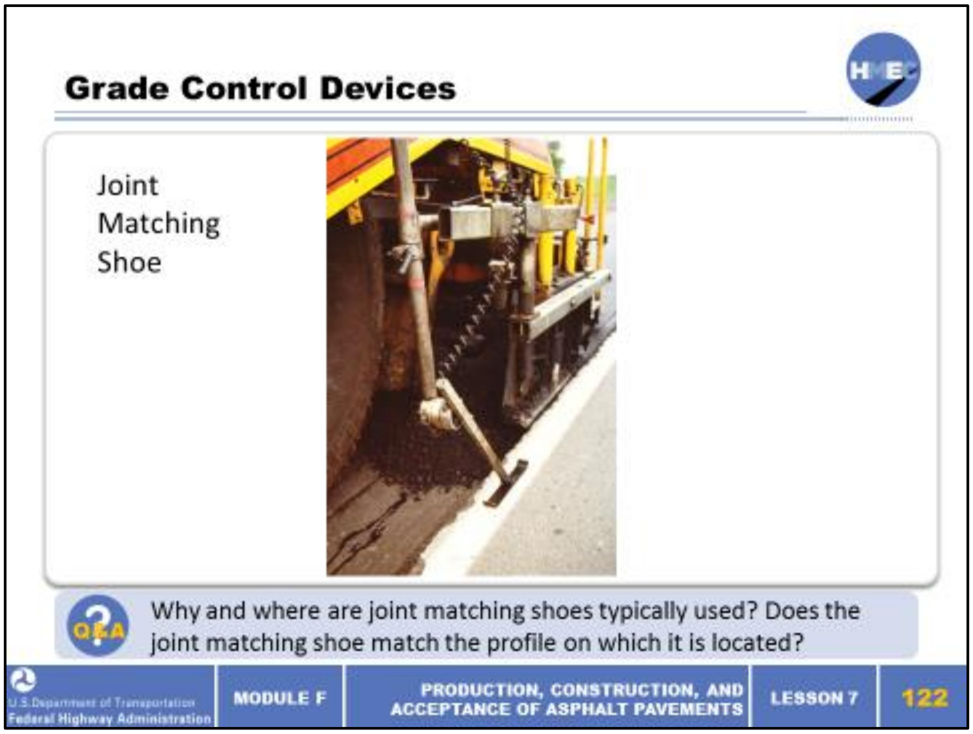
Slide 121

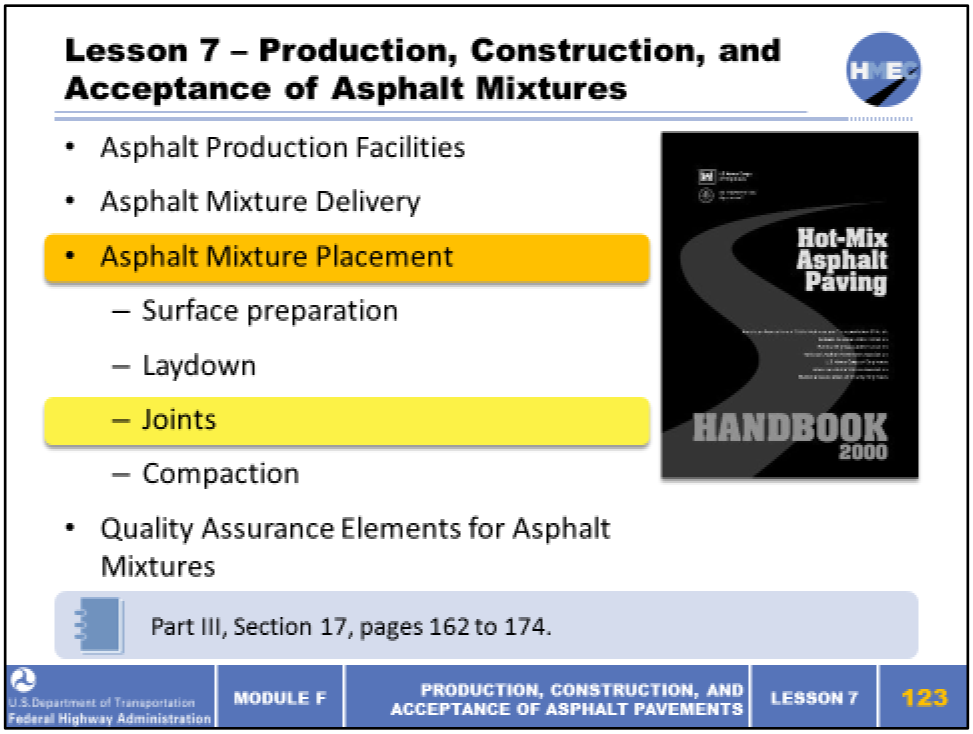
The photograph on the right side of the slide shows the combination of using ultra sonic sensors on the side away from traffic and the use of a floating beam on the side with traffic passing the paver.

Slide 122



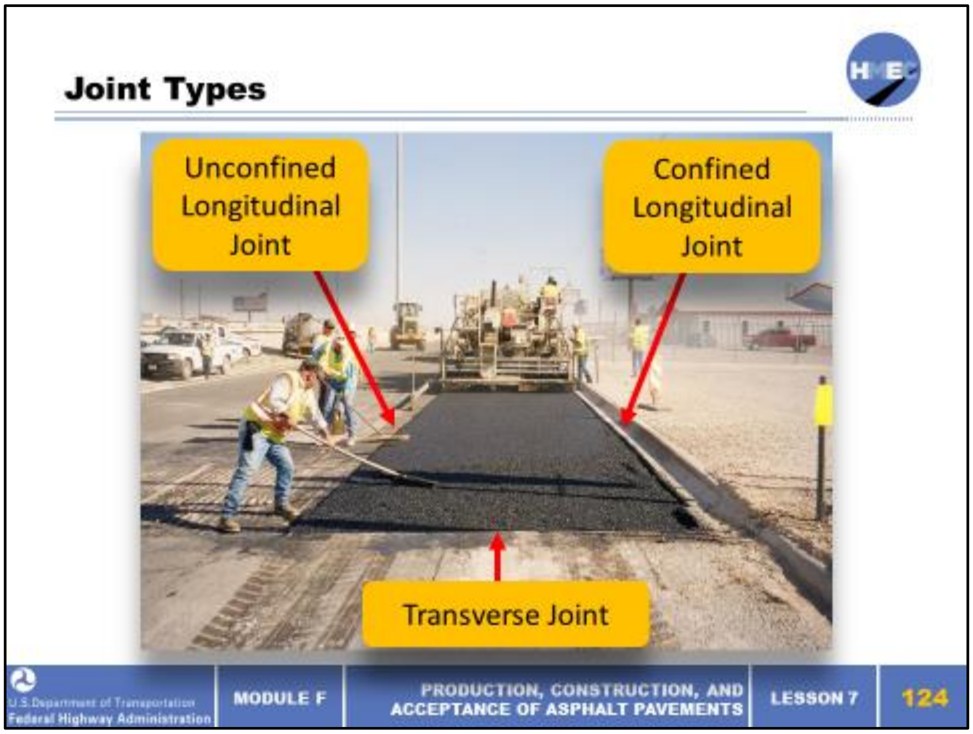
This reference consists of a short (0.3 m or 1 ft.) ski that rides directly on the adjacent surface or curb. For surface courses, the joint shoe is used to match an elevation across the longitudinal joint; however, the mobile reference still represents better paving practice resulting in a smoother surface.

Slide 123



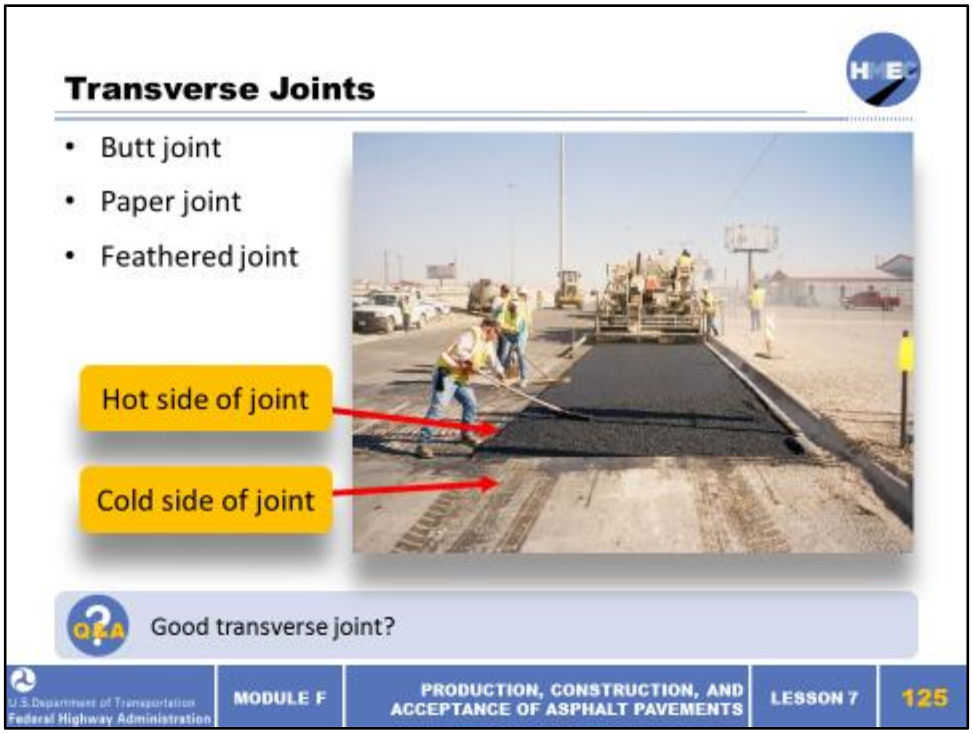
Pages 162 to 174 of Part III, Section 17 in the *Hot-Mix Asphalt Paving Handbook* is focused on good joint construction techniques for both transverse and longitudinal joints.

Slide 124



The longitudinal joints can be further grouped into an unconfined and confined longitudinal joint. The methods used to successfully construct and compact each type differ. The other important terminology related to joints is referred to as the "hot side" and "cold side" of the confined joint. This terminology will be explained under the discussion for both the longitudinal and transverse joints.

Slide 125



Initiate some discussion between the participants on whether the slides shows the construction of a good transverse joint. For this specific transverse joint shown in this slide, an incorrect height of the starter blocks was used and the thickness of the mat was lower than planned to match the elevation of the existing surface being pulled from. This required more mix to be added and raked into place, which is never good practice.

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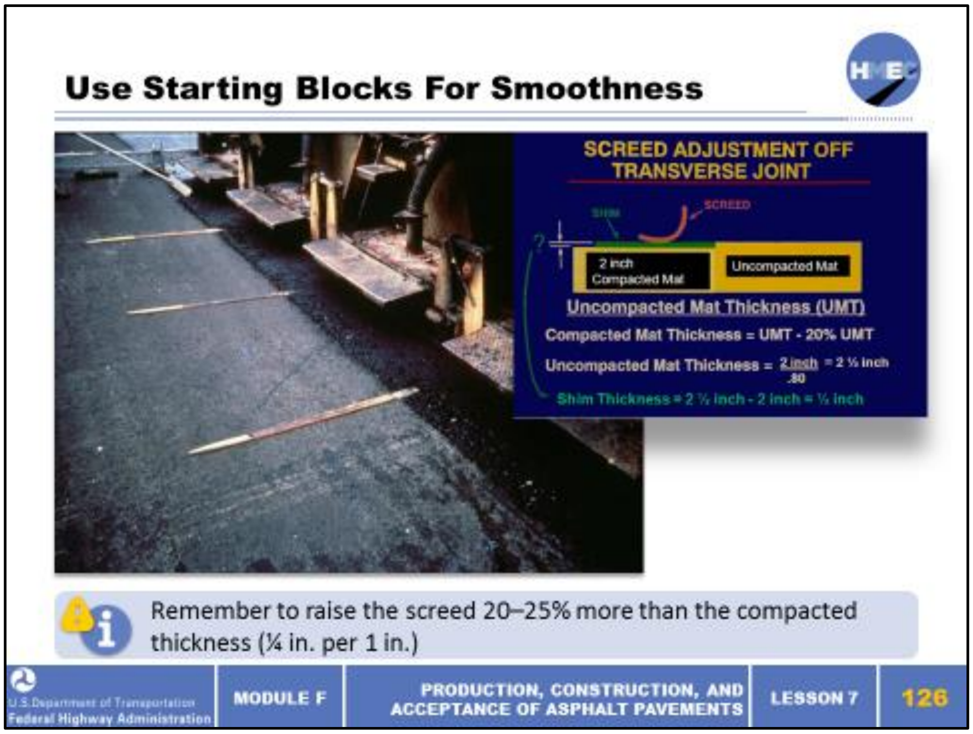
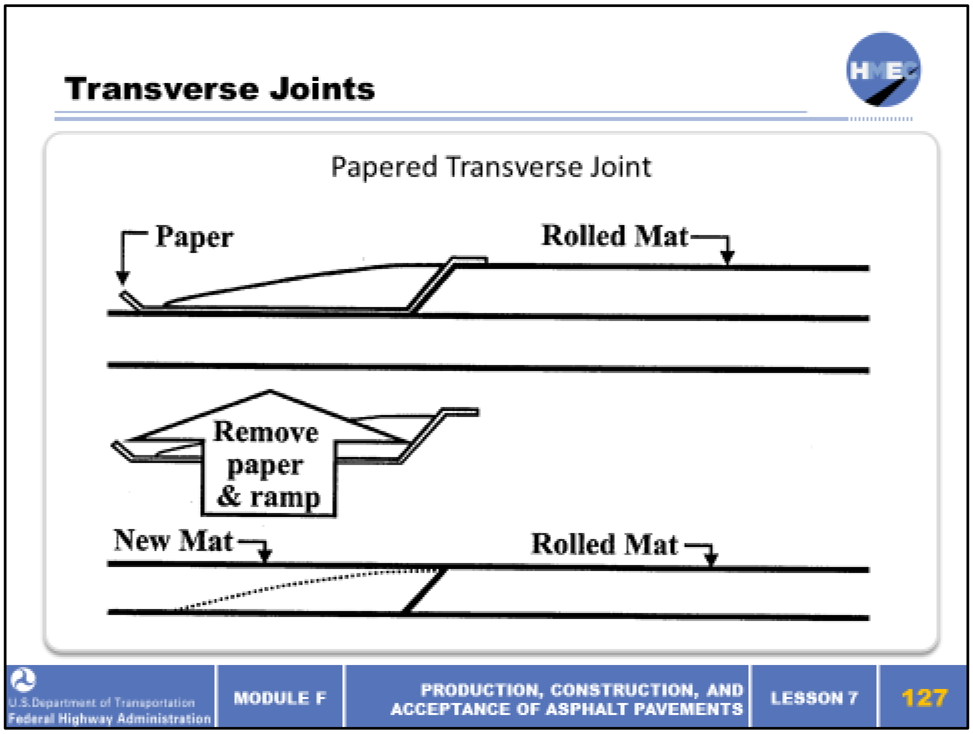
Slide 126



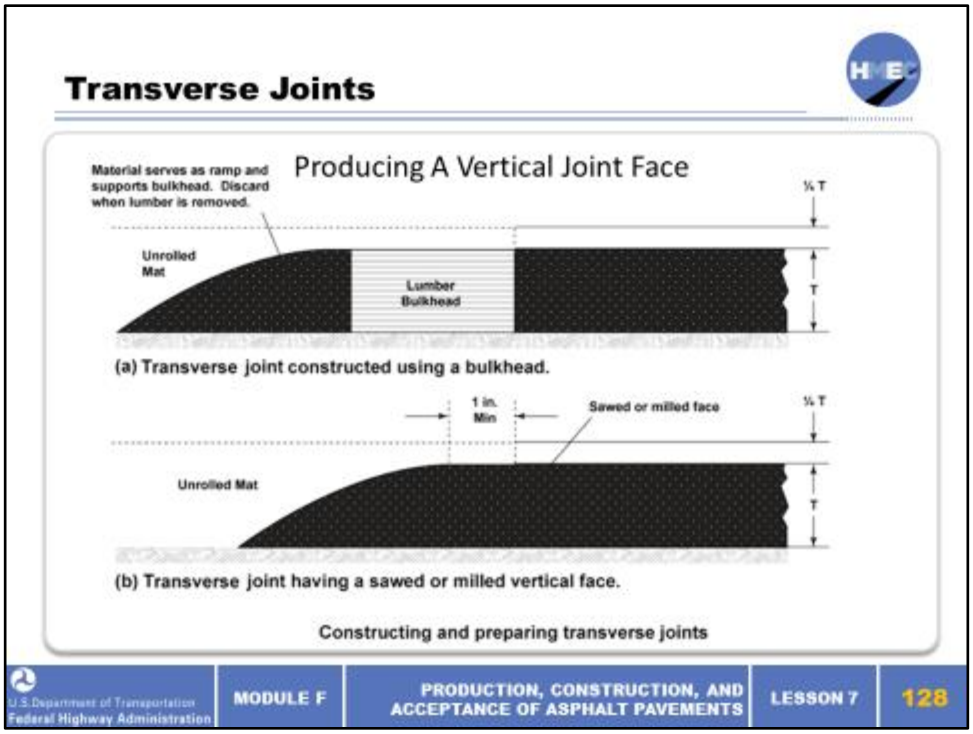
The thickness of the starter blocks should be equal to about 20% of the total planned compacted mat thickness. If starter blocks are not used, then the screed will "drop-off" of the existing mat, forcing an adjustment with the depth crank which will ultimately lead to a dip and a hump in the finished mat near where the pull starts.

Slide 127



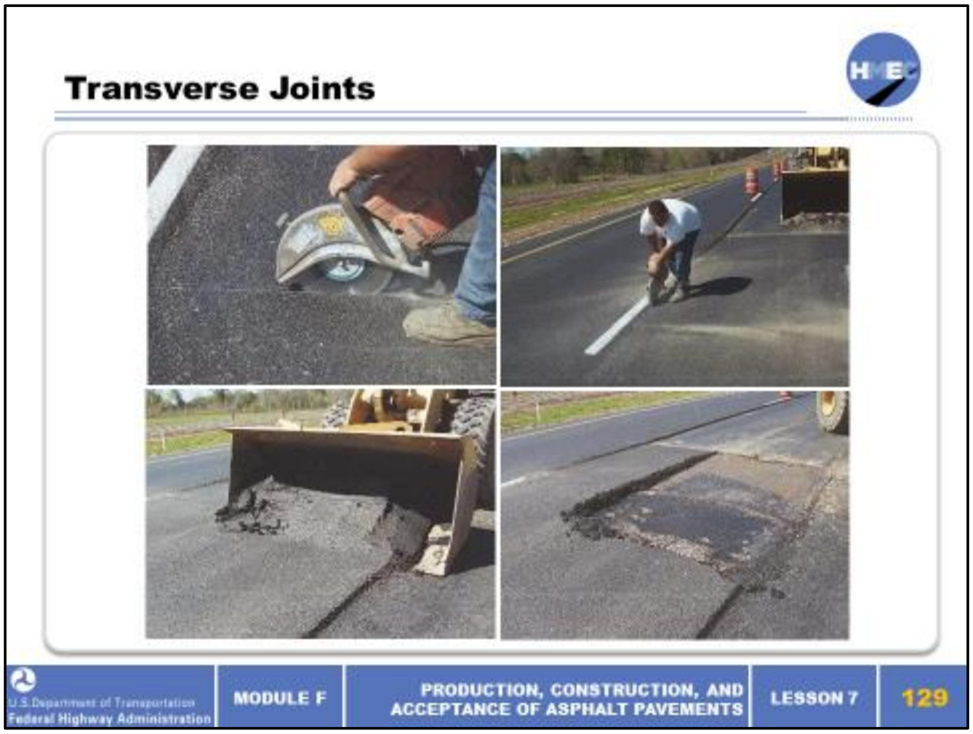
Don't permit dirt or sand to be used instead of paper.

Slide 128



This method is the least commonly used because the same condition can be achieved by sawing and removing the mixture prior to the next day's pull. Another issue with this type of transverse joint is the density adjacent to the block will be lower, in some cases a lot lower than other areas of the mat even though a saw is used to remove a short section of the previously placed mat as shown in the slide.

Slide 129



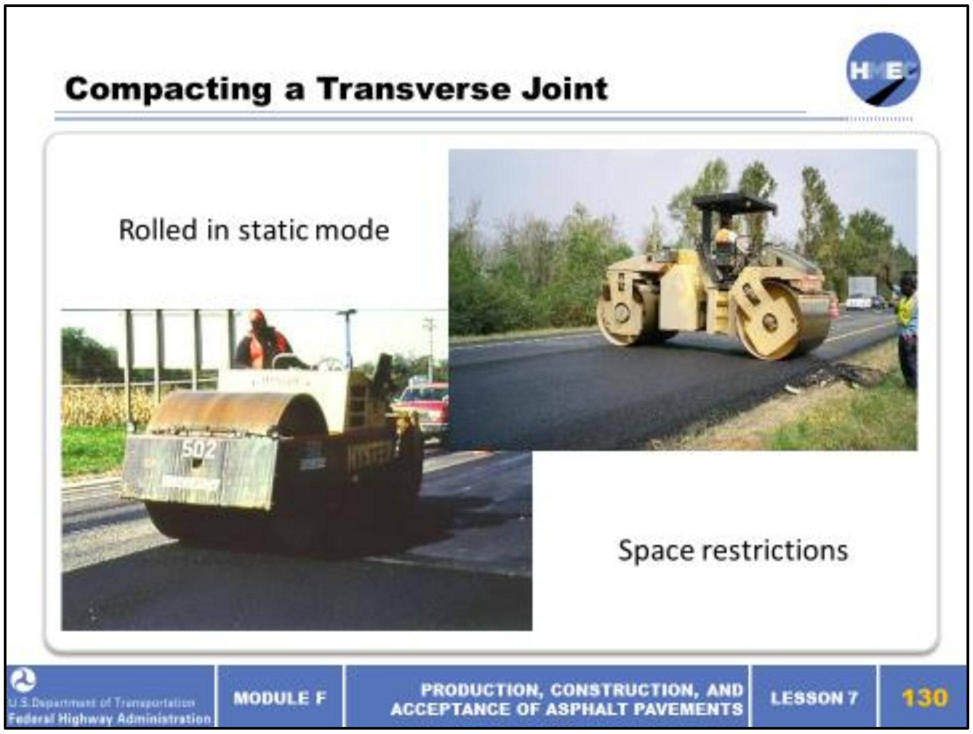
The following provides some additional discussion on each of the above methods.

The use of a nuclear density gauge to identify where the density is similar to that measured within the mat away from the taper is considered the better method because it ensures the density on the cold side of the transverse joint will be similar to the mat density placed during the previous construction day. This process is illustrated in the photographs included in this slide, with the exception that the nuclear density gauge is not shown in the photographs. A saw is used to cut the surface and create a butt longitudinal and transverse joint at the location where the density is similar to the mat density from the previous day's production. A front-end loader is then used to remove the mat within and around the taper that has been marked using the nuclear density gauge. The area is then cleaned and swept and a tack coat applied on the surface and around the butt joints. The transverse joint is then ready for paving. This type of transverse joint is considered to be the best or has a higher probability to perform well.

The other method is to use a straight edge to determine the location where the mat thickness starts to thin out. The photographs in this slide do not illustrate the use of the straight edge. The point where the thickness starts to decrease, however, is where the transverse joint is located. The mat is then sawed, a front end loader is used to remove the cold mat, the area cleaned and swept, and a tack coat applied to the surface and around the butt joins just like for the method where the nuclear density gauge is used. This method is not believed to be as reliable as in using the nuclear density gauge simply because the location where the mat thickness starts to decrease does not necessarily represent the location where the density is similar to the mat density placed during the previous day's production.

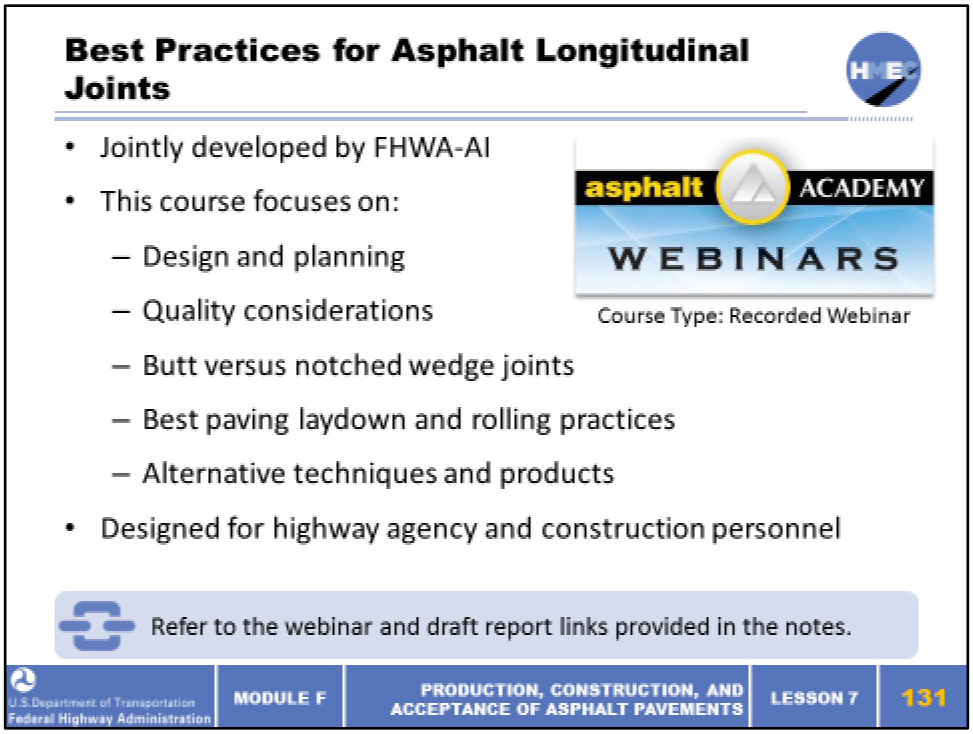
The third method mentioned above results is the least reliable transverse joint because the density at the toe of the taper is probably significantly less than the average mat density of the previous day's production. The transverse joint is prepared similarly to the other two methods, but the mat is sawed transverse where the taper starts from the previous day's production.

Slide 130



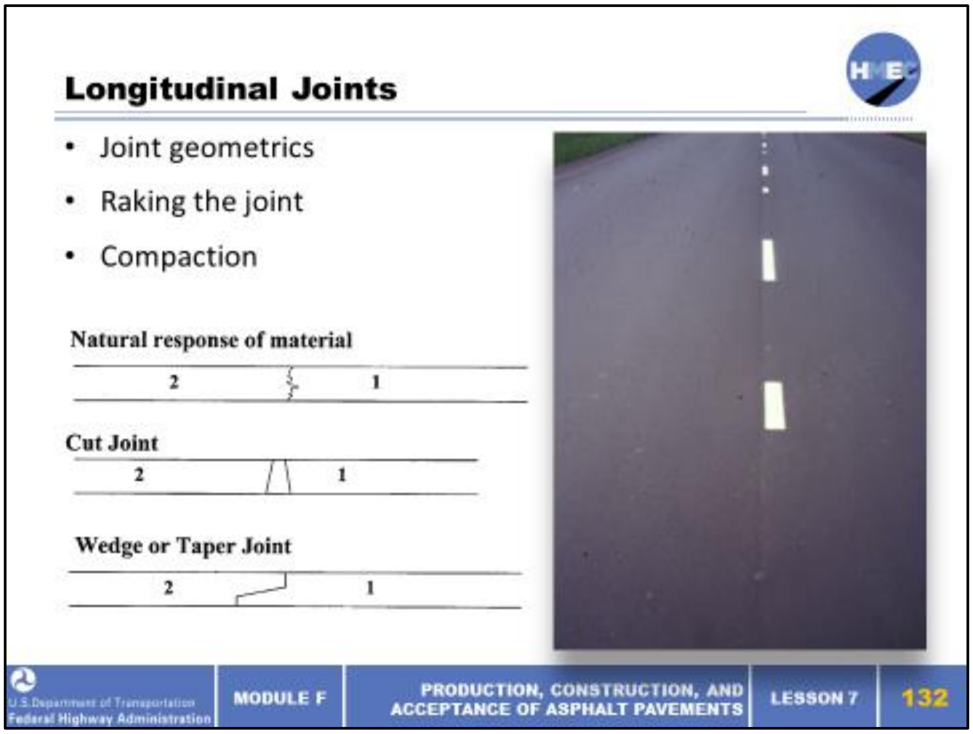
In most cases, there are various restrictions (traffic, as well as steep side slopes and guard rails) that prohibit rolling the joint transversely. For these cases, roll the joint longitudinally, and get the breakdown roller on the mat as soon as the paver is gone. Have the roller pass slowly and completely onto the mat before reversing direction. If the joint is constructed properly, the compactive effort needed will be no different than that needed for the rest of the mat.

Slide 131



This project included a review of available literature and specifications, in-depth interviews with expert paving consultants and contractors, and site visits to States of interest.

Slide 132



Several joint geometries have been used in the past with mixed success. The three most often specified are illustrated in this slide. The natural response of material geometry is the most common and the cut joint is the least common because of time and cost to create that type of joint. Wedge or tapered joints are also common for longitudinal joints that will be opened to traffic prior to placing the adjacent lane. In other words, joint geometry is usually dictated by safety reasons. It should be noted that all joint geometries can provide adequate performance if rolled properly to an adequate density along the longitudinal joint.

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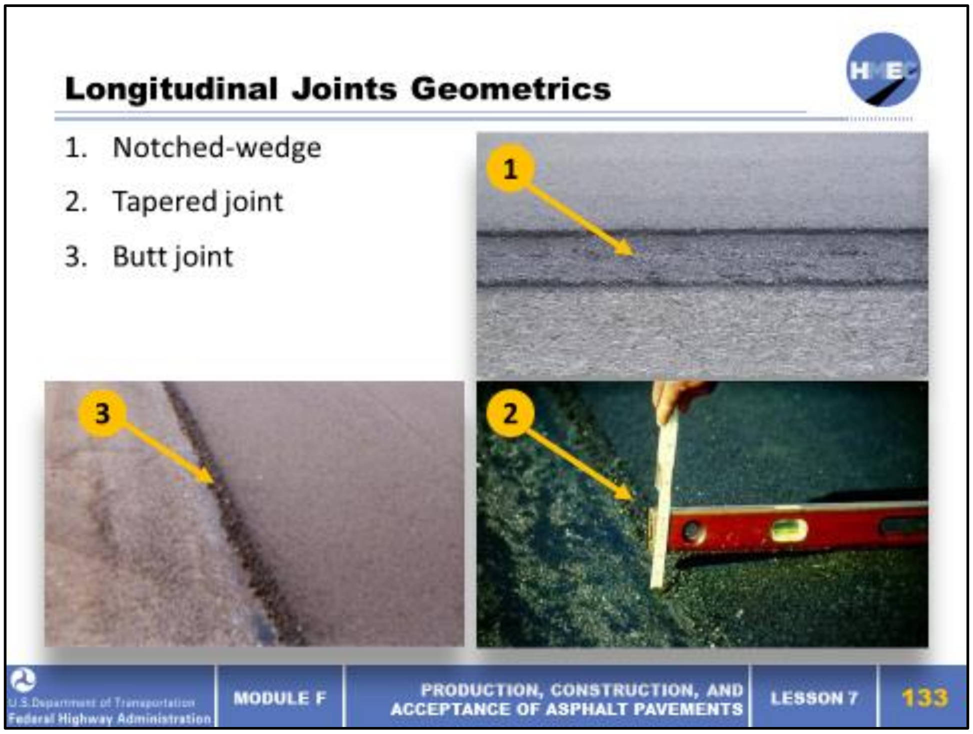
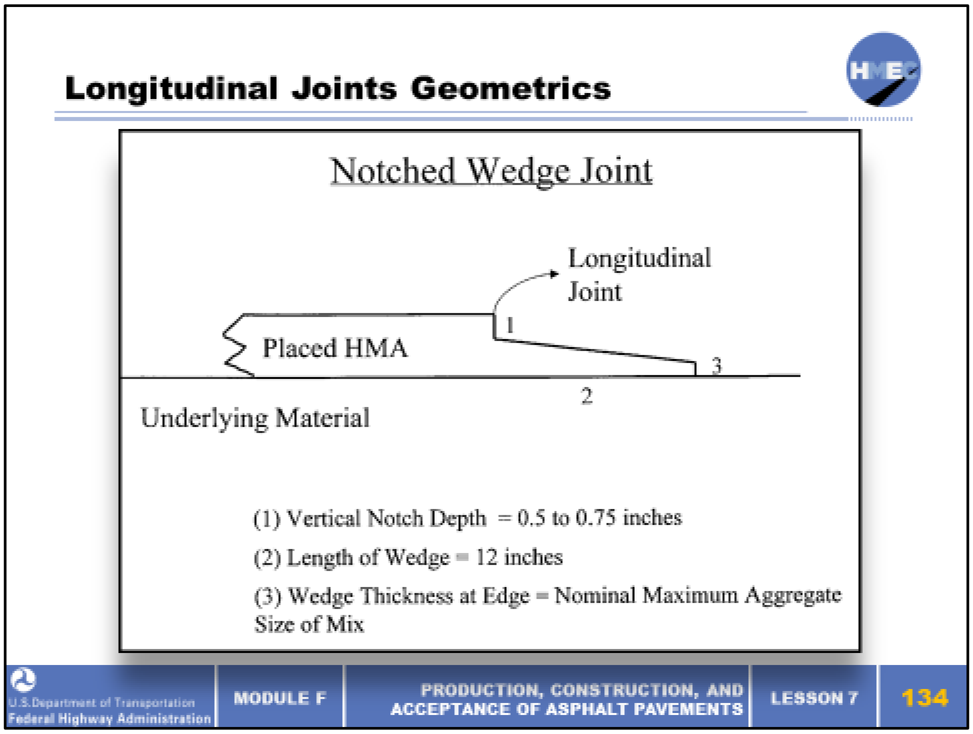
Slide 133



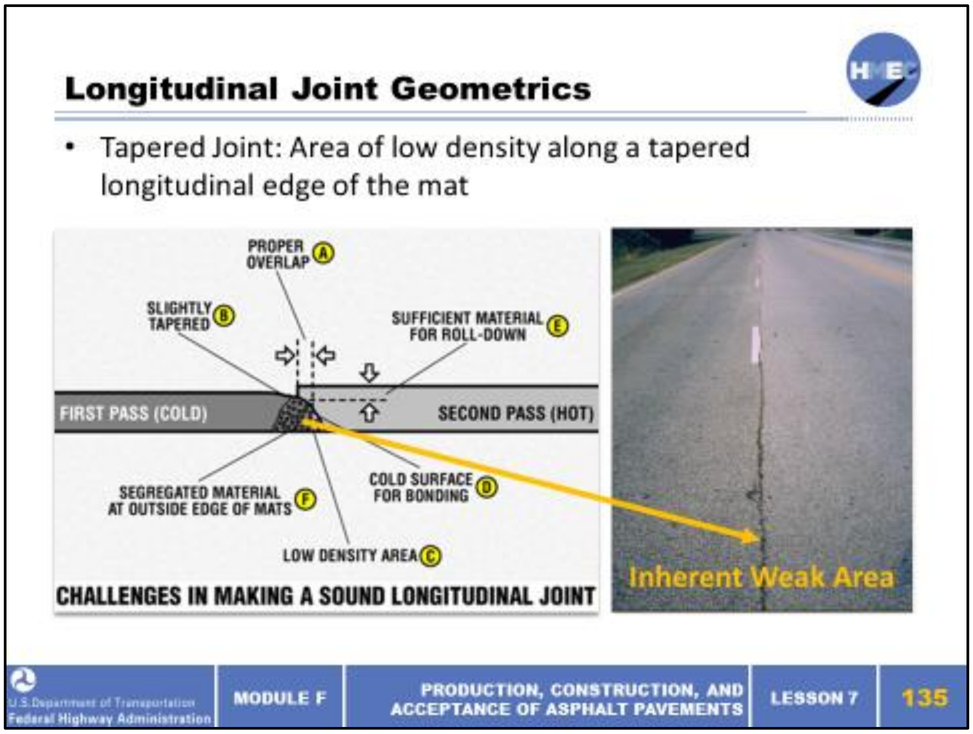
States have been researching whether performance and increased safety can be derived from tapered and notched-wedge joints; both are excellent for fine-grained mixtures. The notched-wedge joint allows traffic to run on it and has provided better performance than for the tapered joints, especially for some of the thicker lifts. An issue with both of these types of joints, however, is placing the adjacent mat along the tapered or notched-wedge joint in terms of getting compaction in the new and previously placed mats. NCAT also conducted a study comparing different types of joints and found the cut joint butt joint was best but cost more to construct. The notched-wedge joint was the next best.

Slide 134



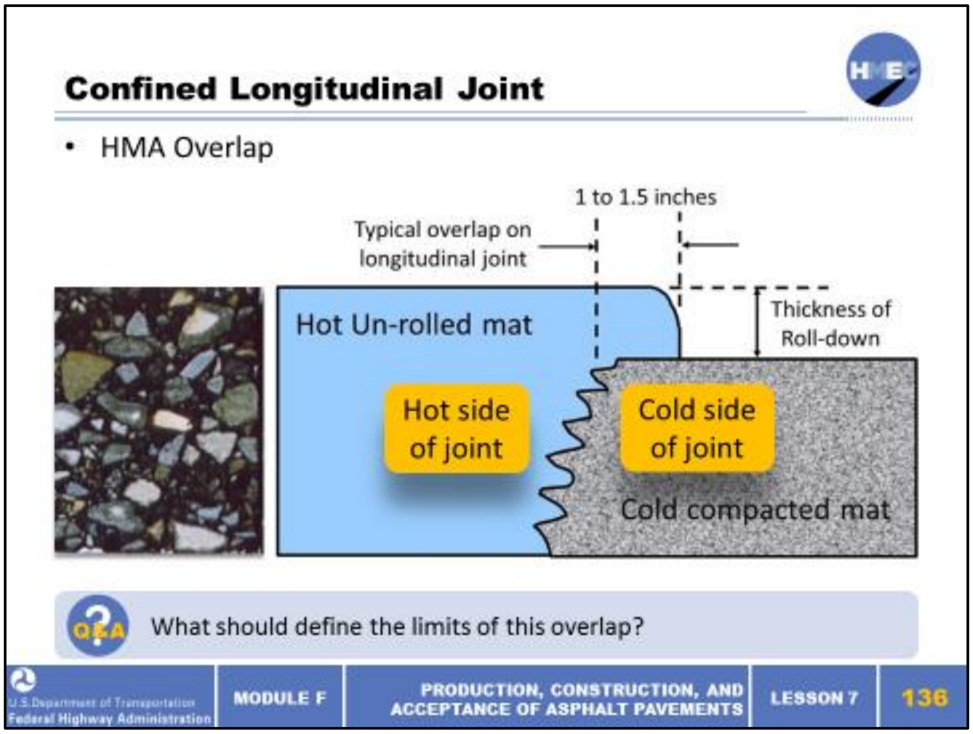
This type of joint is becoming more common in use because of the safety issues with traffic traveling over the longitudinal joint. To form the notched-wedge, a notched-wedge device is available that can be easily attached to the paver.

Slide 135



Tapered joints are being used less and less because of performance issues created by this inherent weak area caused by low densities and some segregation at the end of the taper.

Slide 136



Provide enough overlap to provide a tight joint and allow for the steering tolerance of the paver; typically 25 to 40 mm (0.75 to 1.5 in.) is successful in most cases. The end gate should be in contact with the surface and leave a tight edge.

Slide 137

In addition, the overlapped material should not be "bumped back" prior to the first roller pass (the rake or lute not being used in the photo). With proper rolling of the longitudinal joint, the joint should result in good performance.

Slide 138

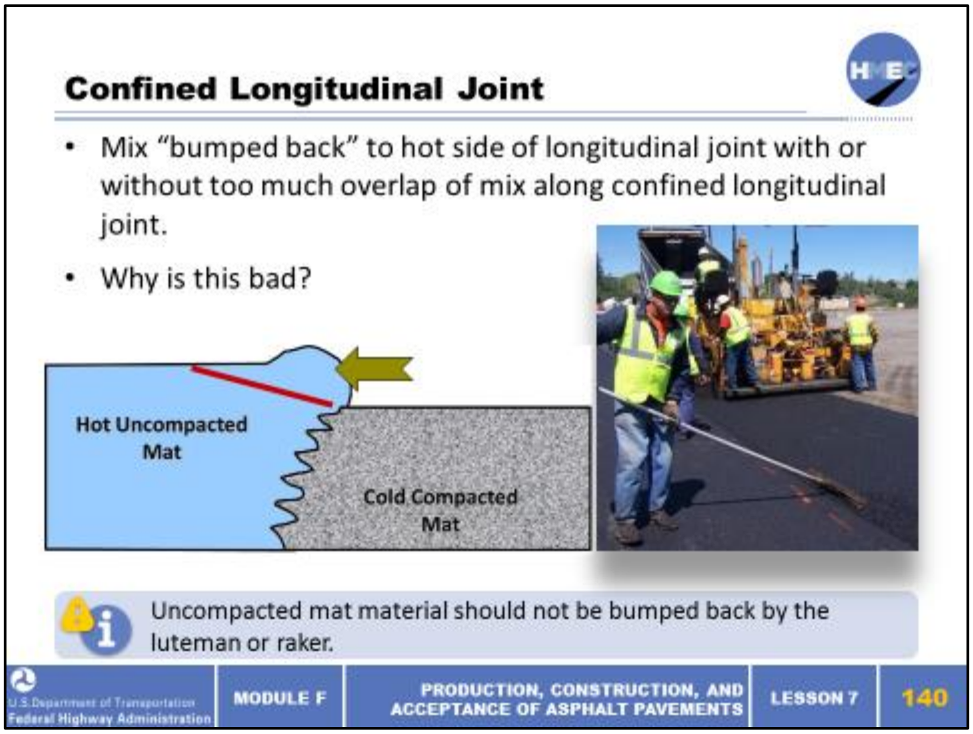
The images and note the hot side and cold side of the confined longitudinal joint.

Slide 139

In the photograph on the left side of the slide, a significant overlap is being placed and the lute or rake is being used to push the mix back or scatter the recently placed mix onto the new mat. The center photograph in the slide shown the pushing and/or scattering mix on the new surface. The photograph on the right side of the slide shows the vibratory roller compacting the longitudinal joint. In areas where the mix was scattered across the new mat, significantly low densities exist along the joint. Even in the areas where the mix was just pushed back to the edge of the joint, densities will be highly variable because pushing the mix back is usually not done on a consistent basis.

As noted on the slide, do not let this happen.

Slide 140



The uncompacted mat material should not be bumped back by the luteman or raker. Bumping the mix back onto the hot side of the mat is never done uniformly so there will be areas along the hot side of the longitudinal joint that may lack sufficient material to be densified when rolled at the interface between the cold and hot side of the joint, and areas where too much mix exists on the cold side of the joint so the roller cantilevers over portions of the hot mat which are not properly compacted. These areas along the joint with insufficient density will deteriorate very rapidly.

“Broadcasting” or scattering the mix back onto the uncompacted mix is another practice that is fairly common that should not be followed, as noted previously. Broadcasting the mix back onto the hot mat usually means the mat thickness on the hot side will be at the same level as the cold mat, and thus, the roller will ride on the cold compacted mat so the density on the hot side of the joint will be significantly low in comparison to the mat interior.

Some contractors use a tow attachment at the rear of the paver to bump the edge back. It is better practice, however, to place the mixture so that no bumping and/or broadcasting of the mix is required.

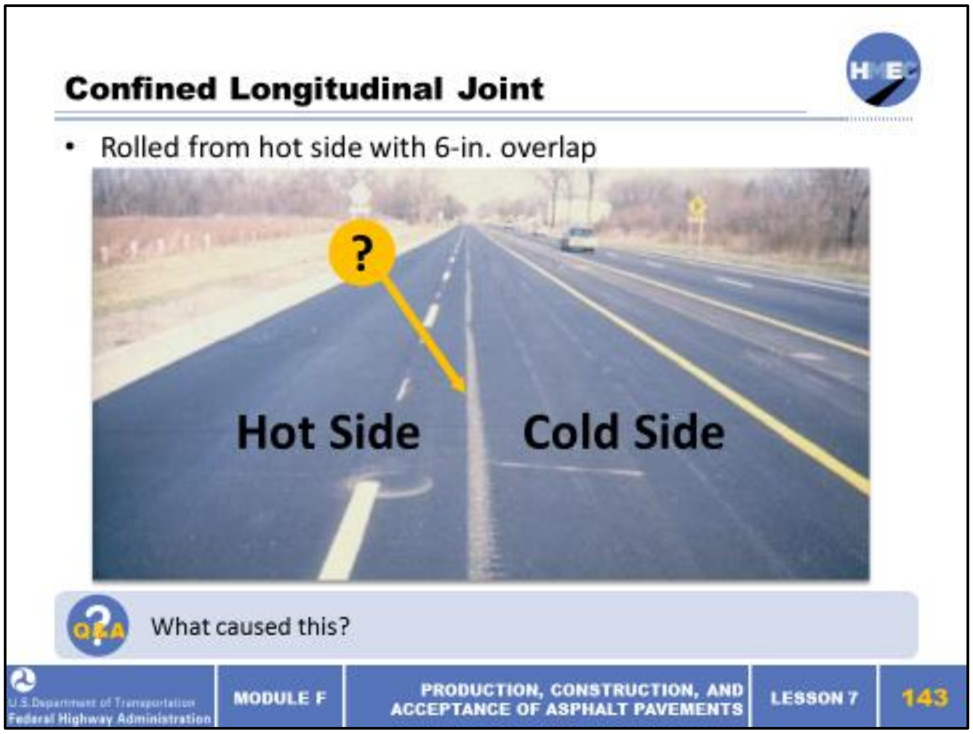
Slide 141

The next series of slides will overview each compaction strategies.

Slide 142

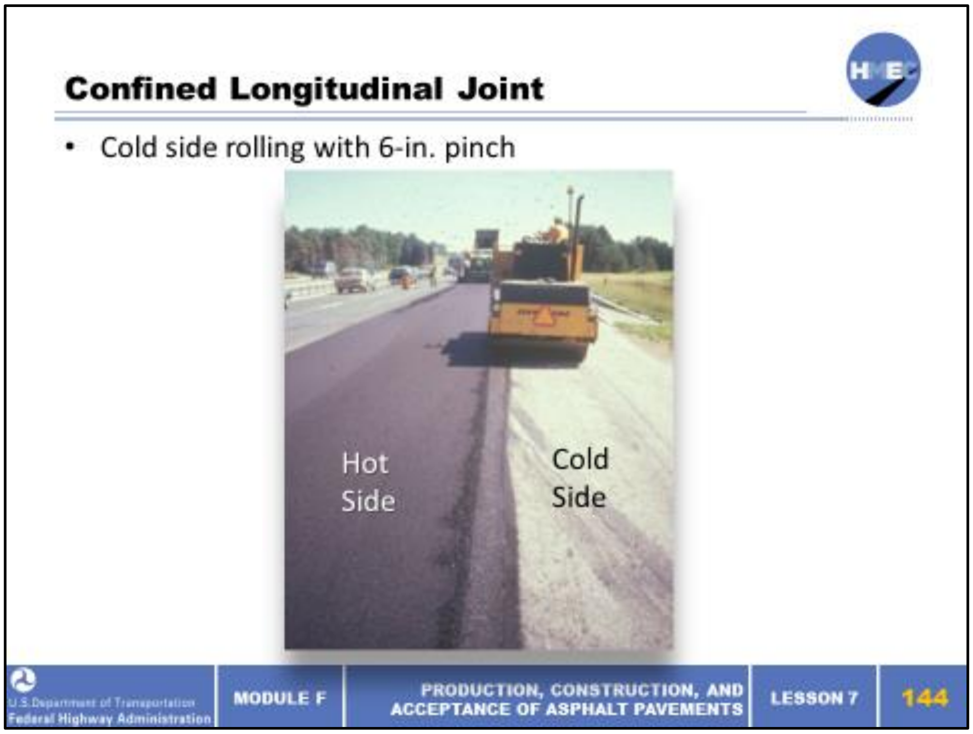
The first roller pass is made with the bulk or most of the drum on the uncompacted hot mat overlapping onto the compacted cold mat by approximately 6 in. The first pass of the roller can be operated in the vibratory mode if a vibratory roller is being used in the primary position directly behind the paver, assuming that the thickness of the uncompacted mat is adequate to facilitate the roll down through the compaction process.

Slide 143



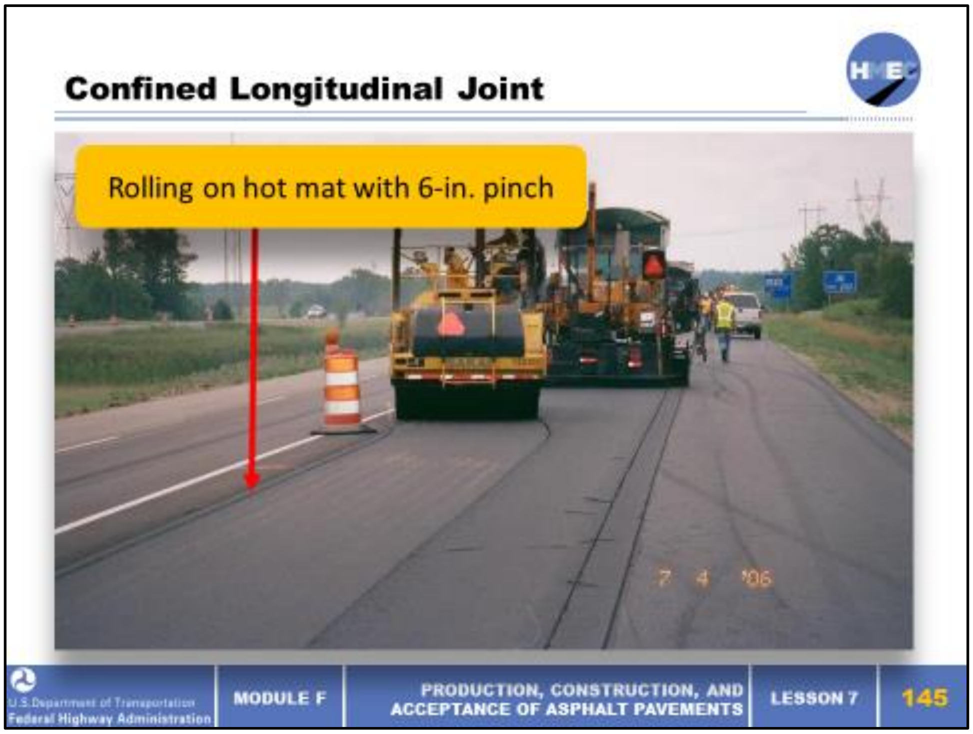
This pavement is now likely to experience premature raveling.

Slide 144



The first roller pass is made with the bulk of the drum on the compacted cold mat overlapping onto the uncompacted mat approximately 6 in., so it pinches the area along the longitudinal joint. If a vibratory roller is used as the primary roller behind the paver, the roller is operated in the static mode.

Slide 145

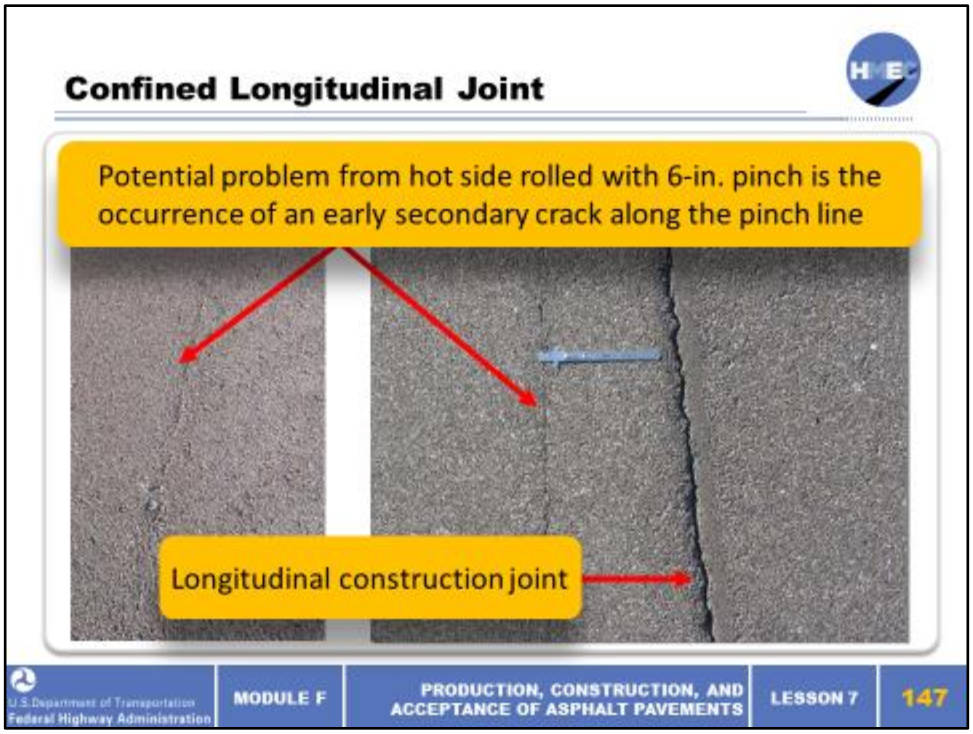


The first roller pass is made with the entire roller drum on the uncompacted, or hot mat and the edge of the roller drum approximately 6 in. from the longitudinal joint. The objective of using this method is to provide confinement and increased density at the joint. The vibratory roller can be operated in the static or vibratory mode for this first pass. It normally is in the vibratory mode being 6-in. from the joint. The second roller pass, when using the hot side with a 6-in. pinch strategy, is not shown on this slide, but is over the 6-in. area not compacted adjacent to the joint and is generally in the static mode, in case it makes contact with the cold surface.

Slide 146

As shown, the roll hot side with 6 in. overlap exhibited the higher average density over the roll cold site with 6 in., pinch. The other strategy of rolling on the hot side with a 6-in. pinch was not used on these projects. Based on this information, the contractor made a decision to compact all confined joints using the strategy of rolling on the hot side with a 6-in. overhang or overlap.

Slide 147



This does not indicate that secondary cracks will always occur on projects where the contractor utilized the strategy of rolling on the hot side with 6-in. pinch method. However, it does indicate the possibility of occurrence compared to the hot side roll with 6-in. overhang or overlap strategy.

Slide 148

Typically, the notched-wedge is constructed with a 0.5 in. upper and lower notch and a 12:1 taper between the upper and lower notch. The device is mounted on the edge of the paver, adjacent to the end gate, and in front of the screed. The device is then adjusted below the screed to form the wedge in the newly placed HMA. A small tow-behind roller weighing approximately 400 lbs. or greater is pulled along the wedge to provide some compaction.

The notched-wedge is rolled or compacted like any other unconfined joint.

Typical problems or issues reported for the notched-wedge have included:
1. Maintaining the upper notch during compaction.
2. Raveling on the lower portion of the wedge.
3. Aggregate pickup on the small-wedge roller.

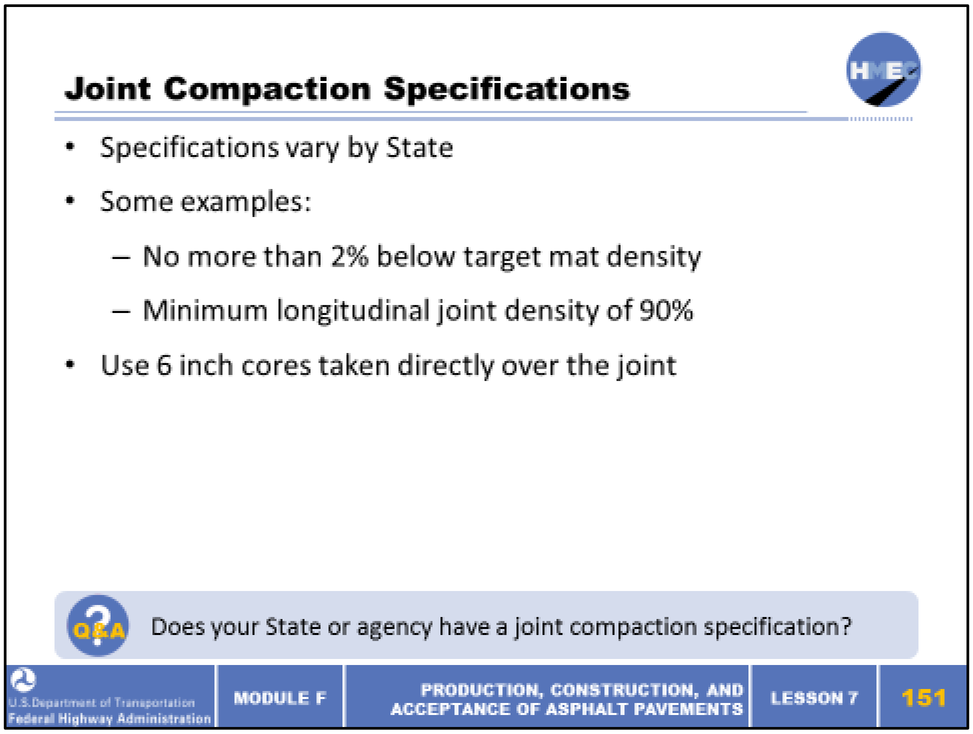
Slide 149

The first roller pass is made with the bulk of the drum on the uncompacted mat overlapping or overhanging the mat by approximately 6 in. The first roller pass is operated in the static mode. For pneumatic rubber tired rollers, the tires should not overhang the mat but be located along the edge of the joint.

Slide 150

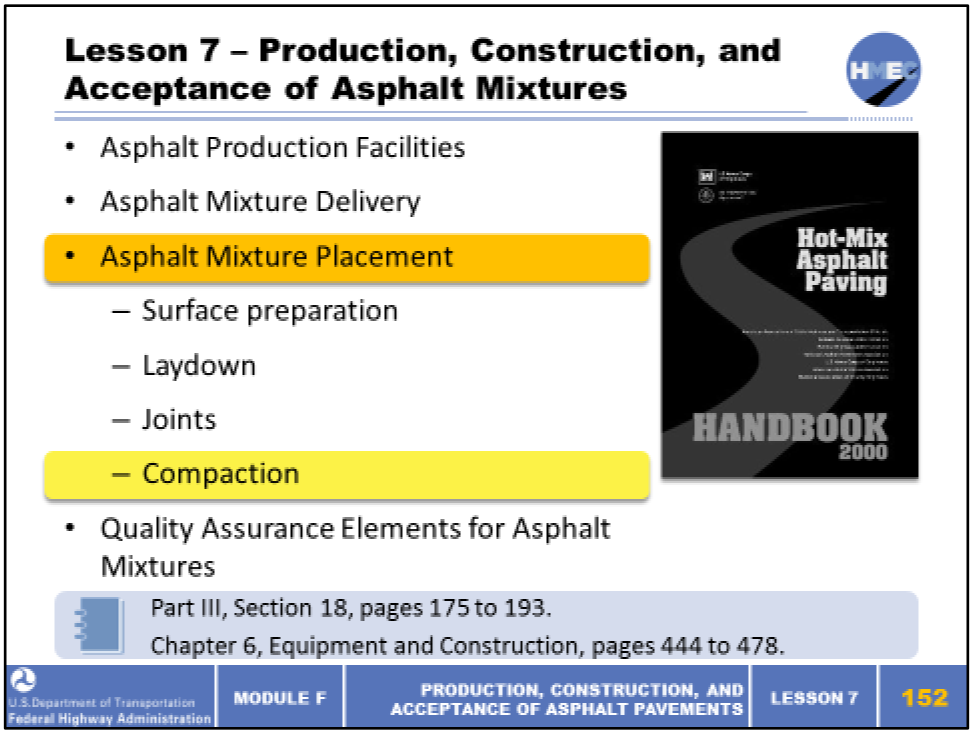
For this project, the mixture did not shove outwards and did not crack along the edge of the mat.

Slide 151



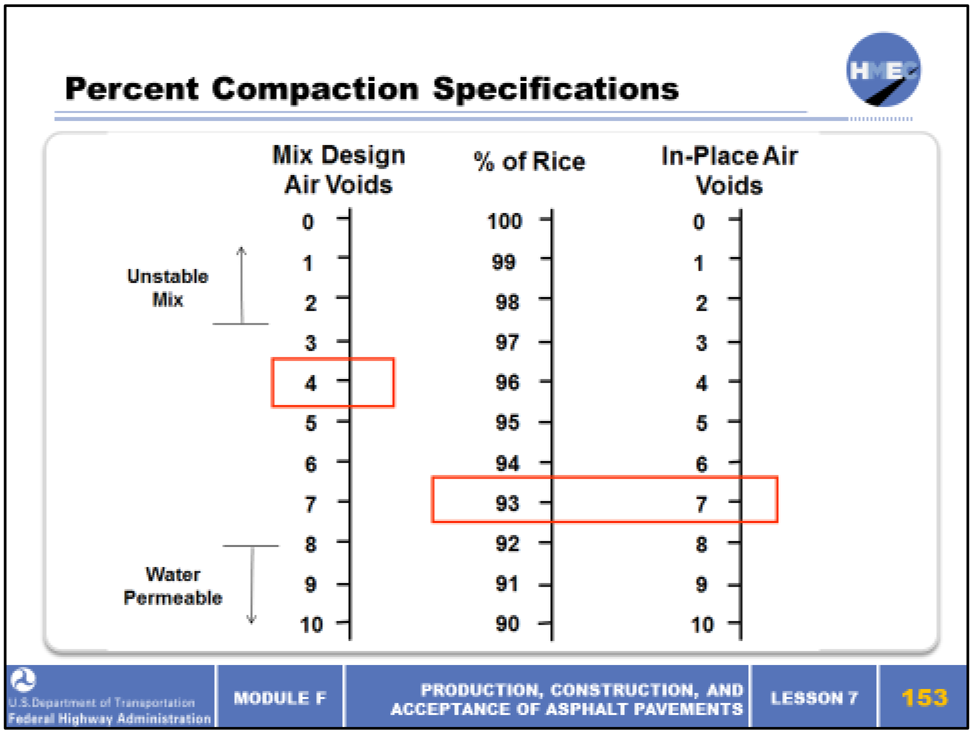
Most agencies that do specify a minimum compaction level or density of a longitudinal joint require that it be within 2 percentage points of the mat compaction level or density. Cores are usually taken directly over the joint for determining the average joint density, as compared to the average mat density.

Slide 152



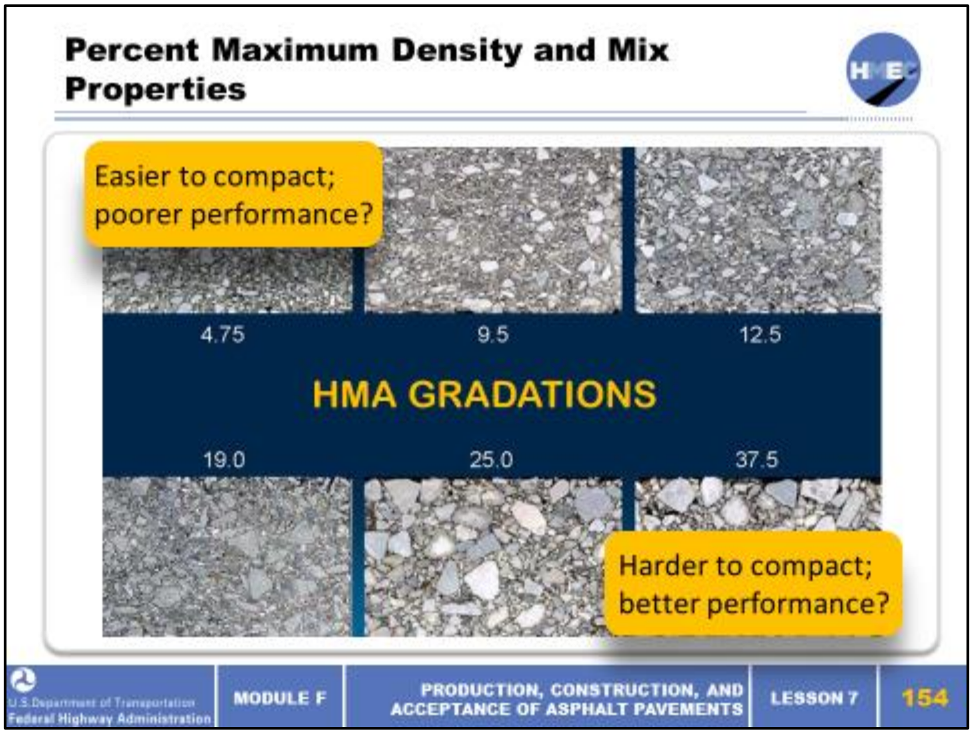
Pages 175 to 193 of Part III, Section 18 in the *Hot-Mix Asphalt Paving Handbook* is focused on mixture compaction to improve the properties resistant to pavement distresses. In addition, pages 444 to 478 in Chapter 6 of the NCAT textbook provide additional information on compacting HMA mixtures.

Slide 153



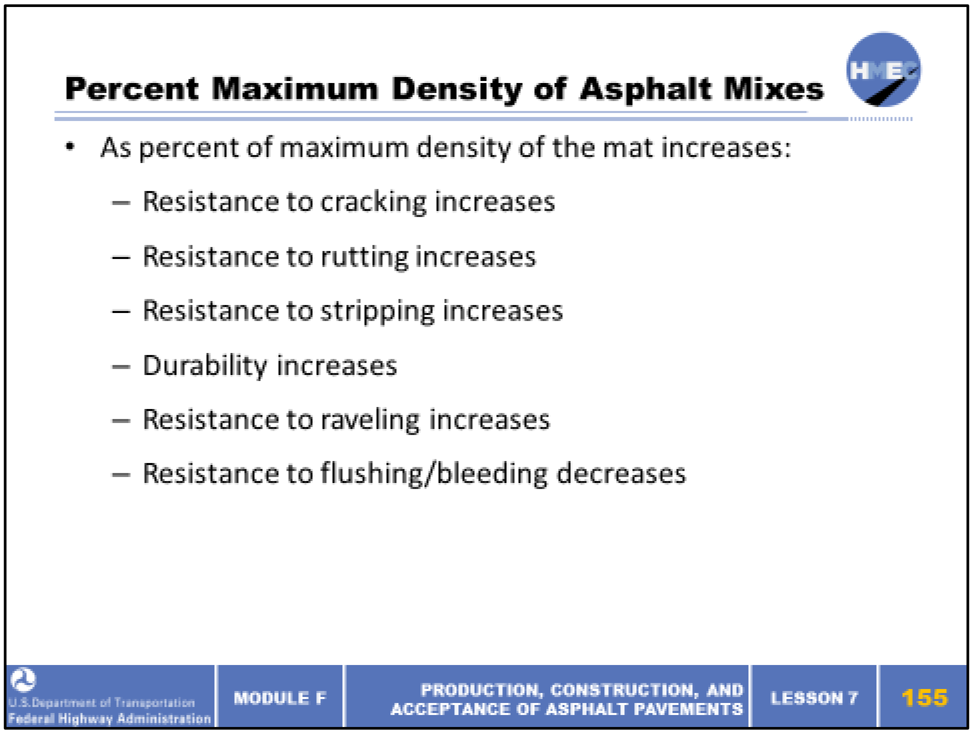
Most agencies target the 7% air void level after construction or compaction by the rollers. The specification value or the minimum and maximum values generally vary from 92 to about 96% compaction or 4 to 8% air voids.

Slide 154



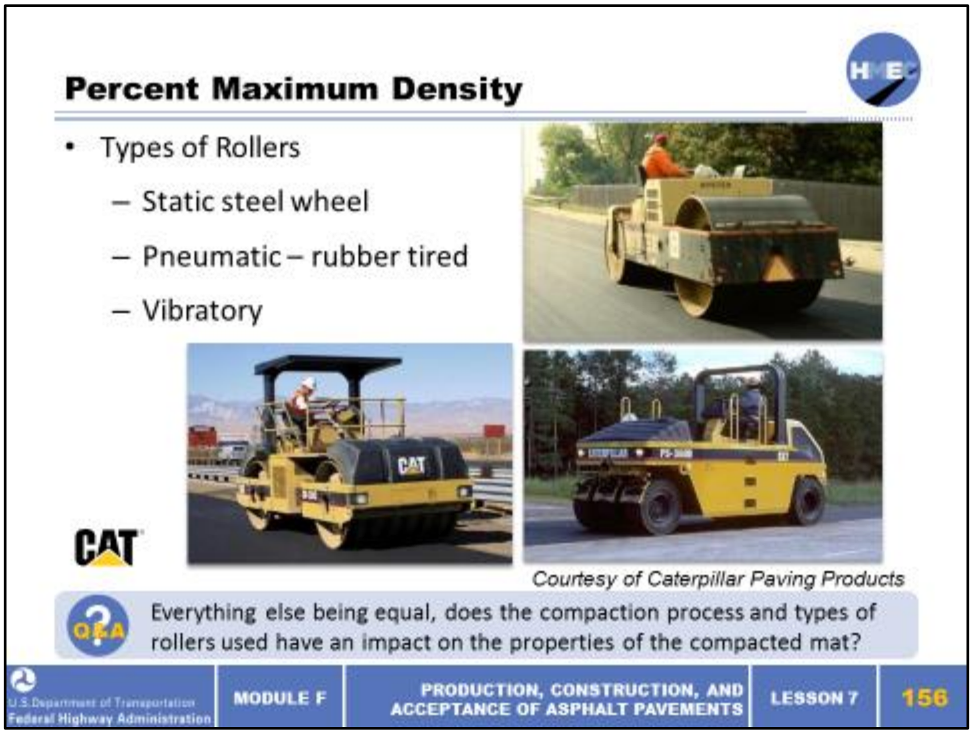
Different mixes compact in different ways and may require adjustments in rollers used and rolling patterns. The amount of compactive effort or number of passes of the rollers varies and is dependent on the properties of the mix. The more harsh the mixture, the greater the compactive effort to reduce the voids to the 5 to 8% level and the greater the resistance to rutting and cracking. The less harsh the mixture, the lower the compactive effort to reduce the voids to 5 to 8% level. If the air voids are only reduced to these levels, the more susceptible the mixture is to rutting and cracking and raveling.

Slide 155



This slide shows the relationship between percent compaction or density and resistance to the occurrence of distress. As percent compaction increases (density increases) along the roadway, the resistant to all distresses increases with one exception: bleeding/flushing.

Slide 156



Three types of self-propelled compaction equipment are currently being used: static steel wheel rollers, pneumatic tire rollers, and vibratory rollers. A few manufacturers have now developed pneumatic vibratory rollers, but they are not in high demand nor used extensively within industry.

Slide 157

Drawbar pull is defined as the horizontal force required to move the roller forward. Rollers with large-diameter rollers have lower drawbar pull (rolling resistance) because they do not tend to penetrate as far into the mix as does a roller with smaller-diameter rollers. Static steel wheel rollers normally range in weight from 2.7 to 12.7 tonnes (3 to 14 tons) and have compression drums that vary in diameter from approximately 1 to 1.5 m (3 to 5 ft.).

The gross weight of the roller can usually be altered by adding ballast to the roller to increase the contact pressure per linear mm or foot, but this adjustment cannot be made while the roller is operating, and is not normally changed during the term of a project.

Slide 158

The tire pressure used depends in part on the number of plies used in the tires.

If the mix is tender, a lower tire pressure will displace the mix less than will a higher pressure in the tire. For a stiff mix, a higher tire pressure can be used, because the mix will be stable enough to support the weight of the roller without the mix shoving laterally under the tires.

Tire pressure is normally kept constant for a particular project, but the level selected should be dependent on the properties of the mix being compacted and the position of the roller on the mat.

Most pneumatic rollers are operated in the intermediate roller position, behind a vibratory or static steel wheel breakdown roller and in front of a static steel wheel finish roller. These rollers are sometimes used for initial rolling of the mix as well as occasionally for finish rolling, with the exception of the wearing surface.

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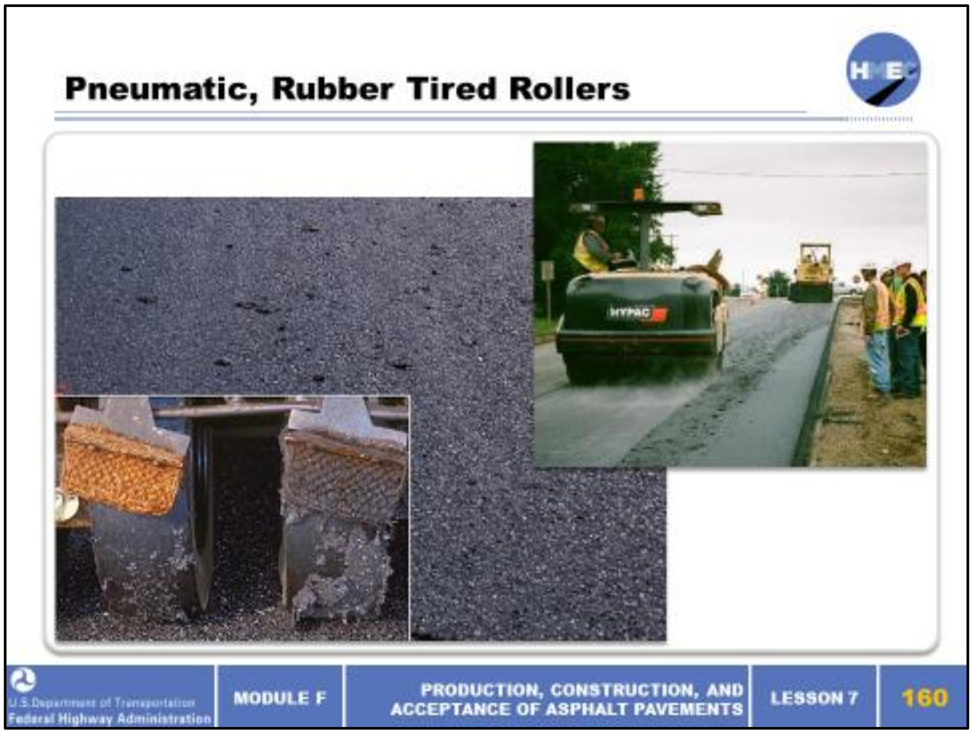
Slide 159

For conventional mixtures, these rollers work very well if the tires are heated prior to compacting the mix.

Slide 160



Mix pick up is shown in the slide from two different projects and should be avoided. Mix pick up is less important on the intermediate or lower layers and critically important for the wearing surface. Although the density of the pavement is probably being achieved, the finished surface is mottled and may have surface blemishes. When pick up occurs, a steel wheel roller can be used to reduce the surface blemishes, as shown in the upper photograph.

To minimize mix pick up on the tires, the tires need to be at an elevated temperature or close to the compaction temperatures. The temperature of the tires is increased by operating the pneumatic roller on an existing pavement first and then by slowly getting them hot on the new pavement. The key is to get the tires hot and keep them hot by keeping the roller moving.

The use of "skirts" not only helps in getting the tires hot but also ensures that they will stay hot all day by limiting the wind's cooling effects.

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Slide 161

Steel Wheel Vibratory Rollers

- Compaction Parameters:
  - Weight
  - Frequency
  - Amplitude

Mat Thickness	Frequency	Amplitude
Thin; < 1.25	Static	Static
1.25 to 2.5	High	Low
> 2.5	High	High

Proper Impact Spacing: 10 to 12 impacts per ft.

Q&A Why is high frequency suggested for all mat thickness levels, except for thin mats?

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Slide 162

A recent survey of major roller manufacturers recommended the following impact spacing.

The impact spacing should be in the range of 30 to 36 impacts per meter (10 to 12 impacts per ft.), or 25 to 30 mm (1 to 1⅕ in.) between impacts, to ensure the highest efficiency of the vibratory rollers and reduce the possibility of leaving ripples in the finished pavement.

Proper impact spacing and amplitude are the keys to successful pavement compaction. A Reed Tacheometer (shown on the screen) is an inexpensive device that can be easily placed on the mat surface to measure the frequency of the vibratory roller.

Newer equipment automatically adjust the frequency to the roller speed so the impacts per foot stays in the proper range.

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Slide 163

Asphalt Mix Compaction

- Typical Roller Position and Speed of Roller, mph

Type of Roller	Breakdown Roller	Intermediate Roller	Finish Roller
Vibratory Steel Wheel	√ 2.0-3.0	No	No
Rubber-Tired or Pneumatic	√ 2.0-3.5	√ 2.5-4.0	No
Static Steel Wheel	√ 2.0-3.5	√ 2.5-4.0	√ 3.0-5.0

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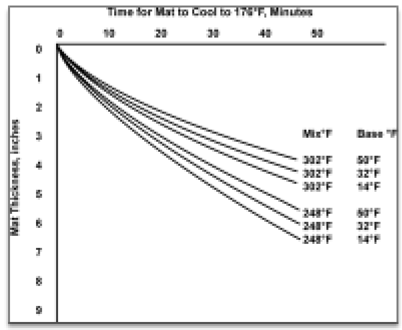
Although the table shows rubber-tired or pneumatic rollers are not used in the finish position, there have been a few cases where these rollers have been used in the finish position. Rubber-tired rollers can be used in the finish position for the lower lifts where the surface defects or roller marks have no impact of the smoothness of the wearing surface. In addition, there have also been cases where the vibratory rollers have been used in the breakdown and intermediate roller position.

Slide 164

Time Available for Compaction

- Factors affecting the time available for compaction:
  - Properties of the materials
  - Environmental variables
  - Laydown site conditions

Factors Affecting Rolling Time	Allows More Time
Mat Thickness	Thick
Mix Temperature	High
Base Temperature	High



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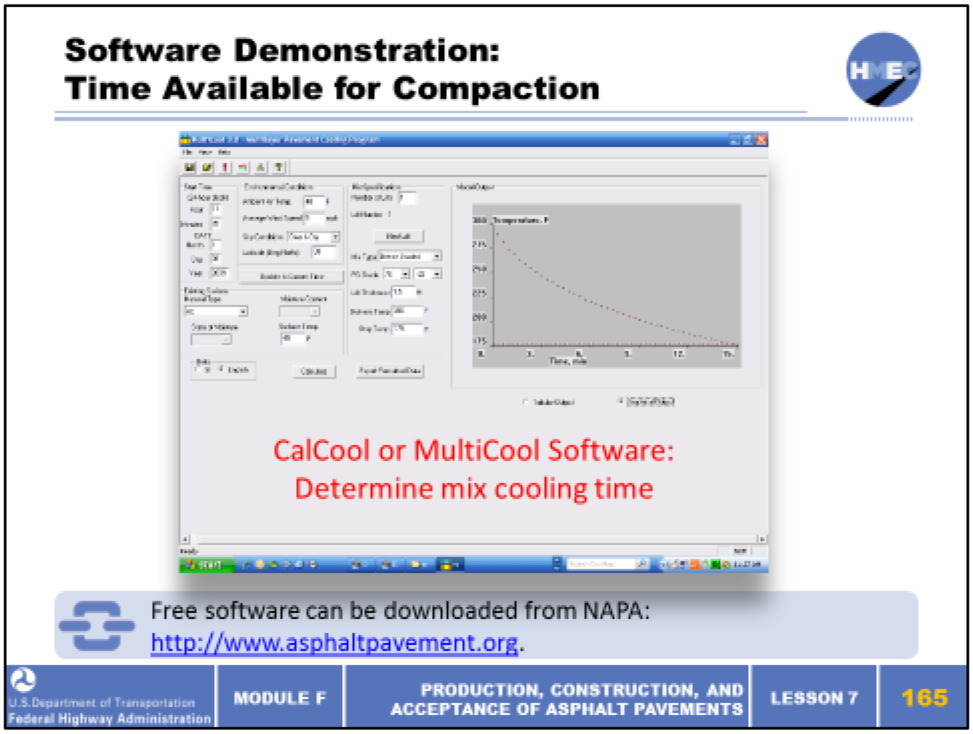
The compaction of HMA is generally achievable at temperatures from 185 to 300 °F. However, the bulk of compaction (achieved with breakdown rollers) must typically be achieved at temperatures above about 250 °F.

Relationships between ambient air temperature, base course temperature, mat thickness, and the time available until mat temperature drops to a level below which additional compaction can be achieved, have been developed. This slide illustrates the relationship graphically for a specific set of conditions. The graphical presentations are commonly referred to as “cooling rate curves.” Cooling rate curves have been published for standard conditions.

Key environmental variables that affect compaction include ambient air and base course temperatures, HMA temperature, wind velocity, and solar flux. There is also an interaction between layer thickness and each of these parameters. This table in this slide summarizes the global relationships between mat thickness, mix temperature, base temperature, and time allowed for rolling operations.

Slide 165



CalCool and MultiCool require the user enter such items as the time of day (entered automatically according to the computer's clock), latitude, air temperature, wind speed, mix information, and lift thickness for each HMA lift, type of material being paved over, its temperature and whether or not the soil or base are frozen. The program then computes how quickly each lift of HMA will cool and when the next lift may be placed. This allows field personnel to make decisions concerning cold weather paving and the scheduling of HMA deliveries to the paving site. While it is no substitute for good judgment and on-the-job observations, it can help in the planning process and in-field decision making.

Free software can be downloaded from NAPA: http://www.asphaltpavement.org.

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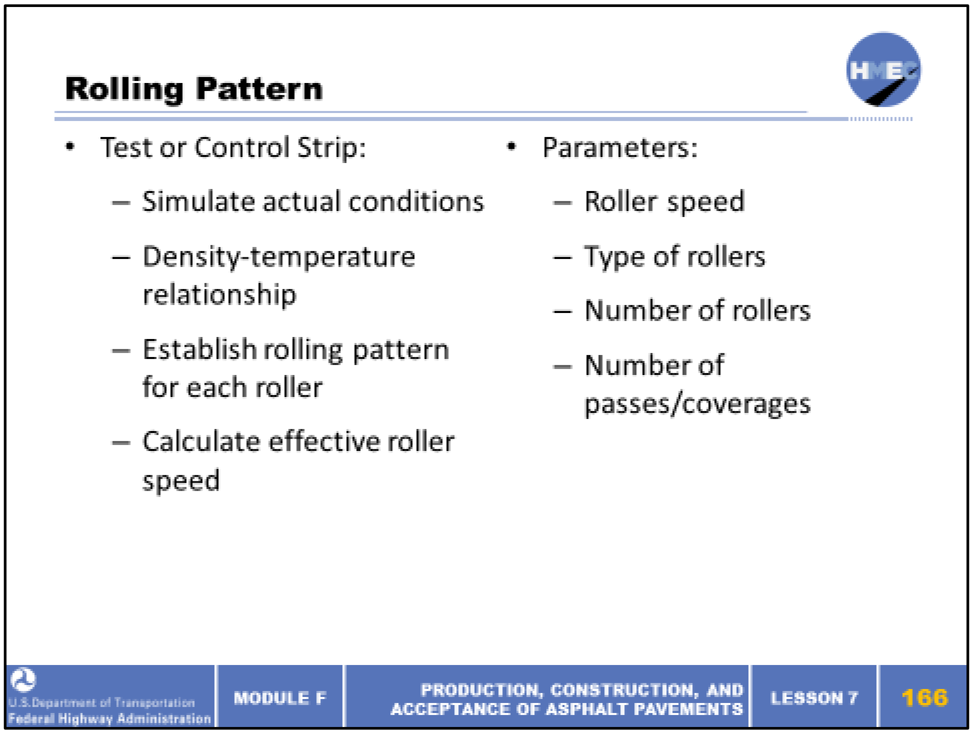
Slide 166



When establishing roller patterns from test strips or production paving, the most appropriate compaction equipment is first identified— generally, the equipment available to the contractor. The width of paving and roller drum widths obviously play into this process. The parameters that are defined through the control strip are listed in the slide.

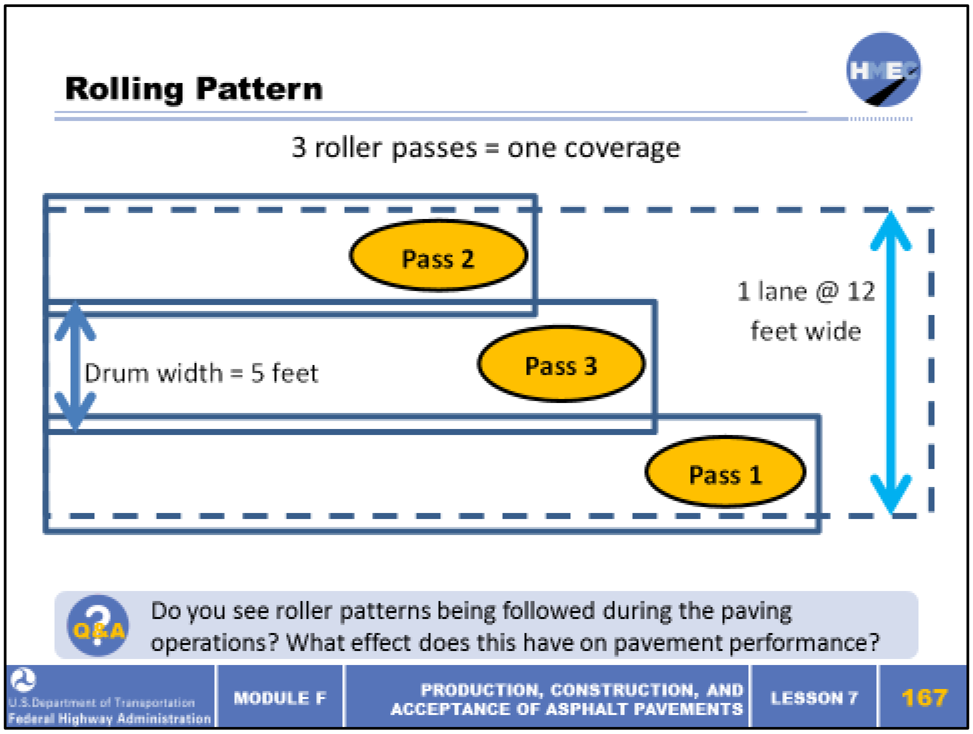
Additionally, the number of roller coverages with each compactor type is identified for the given environmental conditions for reaching the required density within the time available for compaction.

Finally, density measurements must be made to ensure adequate compaction is achieved with the selected equipment using the number of coverages identified.

Test strips are used to confirm/establish roller patterns while simulating actual construction conditions. Some specifications require test strips while others leave it up to the contractor. Many contractors use the first lot as their test or control strip. Under this strategy, the contractor takes on the responsibility for the control strip—just like for the first lot of paving.

Regardless of whether they are required, a construction of a test strip is always good practice when starting a project or using a new mixture.

Slide 167



If the width of the roller drum is 5 ft. (1.5 m), three passes of the roller are needed to cover a 12-ft. (3.7 m) wide mat, including a 6-in. (150 mm) overhang/overlap at each edge of pavement and between the different roller passes. On the other hand, a 7-ft. (2.1 m) wide roller can complete one coverage in two passes.

For thick lifts, some documents recommend, the first pass should be about 0.30 m (1 ft.) in from edge to prevent shoving of the mix laterally. However, with mixtures that have good internal stability, the steel wheel roller can be extended over the end of the mat without any or significant shoving of the mix.

When performing breakdown rolling, the first pass should be made at the unconfined edge on the down-slope side (typically the outside of pavement), or one of the unconfined edges if two exist (first paver pass). The densification along the edge increases the confinement for compacting or densifying the center of the mat. As such, after one unconfined edge is rolled, the roller should roller the other unconfined edge and then move to the center of the mat. The location is obviously dependent on the lane width and width of the roller, as noted above. This procedure provides some additional confinement as successive passes are made towards the center.

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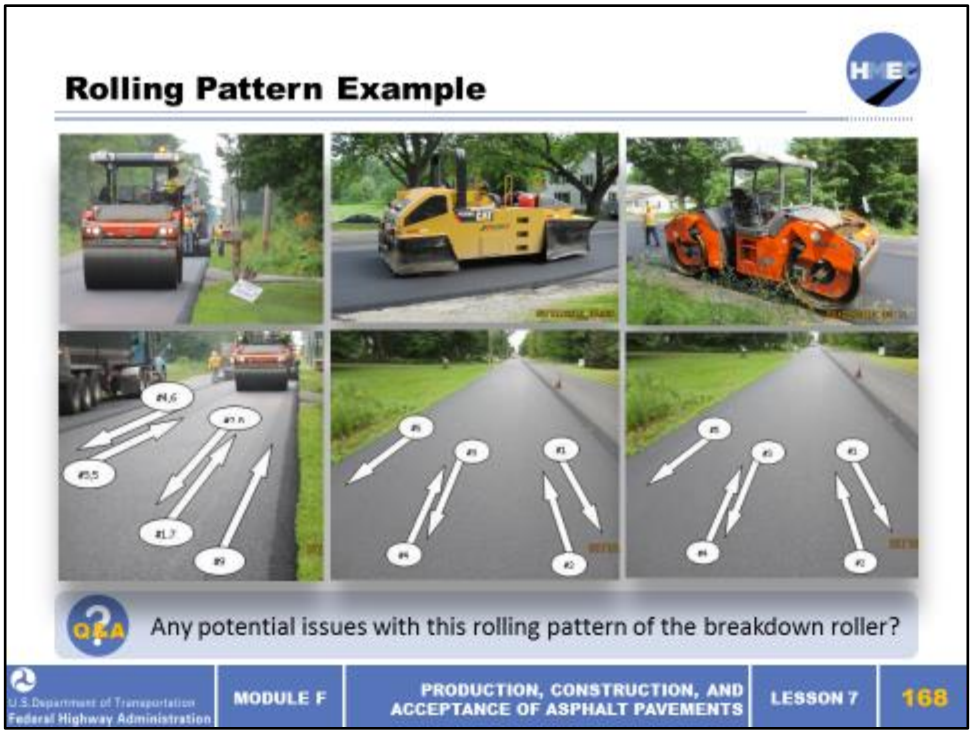
Slide 168


Rolling Pattern Example
Any potential issues with this rolling pattern of the breakdown roller?
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Slide 169

Conversely the rolling patter shown by the track of the roller in the photograph on the right side of the slide is not repeatable because it varies from one side of the mat to the other side throughout the rolling zone. The reason the roller operator was making these types of passes is that no one told the operator it was important to be consistent so uniform densities are obtained.

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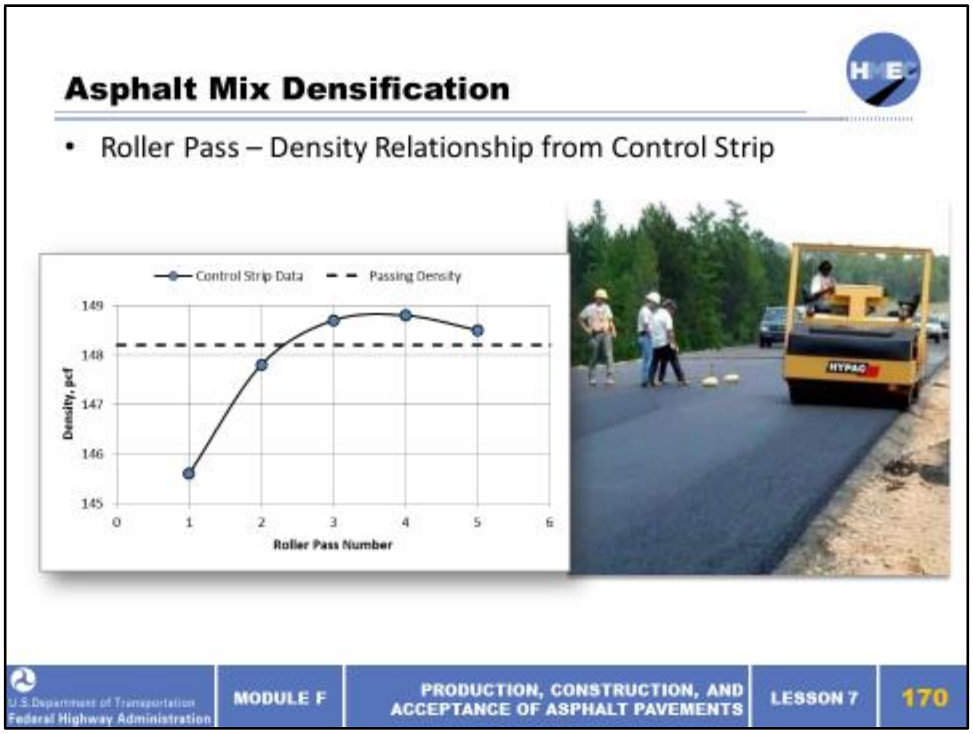
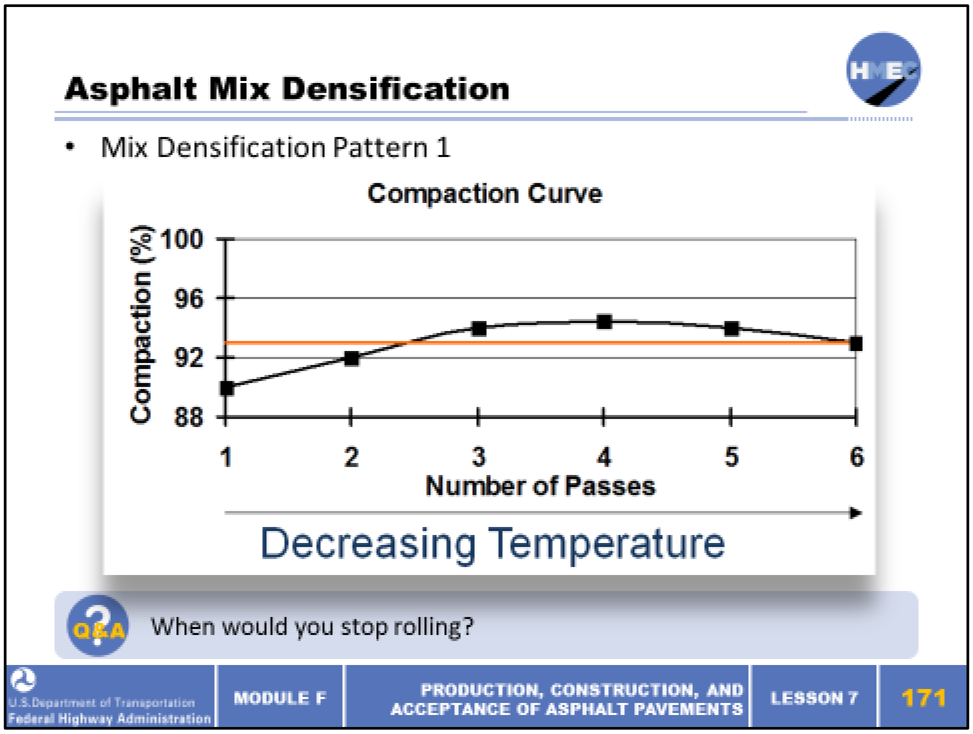
Slide 170



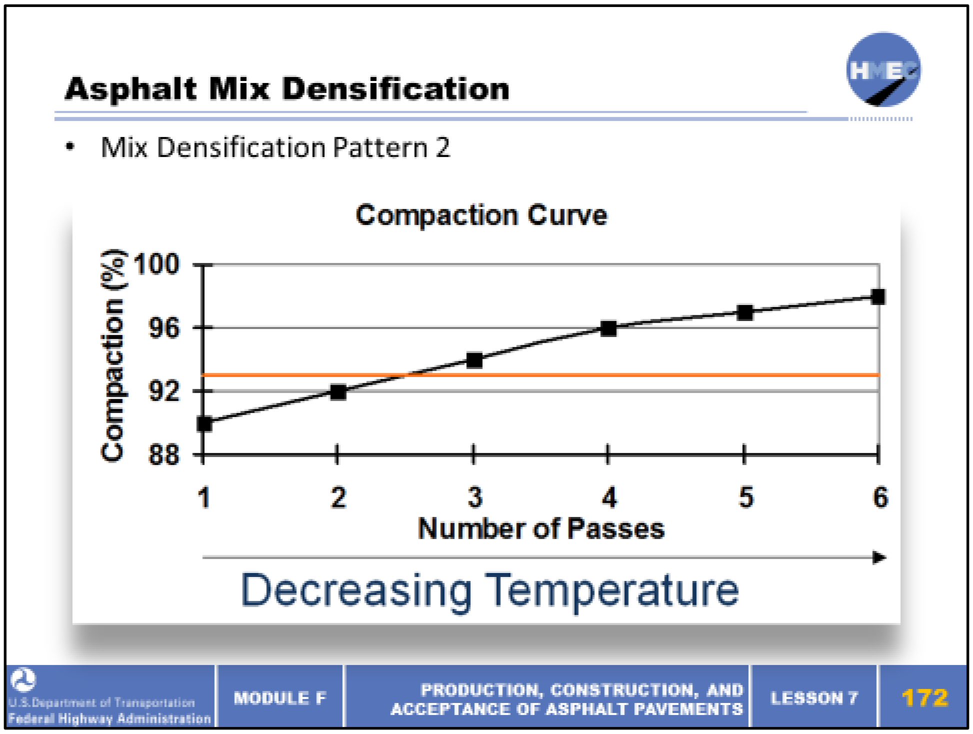
It is also important for the test strip to measure the density after each pass of the roller to determine when the mix density levels off and then begins to decrease. Nuclear density gauges are effective tools for use in establishing roller patterns and controlling compaction on projects. It is absolutely critical, however, that the gauges be properly calibrated and correlated to density measured on cores.

Slide 171



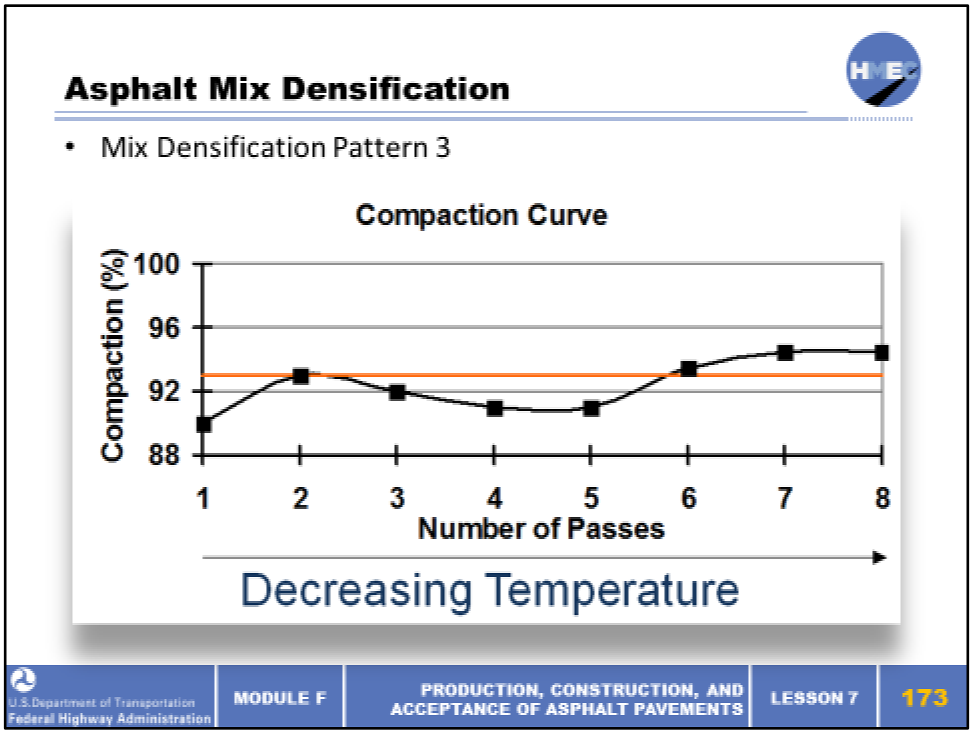
Temperature is critical for each pass, so it should be monitored and collected with each density reading.

Slide 172



The mix can easily be over-compacted so a new design should be developed. Some contractors stop or discontinue the test strip when the specified density or percent compaction is achieved and are later involved in disputes with the owner when premature rutting occurs. When it is discontinued, the type of densification pattern is never determined.

Slide 173



Temperatures are equally important as well as the density of the mix so the contractor can define those temperatures that cause a drop or decrease in the mat density. In most cases after the temperature has dropped sufficiently the mat density will continue to increase with increasing number of passes until the mat cools below the temperature at which no additional densification occurs. For this example, the roller operator should stop after pass 7.

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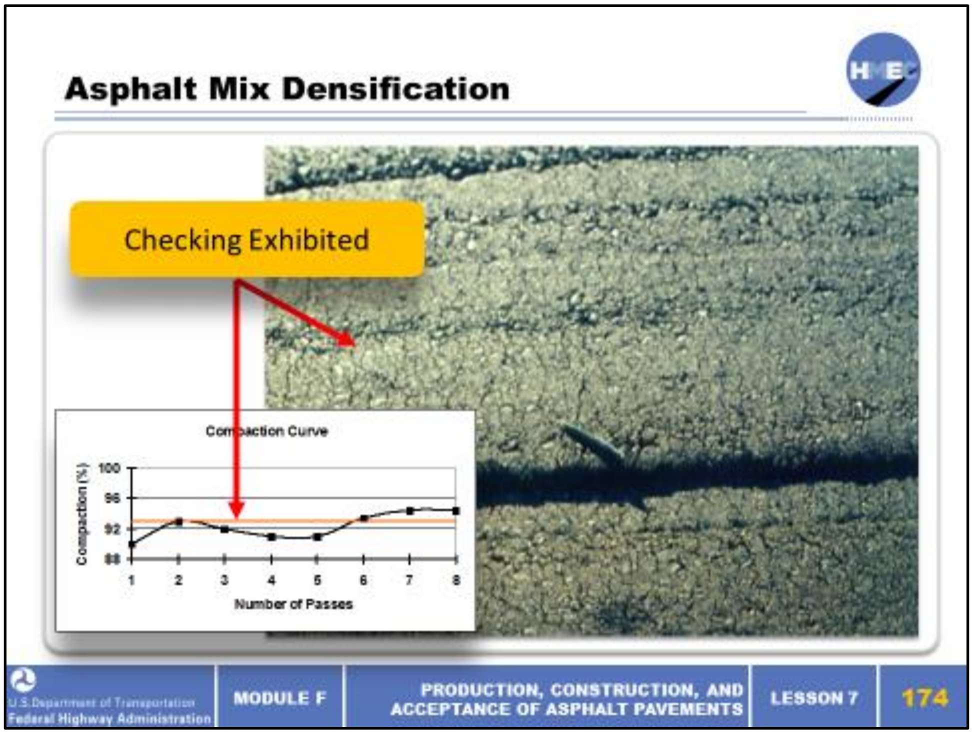
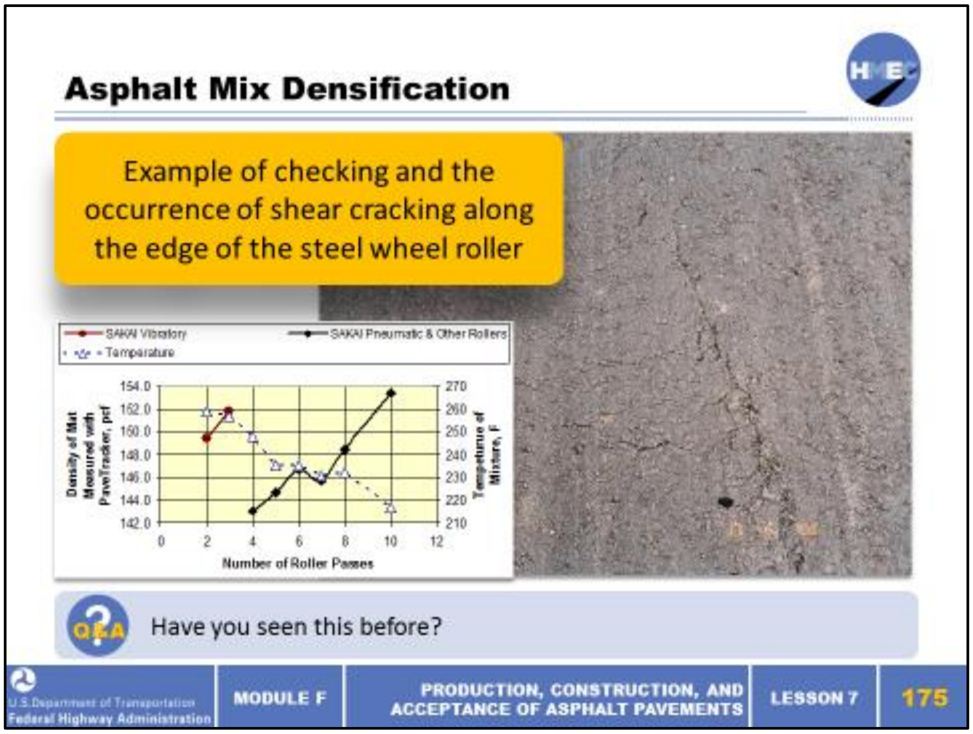
Slide 174



The severity of checking depends on the tenderness of the mix and physical properties of the mix. When checking is observed, all rolling should be discontinued. In some cases, the mix will need to be redesigned to eliminate the checking.

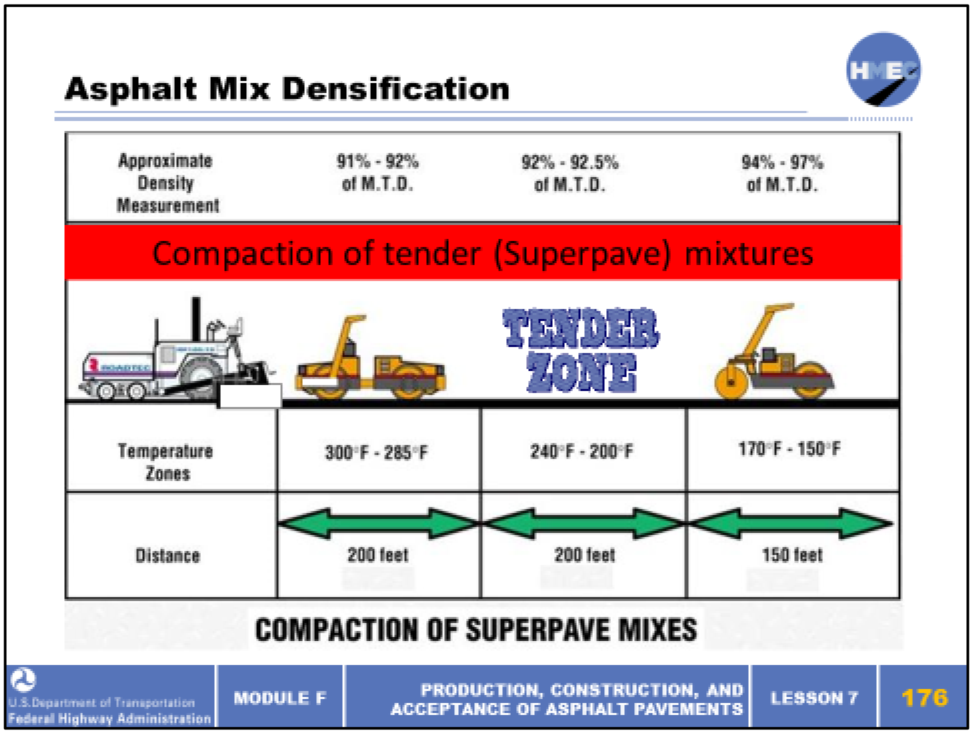
Slide 175



The fourth roller pass, which was with the pneumatic rubber tired roller, caused a significant reduction in the density of the mix. The temperature sensitive zone for this mix was 230 to 250 °F. In an effort to try and increase the density of the mix, the contractor continued rolling the mix with another steel wheel roller, which resulted in the cracks shown on the slide.

The agency/owner penalized the contractor for low densities, which became a dispute between the contractor and owner. This dispute could have been avoided by simply delaying or not rolling the mat between 230 and 250 °F.

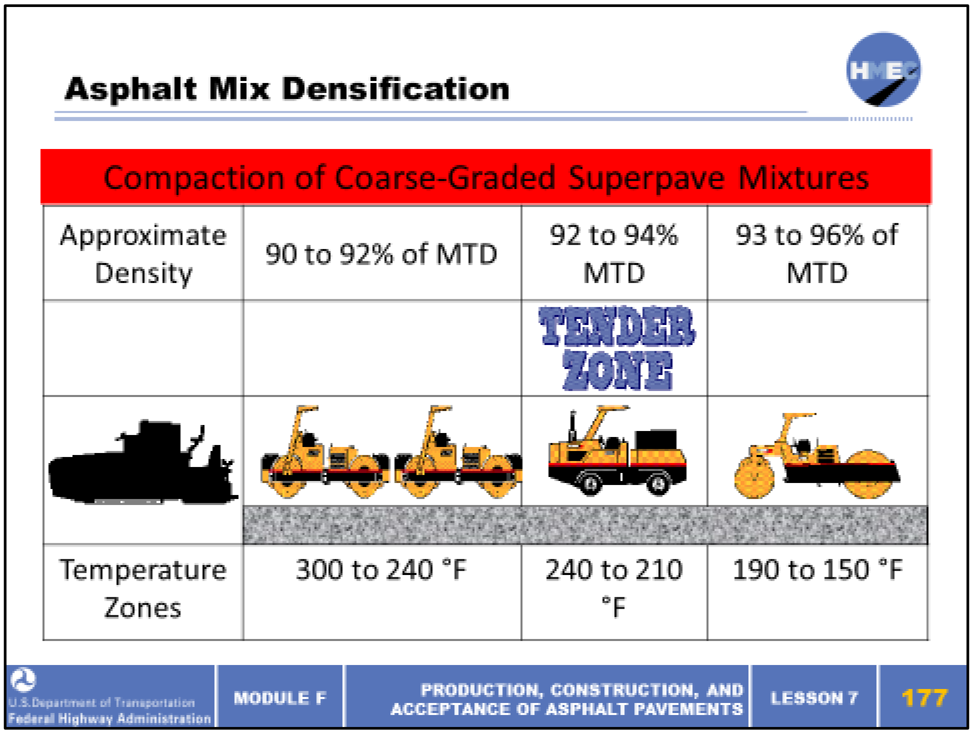
Slide 176



A "tender zone" was commonly reported. This is a temperature zone, typically from about 200 to 250 °F in which additional compactive effort is ineffective and in many cases is detrimental, as shown by the previous examples.

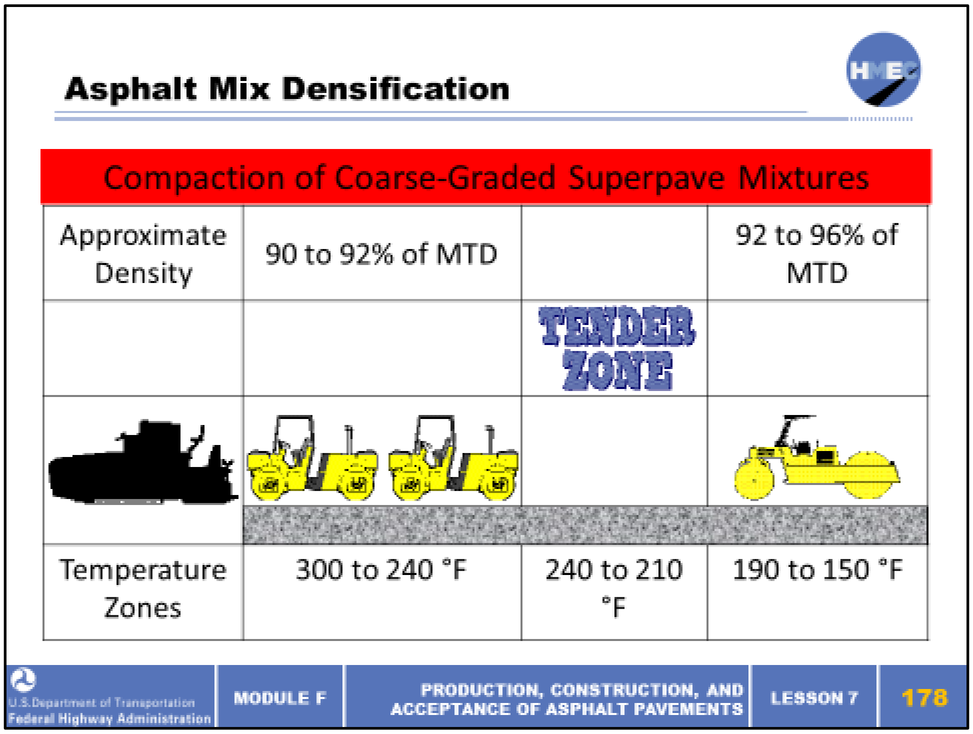
The common solution to this issue is to complete breakdown compaction prior to the temperature dropping below 250 °F, then temporarily stop compaction until the temperature is about 220 °F and perform the finish rolling as illustrated in this slide. No rollers are used within the temperature sensitive zone.

Slide 177



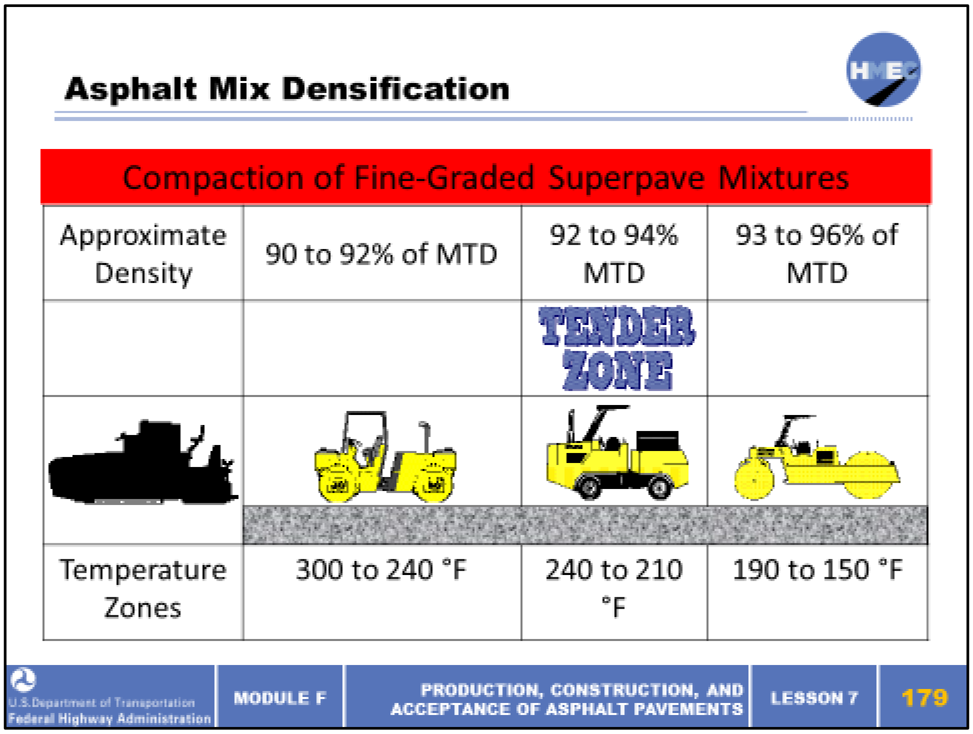
For coarse-graded mixtures, a pneumatic rubber tired roller can be used within the temperature sensitive zone for continued densification of the mat.

Slide 178



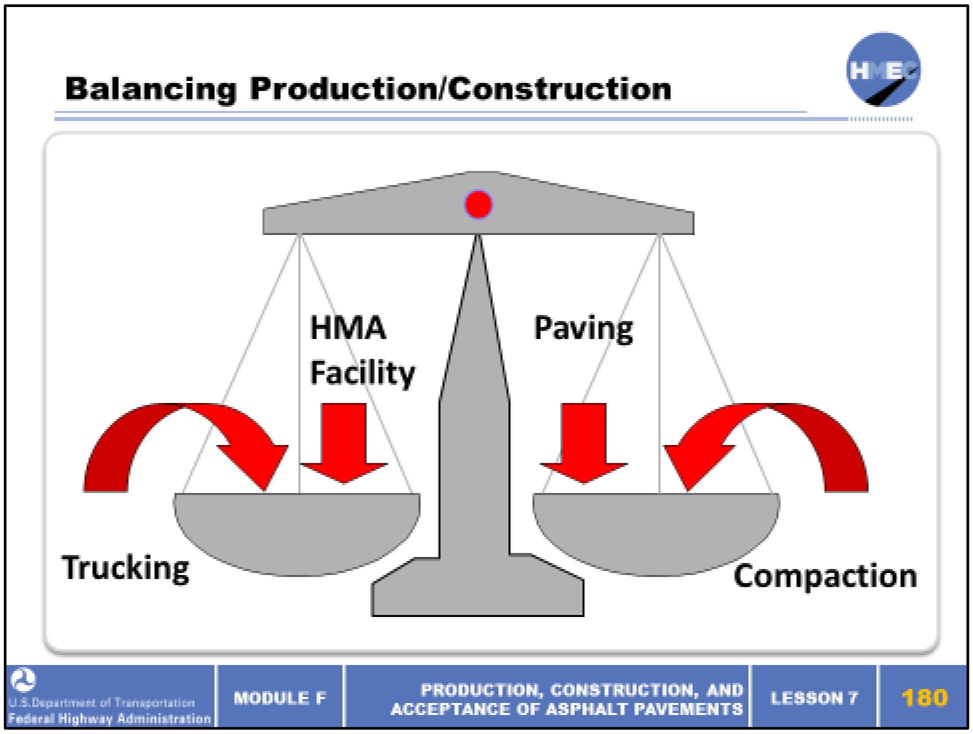
For this case, no steel roller should be utilized to roll the mix within the temperature sensitive zone or tender zone.

Slide 179



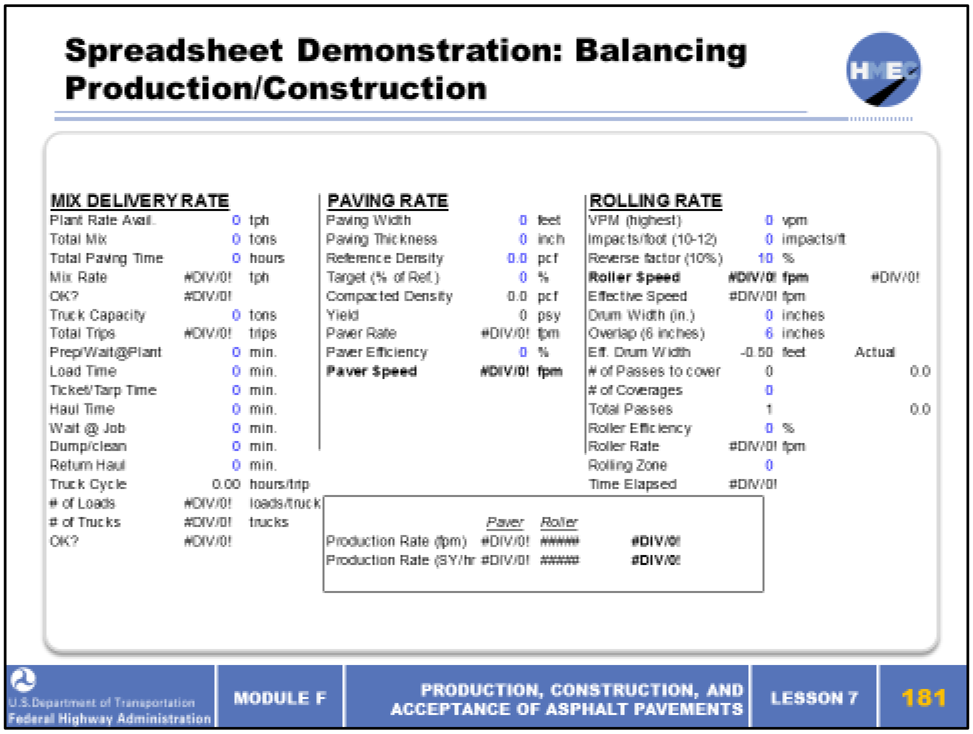
Similar to the coarse-graded mixtures, a pneumatic rubber-tired roller is used for continuing the densification process in the tender zone of the mix.

Slide 180



Although fairly simple to calculate on paper, balancing production rates in real life is much more difficult, due to the number of complicating factors that arise daily. Trucks tend to bunch up and arrive all at once, the paver speed can vary, and rollers do need to be re-watered. The best way to deal with this is to buffer the operation with a slightly higher roller production rate, enabling you to deal with these situations as they arise. Setting up a paving operation with too low a compaction production rate is not recommended.

Slide 181


Spreadsheet Demonstration: Balancing Production/Construction
HMEC
MIX DELIVERY RATE
Plant Rate Avail. 0 tph
Total Mix 0 tons
Total Paving Time 0 hours
Mix Rate #DIV/0! tph
OK? #DIV/0!
Truck Capacity 0 tons
Total Trips #DIV/0! trips
Prep/Wait@Plant 0 min.
Load Time 0 min.
Ticket/Tarp Time 0 min.
Haul Time 0 min.
Wait @ Job 0 min.
Dump/clean 0 min.
Return Haul 0 min.
Truck Cycle 0.00 hours/trip
# of Loads #DIV/0! loads/truck
# of Trucks #DIV/0! trucks
OK? #DIV/0!
PAVING RATE
Paving Width 0 feet
Paving Thickness 0 inch
Reference Density 0.0 pcf
Target (% of Ref.) 0 %
Compacted Density 0.0 pcf
Yield 0 psy
Paver Rate #DIV/0! fpm
Paver Efficiency 0 %
Paver Speed #DIV/0! fpm
ROLLING RATE
VPM (highest) 0 vpm
impacts/foot (10-12) 0 impacts/ft
Reverse factor (10%) 10 %
Roller Speed #DIV/0! fpm #DIV/0!
Effective Speed #DIV/0! fpm
Drum Width (in.) 0 inches
Overlap (6 inches) 6 inches
Eff. Drum Width -0.50 feet Actual
# of Passes to cover 0 0.0
# of Coverages 0
Total Passes 1 0.0
Roller Efficiency 0 %
Roller Rate #DIV/0! fpm
Rolling Zone 0
Time Elapsed #DIV/0!
Paver Roller
Production Rate (fpm) #DIV/0! ###### #DIV/0!
Production Rate (SY/hr #DIV/0! ###### #DIV/0!
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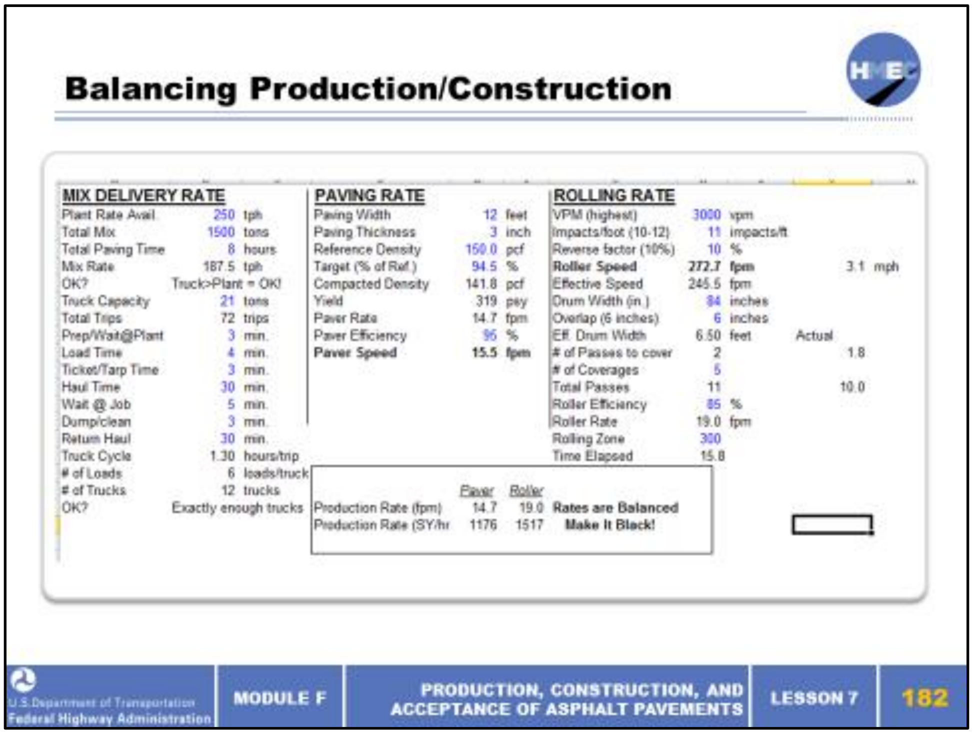
Slide 182


Balancing Production/Construction
MIX DELIVERY RATE
Plant Rate Avail. 250 tph
Total Mix 1500 tons
Total Paving Time 8 hours
Mix Rate 187.5 tph
OK? Truck>Plant = OK!
Truck Capacity 21 tons
Total Trips 72 trips
Prep/Wait@Plant 3 min.
Load Time 4 min.
Ticket/Tarp Time 3 min.
Haul Time 30 min.
Wait @ Job 5 min.
Dump/clean 3 min.
Return Haul 30 min.
Truck Cycle 1.30 hours/trip
# of Loads 6 loads/truck
# of Trucks 12 trucks
OK? Exactly enough trucks
PAVING RATE
Paving Width 12 feet
Paving Thickness 3 inch
Reference Density 150.0 pcf
Target (% of Ref.) 94.5 %
Compacted Density 141.8 pcf
Yield 319 psy
Paver Rate 14.7 fpm
Paver Efficiency 95 %
Paver Speed 15.5 fpm
ROLLING RATE
VPM (highest) 3000 vpm
Impacts/foot (10-12) 11 impacts/ft
Reverse factor (10%) 10 %
Roller Speed 272.7 fpm 3.1 mph
Effective Speed 245.5 fpm
Drum Width (in.) 84 inches
Overlap (6 inches) 6 inches
Eff. Drum Width 6.50 feet Actual
# of Passes to cover 2 1.8
# of Coverages 5
Total Passes 11 10.0
Roller Efficiency 85 %
Roller Rate 19.0 fpm
Rolling Zone 300
Time Elapsed 15.8
Paver Roller
Production Rate (fpm) 14.7 19.0 Rates are Balanced
Production Rate (SY/hr 1176 1517 Make It Black!
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MIX DELIVERY RATE

Plant Rate Avail.	250	tph
Total Mix	1500	tons
Total Paving Time	8	hours
Mix Rate	187.5	tph
OK?	Truck>Plant = OK!	
Truck Capacity	21	tons
Total Trips	72	trips
Prep/Wait@Plant	3	min.
Load Time	4	min.
Ticket/Tarp Time	3	min.
Haul Time	30	min.
Wait @ Job	5	min.
Dump/clean	3	min.
Return Haul	30	min.
Truck Cycle	1.30	hours/trip
# of Loads	6	loads/truck
# of Trucks	12	trucks
OK?	Exactly enough trucks	

PAVING RATE

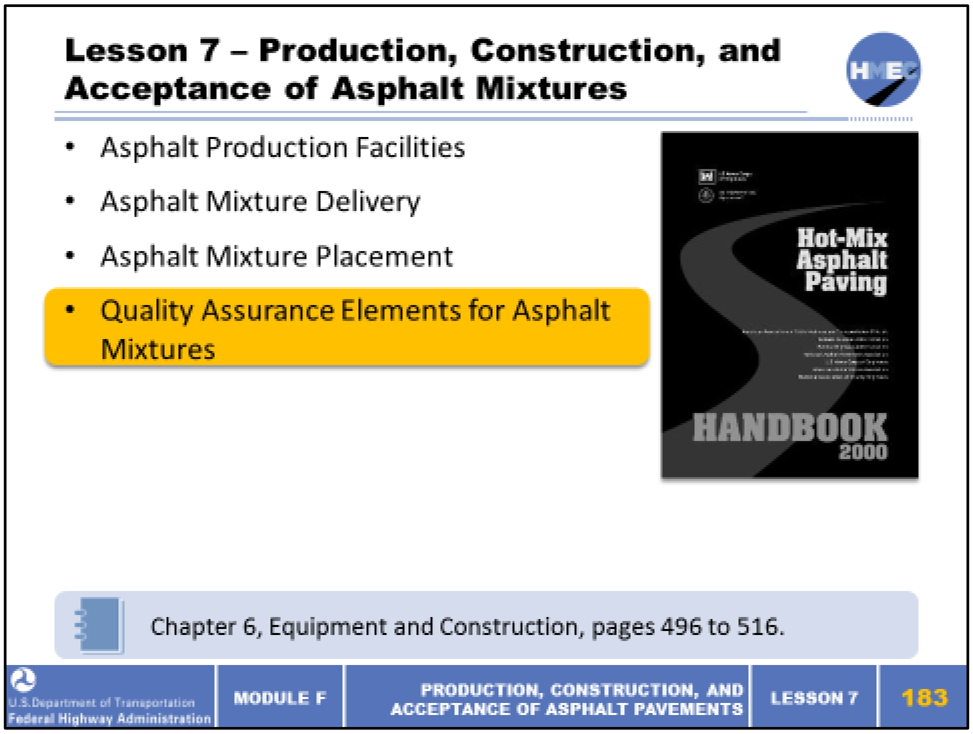
Paving Width	12	feet
Paving Thickness	3	inch
Reference Density	150.0	pcf
Target (% of Ref.)	94.5	%
Compacted Density	141.8	pcf
Yield	319	psy
Paver Rate	14.7	fpm
Paver Efficiency	95	%
Paver Speed	**15.5**	**fpm**

ROLLING RATE

VPM (highest)	3000	vpm		
Impacts/foot (10-12)	11	impacts/ft		
Reverse factor (10%)	10	%		
Roller Speed	**272.7**	**fpm**	3.1	mph
Effective Speed	245.5	fpm		
Drum Width (in.)	84	inches		
Overlap (6 inches)	6	inches		
Eff. Drum Width	6.50	feet	Actual	
# of Passes to cover	2		1.8	
# of Coverages	5			
Total Passes	11		10.0	
Roller Efficiency	85	%		
Roller Rate	19.0	fpm		
Rolling Zone	300			
Time Elapsed	15.8			

	Paver	*Roller*	
Production Rate (fpm)	14.7	19.0	**Rates are Balanced**
Production Rate (SY/hr	1176	1517	**Make It Black!**

Slide 183



Pages 496 to 516 of Chapter 6 in the NCAT textbook is focused on mixture/material specifications, statistical concepts and quality assurance methods for judging the quality of the asphalt concrete mixtures and mat.

Slide 184

As noted in Module A, there are different types of specifications. Most acceptance specifications used for asphalt concrete mats or mixtures can be grouped into two general types: method specification and end-result specification. The method specification dictate to the contractor what equipment and methods should be used to place the asphalt concrete mat. Method specifications are still used but are becoming less common. End-result specifications identify an end-result condition or property of the asphalt concrete mat and are becoming the more common type used for asphalt concrete mixtures.

This section only focuses on the end-result type of specification and the properties commonly used to evaluate the quality of the mat, or answering the question: Did we get what we ordered? Relative to asphalt mixtures and mats, the mix quality characteristics are set up or identified to estimate that the product will provide the expected service life as designated by the owner. The expected service life of the new asphalt concrete mat/mixture or asphalt concrete overlay is defined by its resistance to rutting, fatigue cracking, transverse cracking, raveling, bleeding/flushing, and roughness. The previous lessons under Module F have identified the important parameters and/or properties related to performance or resistance to various distresses. This section summarizes from Module A those parameters or properties that are included in many agencies acceptance plans for defining the quality characteristics of an asphalt concrete mixture and mat.

Module A on quality assurance described the elements of a quality assurance program and how to interpret the results in terms of evaluating the acceptance of a specific product.

Slide 185

Discussion: Mix Quality Characteristics

HMEC

- What quality characteristics or properties are included in your agency's acceptance plan?
- Are the characteristics classified as:
  - Volumetric
  - Mechanistic
  - Empirical
- Where should the mix be sampled to evaluate the acceptability of the mix for each of the quality characteristics identified?

Let's discuss quality characteristics or properties.

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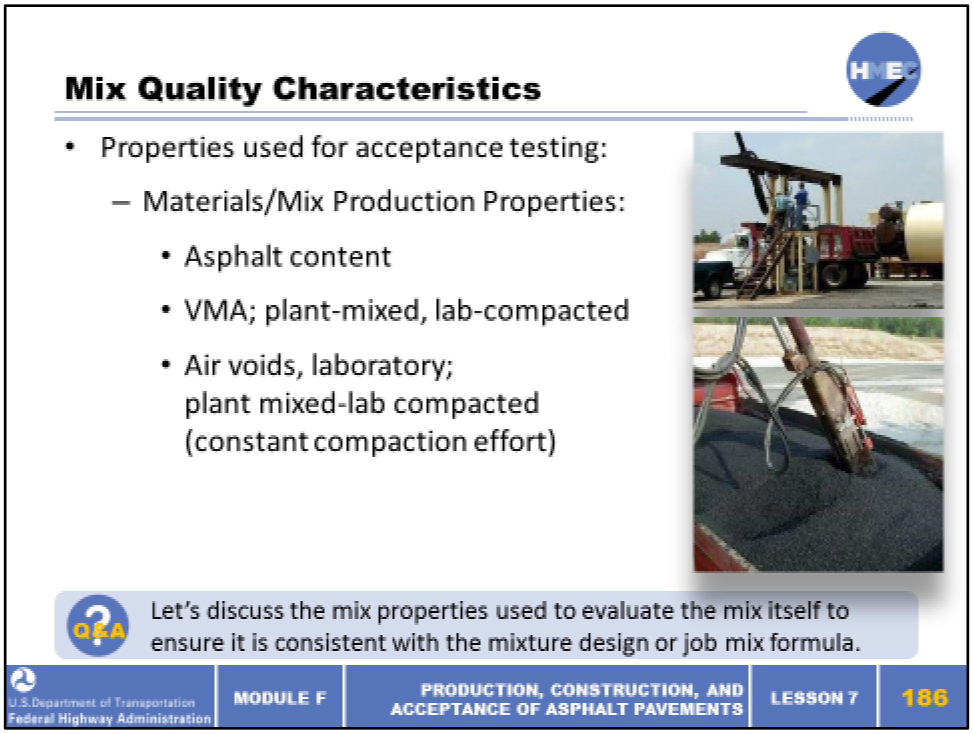
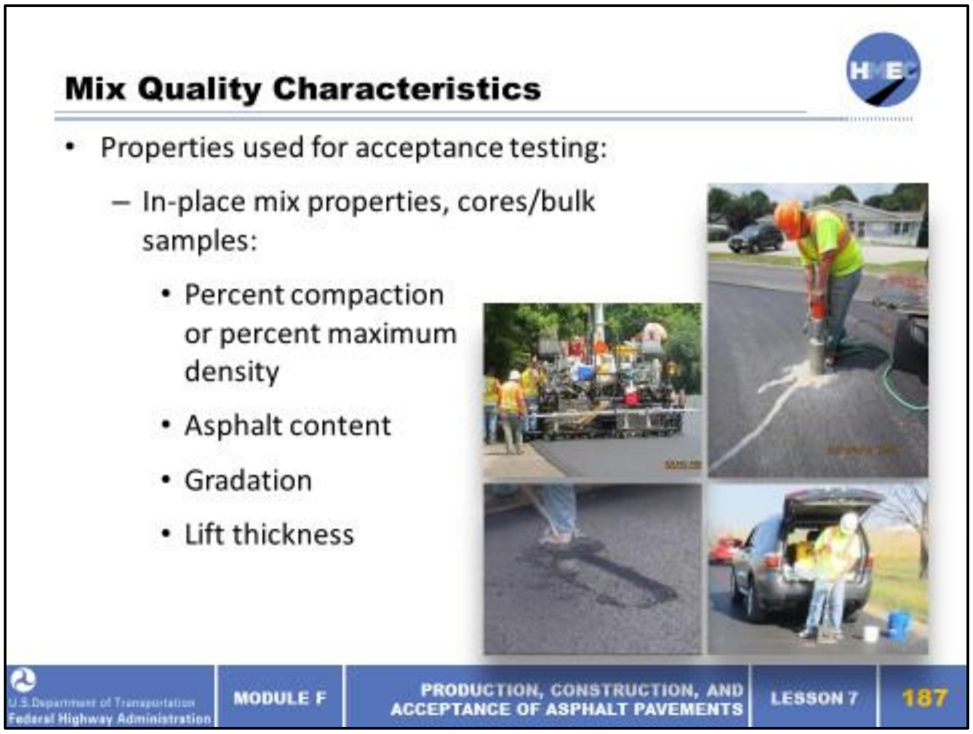
Slide 186



This slide lists the mix properties that are used to evaluate the mix itself to ensure it is consistent with the mixture design or job mix formula that has been adjusted to production conditions. The asphalt mixture can be sampled at the plant for evaluating these volumetric properties. Samples of the mixture can be taken manually with shovels (the more common) or use of automated devices which can collect samples within the center of the truck rather than along the surface of the mixture in the bed of the trucks. When shovels are used to collect samples of the mix, trenches should be formed or excavated prior to collecting the sample to ensure it is representative of the mix deeper in the bed, but the sample collected is heavily dependent of the technician so as to not get a segregated or biased sample.

The technician should identify whether the delivery trucks are being loaded properly, as discussed in the previous section of Lesson 7 on mixture delivery—loading and unloading trucks. The instructor should point out in the photograph showing the mix in the truck bed that the mix is properly against the front of the bed and against the tailgate.

Slide 187

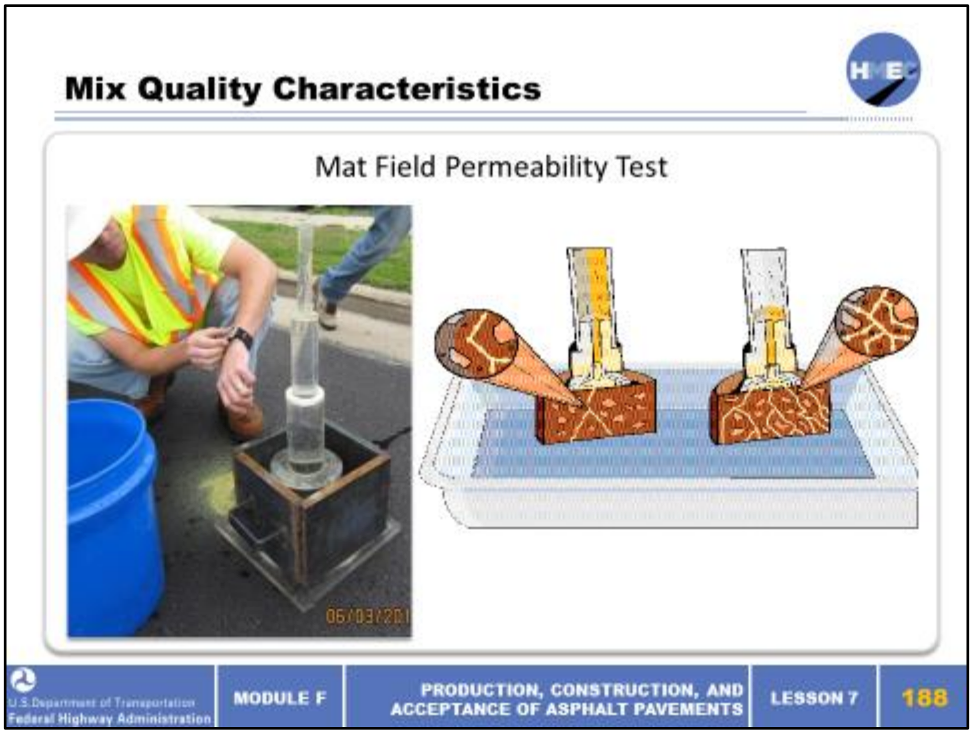


The contractor typically uses a nuclear or non-nuclear density gauge for controlling the rolling-compaction operation, while most agencies require the use of cores for measuring the density to determine the percent compaction or percent maximum density for acceptance. Cores are also used to measure the thickness of the lift or layer for those agencies that include lift thickness as part of their acceptance plan.

Gradation is the other parameter or volumetric property included in some agencies acceptance plan. Bulk samples of the asphalt mixture for the gradation tests are usually taken at the same time and location for which the asphalt content is determined for acceptance. As noted from the discussion during the previous slide; however, the best location for measuring the gradation is behind the paver prior to the roller passing over the mat because of potential segregation or changes in the gradation through the paver. Two types of samples are extracted: the more accurate or common is to place a plate on the existing surface and have the paver pass over it for collecting the mix (plate not shown in the photo behind the paver) or to use a shovel and recover the mix from a designated area (shown in one of the photos included in the slide). The majority of agencies use the plate method for collecting a sample of the mix for measuring the gradation when specified for an acceptance plan.

Permeability is considered a mix quality characteristic, but is not included on this slide because no known agency uses mat permeability as an acceptance parameter. Mat permeability tests take much longer to perform and exhibit a higher variability than some of the other mix quality characteristics. The instructor, however, should note its importance rather than percent compaction or percent maximum density in terms of the hardening and stripping or moisture damage. Details on the mat permeability test are provided in the following slide.

Slide 188



The test equipment for the field permeability test consists of different diameters of clear cylinders stacked on one another so the time can be measured as the head of water falls through a specific cylinder. Putty or a sealer material is placed on the bottom of the base plate to ensure it is sealed to the surface of the mat. A steel square frame is placed on the top of the base plate of the permeability device to keep the base plate tightly sealed to the mat. Water is placed in the cylinders with a funnel. The time for the head of water to fall over a specific distance is measured to calculate the permeability.

Although mat permeability is considered a quality characteristic of the in place mixture, it is not used for acceptance by any State agency at this time. Mat permeability measurements are primarily used for forensic investigations as a diagnostic tool.

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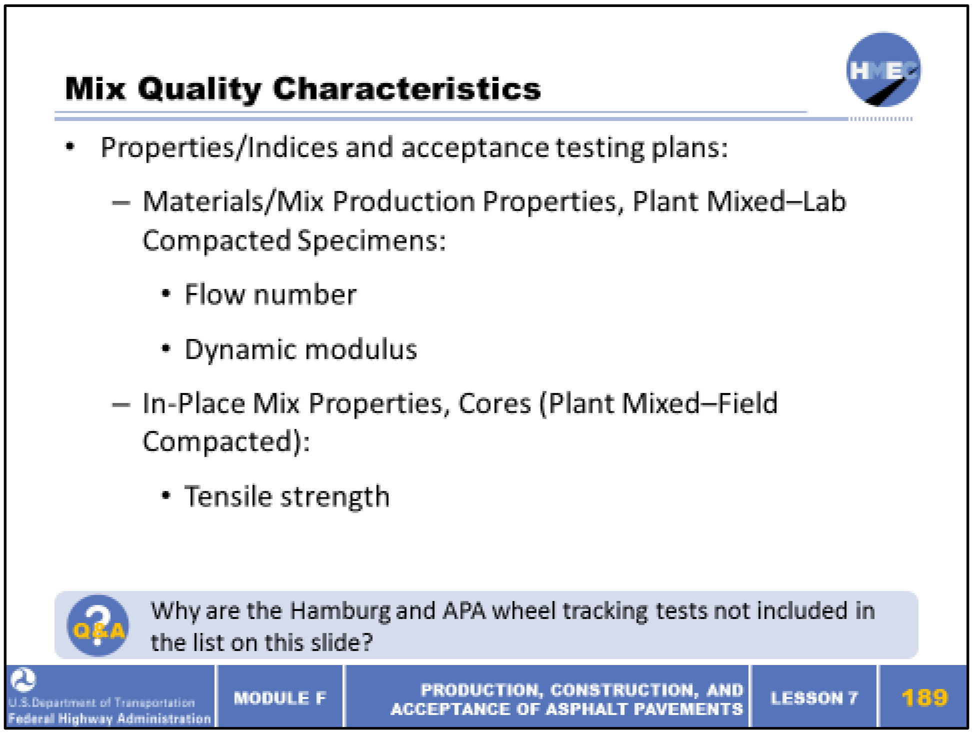
Slide 189



Few, if any, agencies use flow number and dynamic modulus within their acceptance plan because these have only recently been proposed or used for mixture design. However, outcomes from the Hamburg and APA wheel tracking tests have been used for mixture design verification. Similarly, these empirical tests are typically not used in an agency's acceptance plan.

Slide 190

This photo illustrates an inertial-based profiler system. Ride specifications will provide a better ride and result in less pavement damage due to excessive wear, snowplow damage, etc.

Skid resistance or friction has been excluded from most agencies acceptance plans or programs. However, it is an important surface characteristic.

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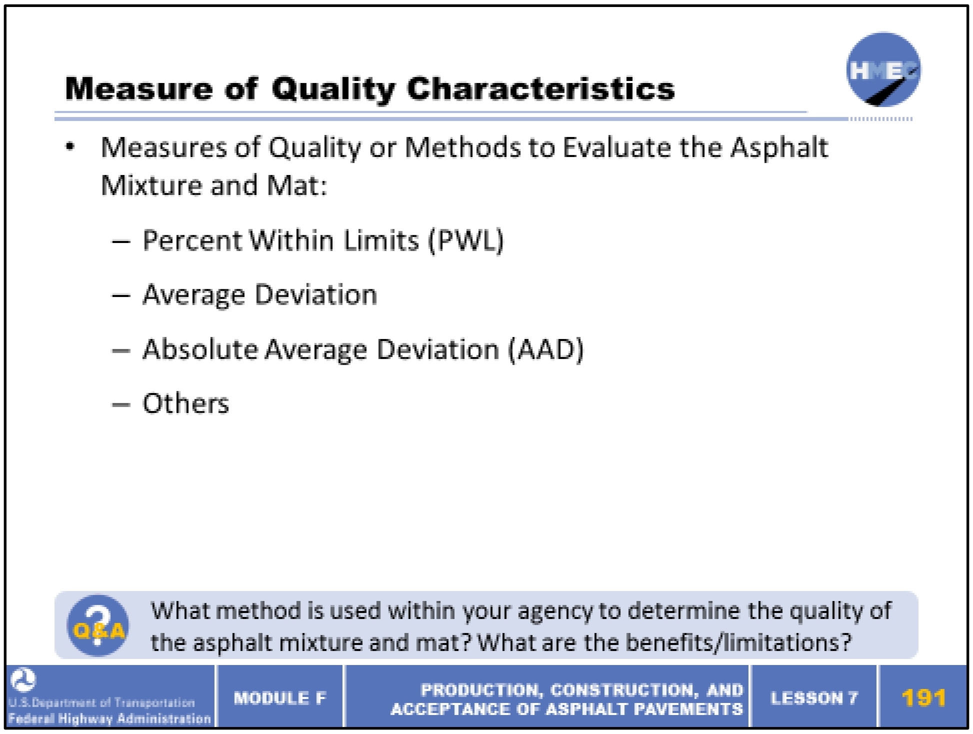
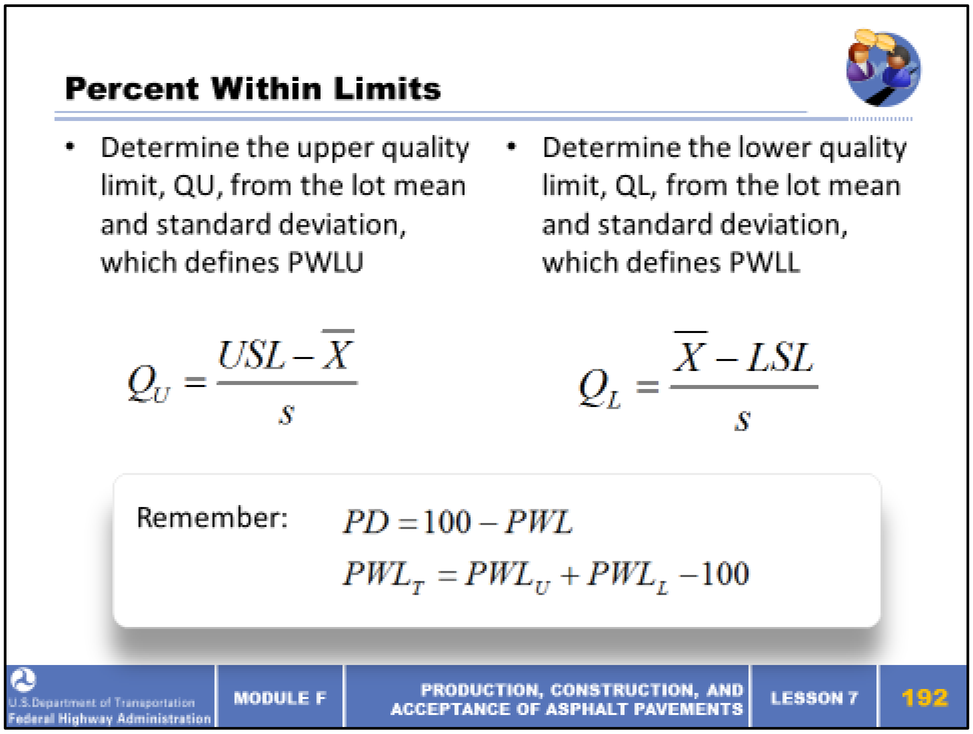
Slide 191



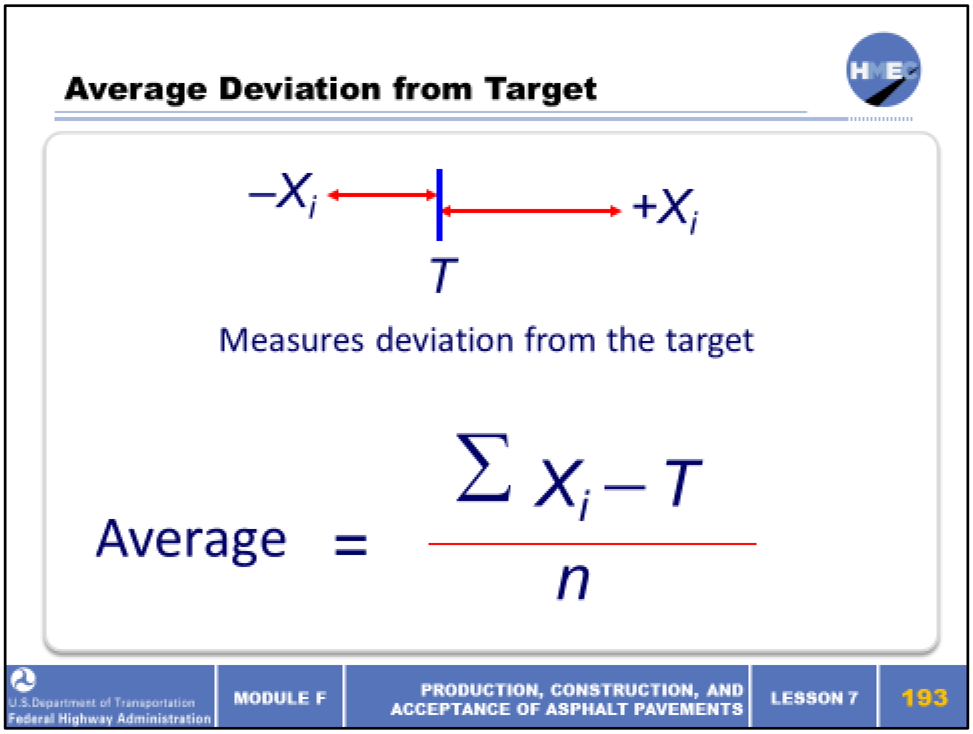
The 2004 NCHRP synthesis referenced and reported the usage of acceptance compliance measures for HMA mixture properties that were provided in an earlier slide. Most of the methods used to evaluate or determine the acceptability of the asphalt mix and mat can be grouped into three basic categories: Percent within limits (which is the most commonly used method), average deviation, and absolute average deviation.

Slide 192



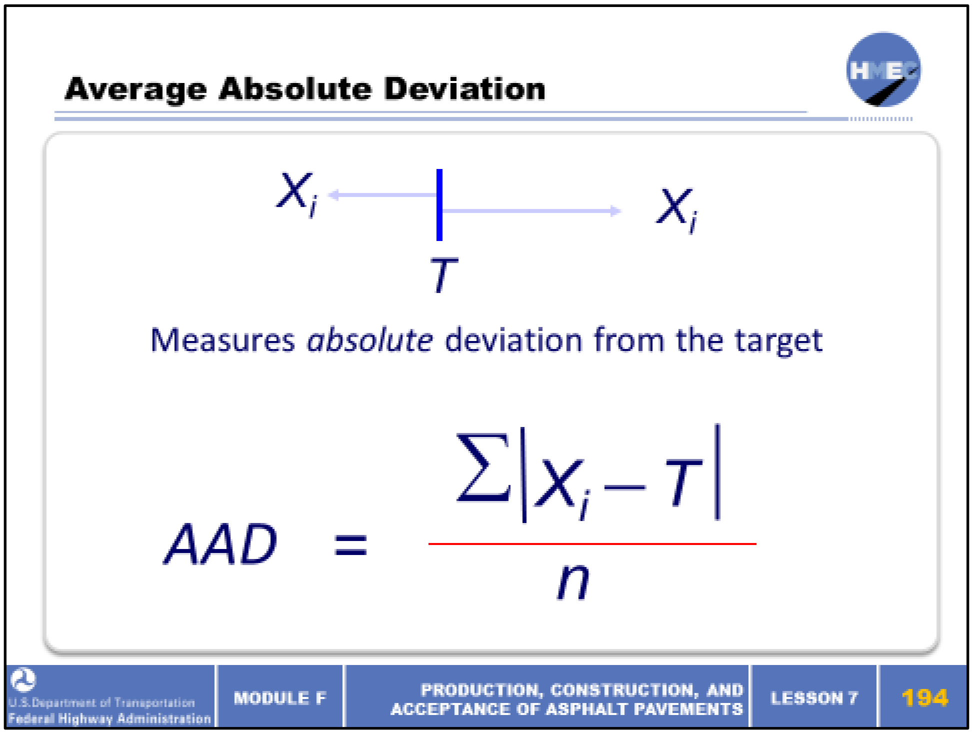
The area under the normal curve can be calculated (as shown in Module A) to determine the percentage of the population that is within certain limits. Similarly, the percentage of the lot that is within the specification limits can be estimated. Instead of using the Z-value and the standard normal curve, a similar statistic, the quality index (*Q*), is used to estimate PWL. This was discussed and demonstrated under Module A.

Slide 193



If two test results in the morning are below the target value, there is a strong incentive for the contractor to increase the process mean in the afternoon in an effort to get two higher test results so that the average of the four tests for the lot will be near the target value. In essence, this acceptance approach encourages the contractor to increase process variability by making frequent adjustments to the process mean.

Slide 194


Average Absolute Deviation
HMEC
X_i
T
X_i
Measures *absolute* deviation from the target
$$AAD = \frac{\sum |X_i - T|}{n}$$
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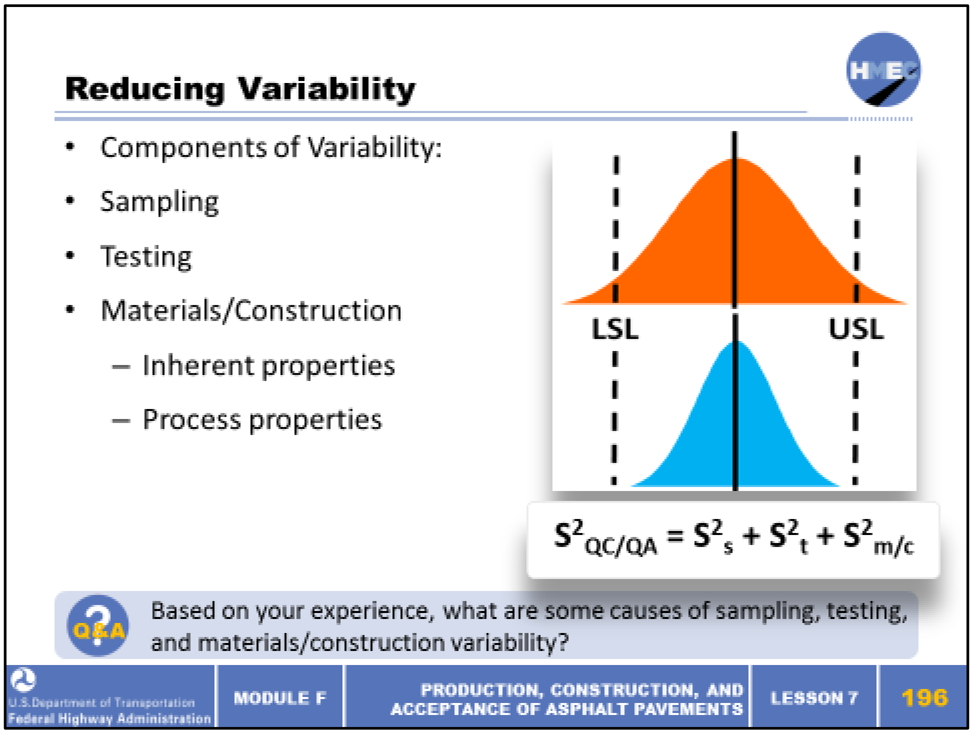
Slide 195

Inspection is a critical element of any acceptance plan, which has been ignored in the previous slides discussing the mix quality characteristics.

Many agencies are starting to use contractor's test results for acceptance. In this case, most contractors perform the same tests used for control, which is not encouraged. The instructor should identify the perception of the contractor when they are required to sample the mix and even perform the tests for acceptance which affects the contractors decision for actually controlling the mixture to make the right decisions at the right time. For the option where the agency uses the contractor's test results for acceptance, the agency needs to perform verification testing.

Another important issue of the acceptance plan is calibration of equipment. Calibration should be included in order to avoid conflicts when QC and acceptance using different equipment to measure the roughness and other properties of the mixture/mat.

Slide 196



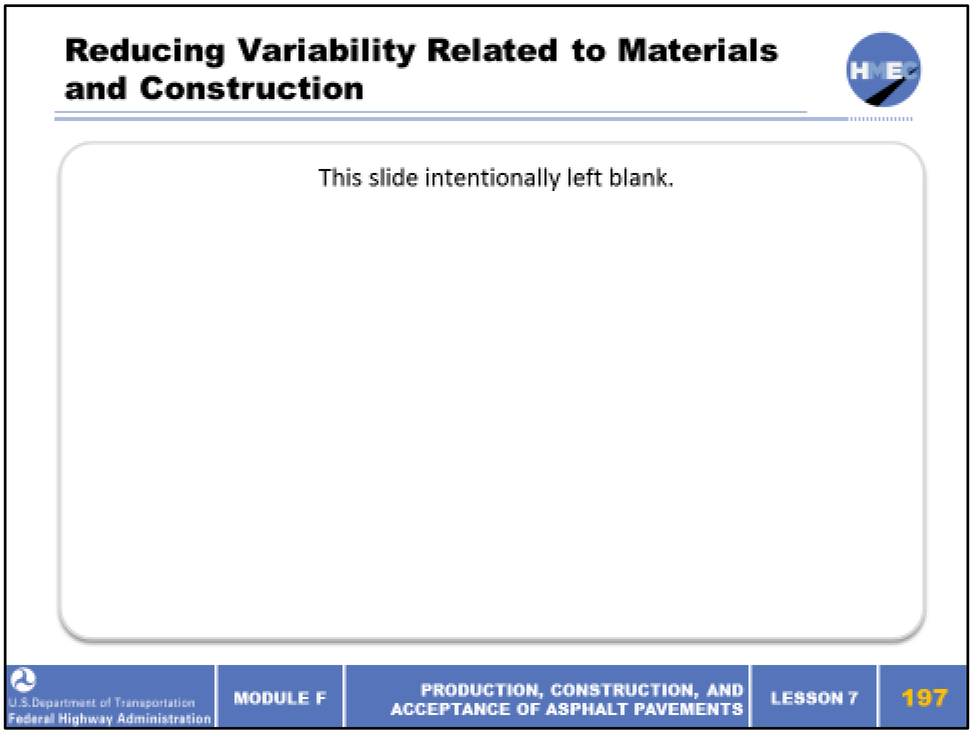
As indicated throughout this lesson and in Module A, variability can significantly impact the payment received for a pavement lot. Reducing variability in the finished HMA pavement can strongly improve chances of receiving a bonus. More importantly, reducing variability will usually result in longer-lasting pavements.

On the other hand, disregarding the impact of variability can lead to substantial penalties and even removal and replacement of the lot. As noted in Module A, the sources of variability during the construction of an HMA pavement are grouped into three sources: sampling, testing, and materials/construction. The materials and construction variability can be further grouped into two categories: inherent properties which relate to the material and cannot be controlled by the contractor unless the source is changed; and process which relate to properties or production processes that are under the control of the contractor. Both inherent and process properties were also defined and explained under Module A.

Ways to reduce the variability from each potential source are discussed in the following slides.

Sampling variability has been estimated to constitute 10 to 30% of the total variability associated with the finished HMA layer. Sampling location, method, size, and splitting method can all contribute to sampling variability. The testing variability is defined by the precision of the test procedure. Most AASHTO and ASTM methods or procedures have precision statements in determining the expected deviation test results. The variability from the materials/construction activities is the most important component of variability for which the contractor controls.

Slide 197


Reducing Variability Related to Materials and Construction
HMEC
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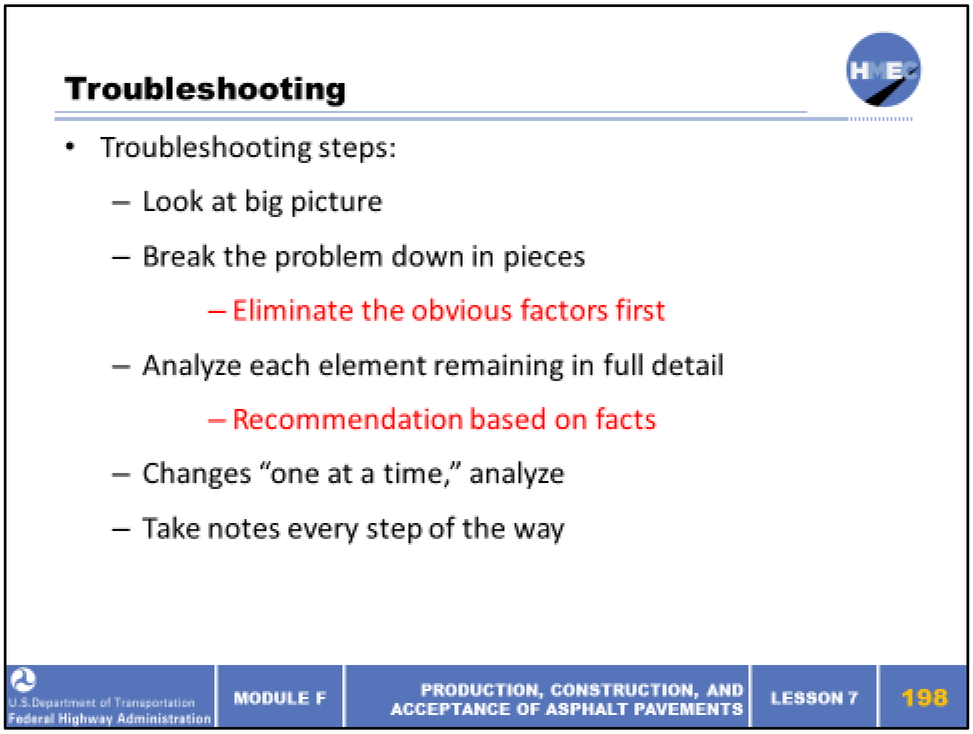
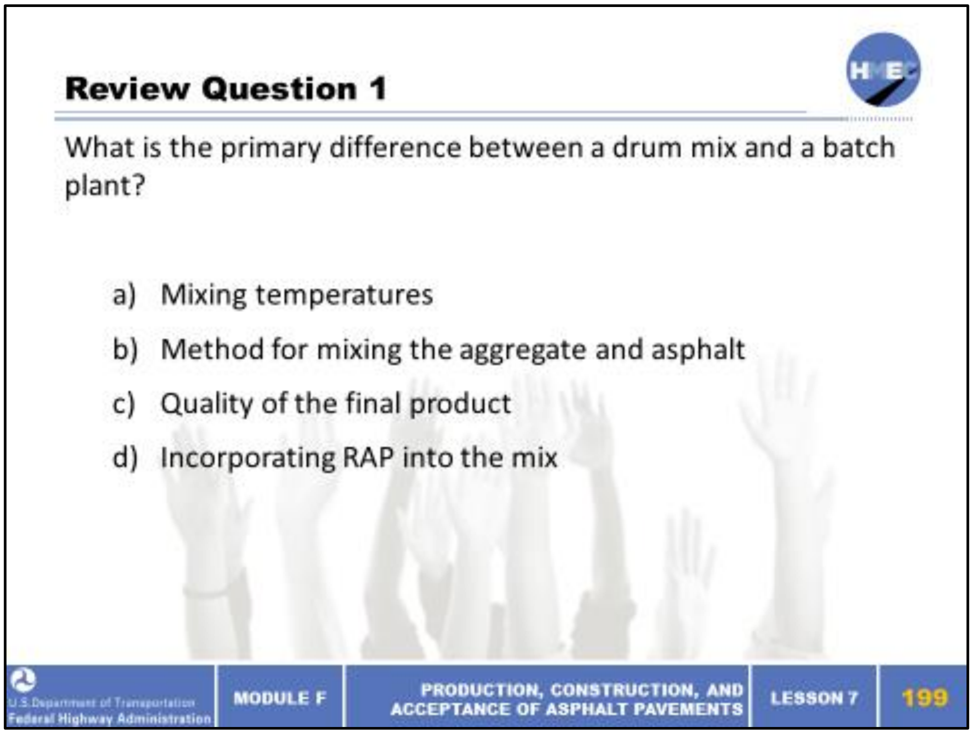
Slide 198



Charts are available from NHI Course 131045 related to the test outcome in terms of acceptance and control. These charts were created for tools in terms of interpreting the cause for values that exceed the limits for selected quality characteristics. However, no one chart can reproduce or simulate actual construction conditions so one needs to still go through the process; the charts just help speed up the process and help to look at the big picture.

Refer to Mat Problem and Troubleshooting Guide (Extracted with permission from NAPA) in the Resources folder on the tablet.

Slide 199


Review Question 1
What is the primary difference between a drum mix and a batch plant?
a) Mixing temperatures
b) Method for mixing the aggregate and asphalt
c) Quality of the final product
d) Incorporating RAP into the mix
U.S. Department of Transportation
Federal Highway Administration
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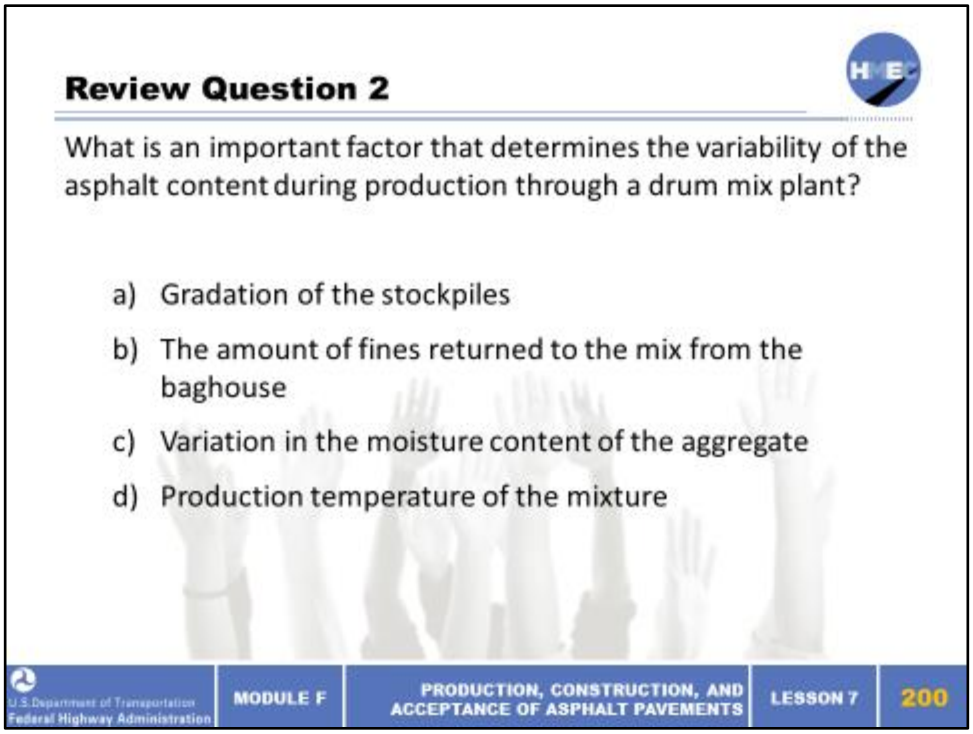
Slide 200


Review Question 2
What is an important factor that determines the variability of the asphalt content during production through a drum mix plant?
a) Gradation of the stockpiles
b) The amount of fines returned to the mix from the baghouse
c) Variation in the moisture content of the aggregate
d) Production temperature of the mixture
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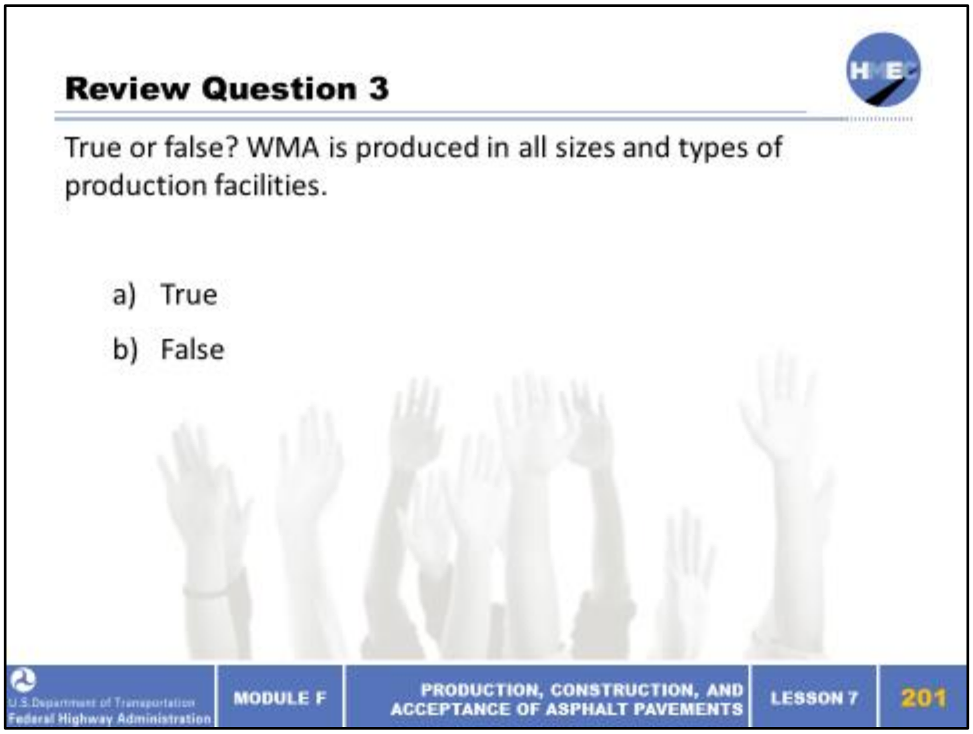
Slide 201


Review Question 3
True or false? WMA is produced in all sizes and types of production facilities.
a) True
b) False
U.S. Department of Transportation
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MODULE F
PRODUCTION, CONSTRUCTION, AND ACCEPTANCE OF ASPHALT PAVEMENTS
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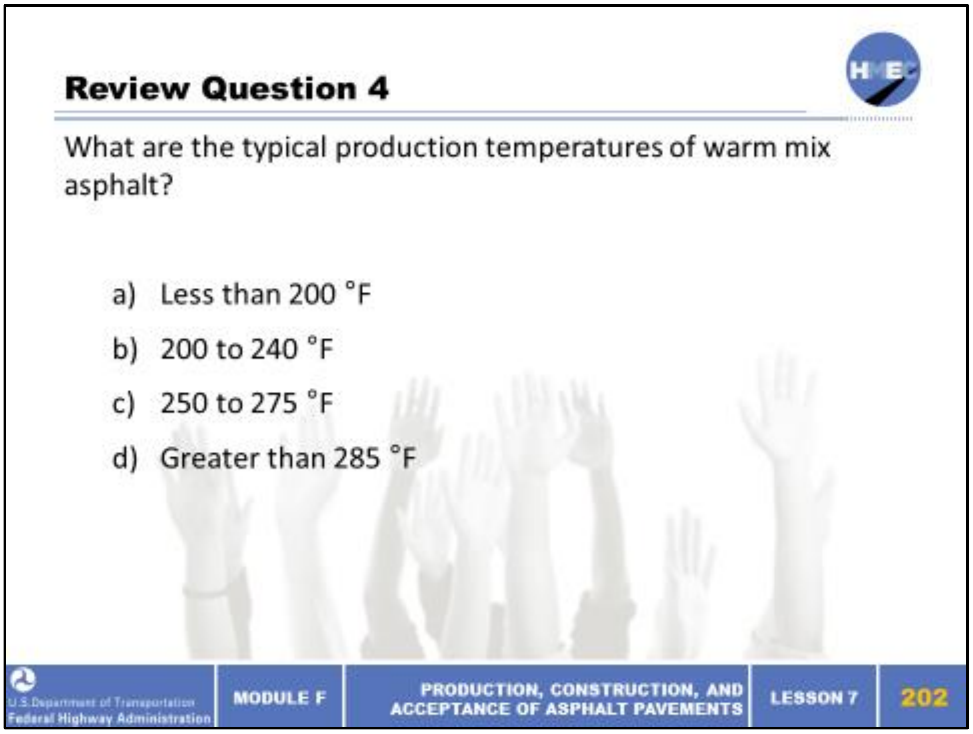
Slide 202


Review Question 4
What are the typical production temperatures of warm mix asphalt?
a) Less than 200 °F
b) 200 to 240 °F
c) 250 to 275 °F
d) Greater than 285 °F
U.S. Department of Transportation
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MODULE F
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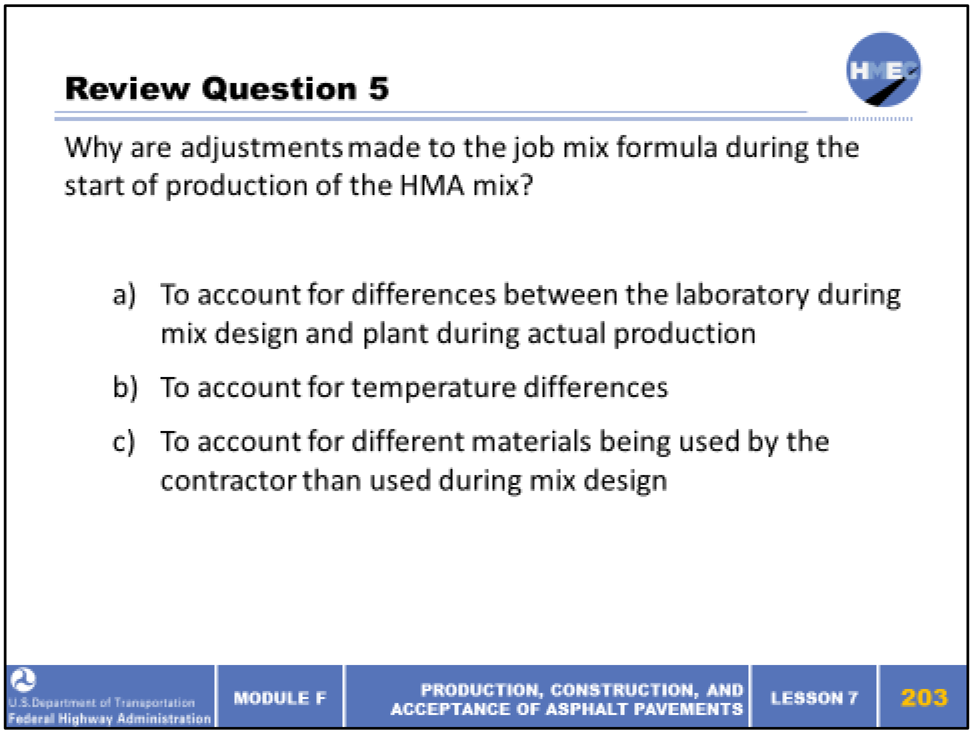
Slide 203


Review Question 5
Why are adjustments made to the job mix formula during the start of production of the HMA mix?
a) To account for differences between the laboratory during mix design and plant during actual production
b) To account for temperature differences
c) To account for different materials being used by the contractor than used during mix design
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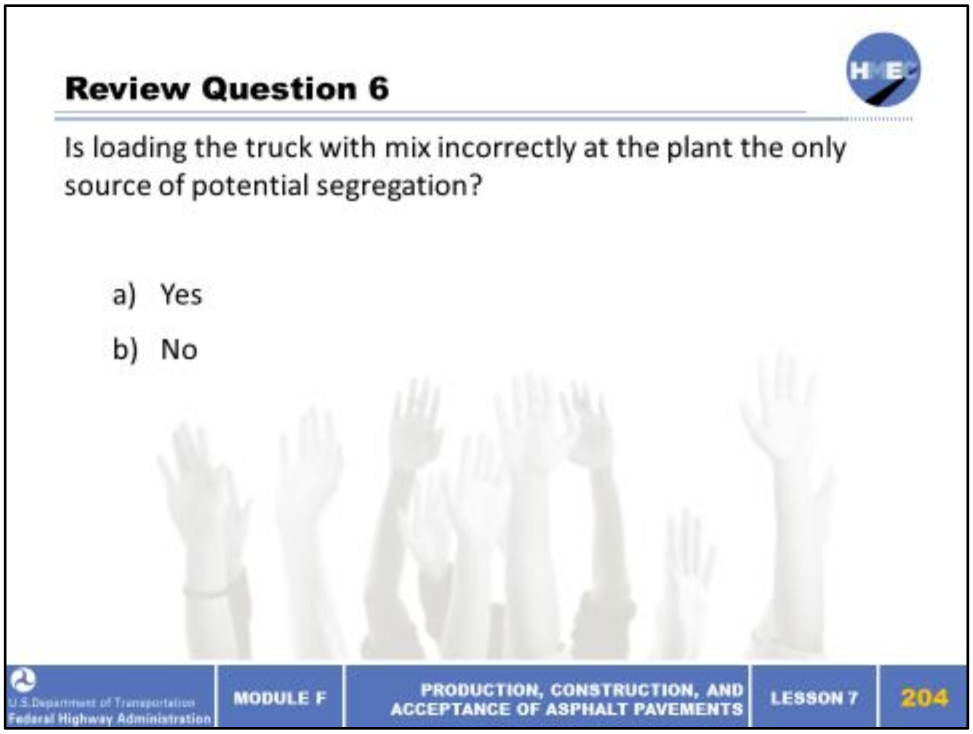
Slide 204


Review Question 6
Is loading the truck with mix incorrectly at the plant the only source of potential segregation?
a) Yes
b) No
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MODULE F
PRODUCTION, CONSTRUCTION, AND ACCEPTANCE OF ASPHALT PAVEMENTS
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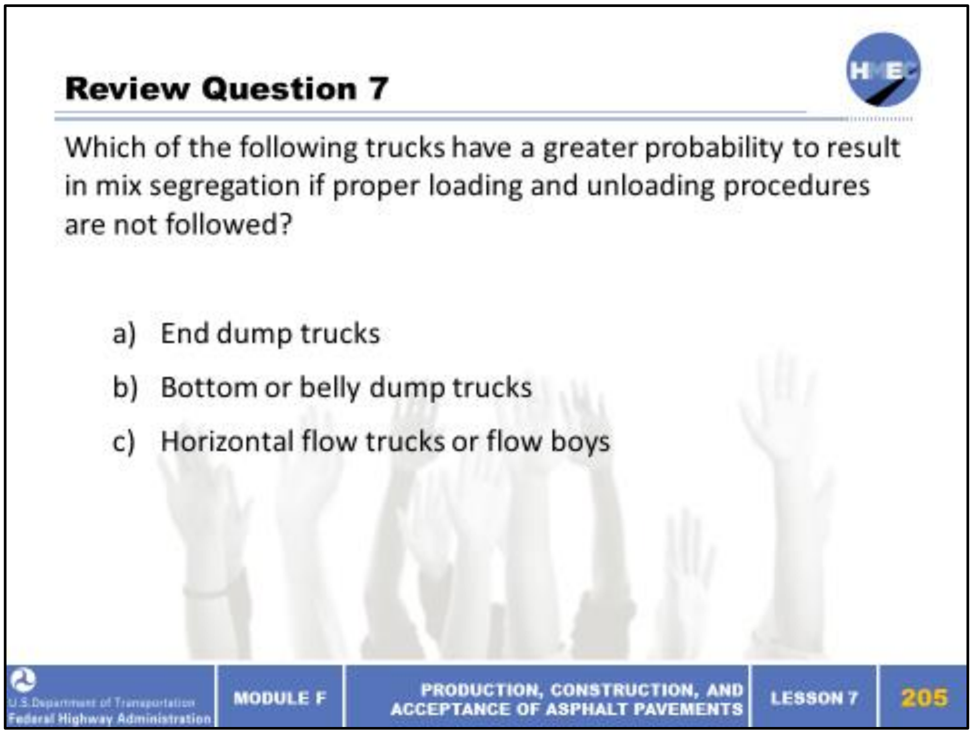
Slide 205


Review Question 7
Which of the following trucks have a greater probability to result in mix segregation if proper loading and unloading procedures are not followed?
a) End dump trucks
b) Bottom or belly dump trucks
c) Horizontal flow trucks or flow boys
U.S. Department of Transportation Federal Highway Administration
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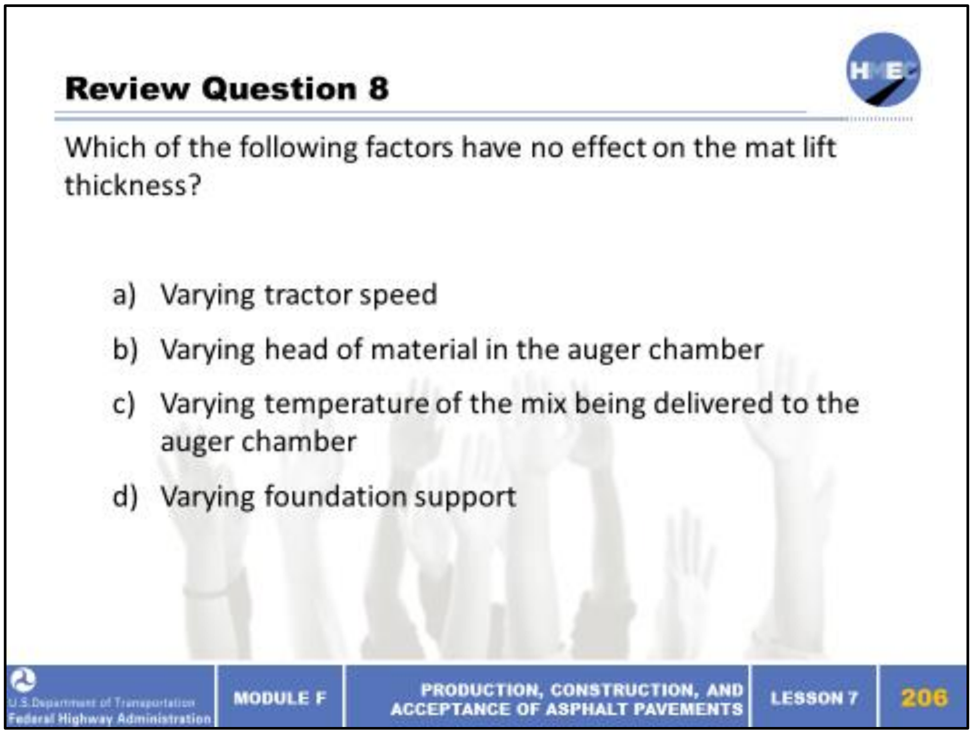
Slide 206


Review Question 8
Which of the following factors have no effect on the mat lift thickness?
a) Varying tractor speed
b) Varying head of material in the auger chamber
c) Varying temperature of the mix being delivered to the auger chamber
d) Varying foundation support
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PRODUCTION, CONSTRUCTION, AND ACCEPTANCE OF ASPHALT PAVEMENTS
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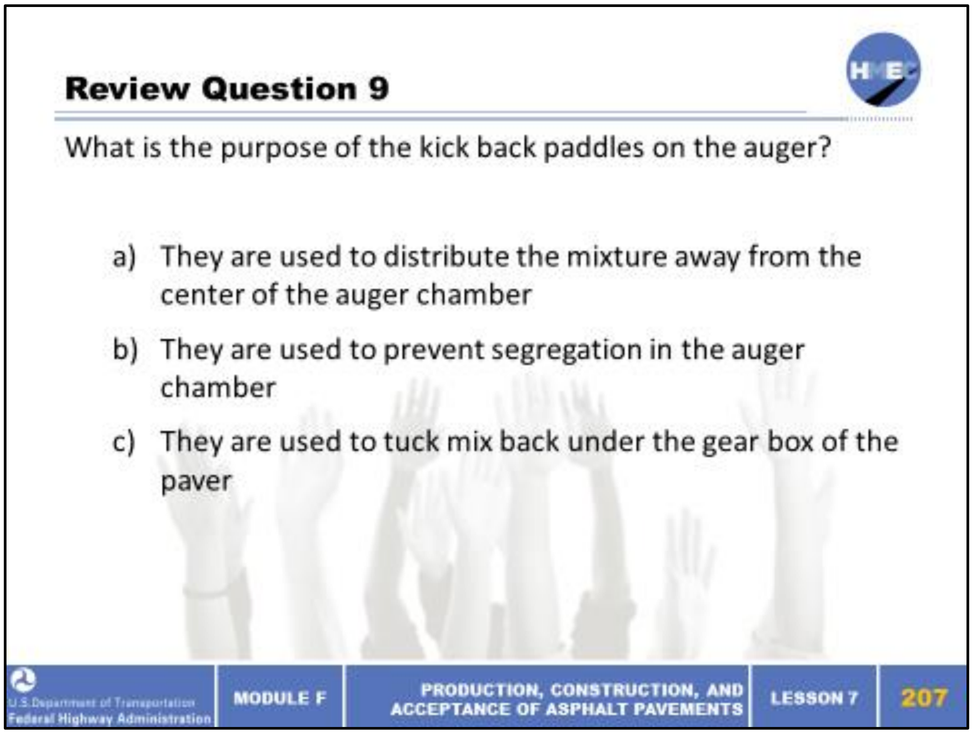
Slide 207


Review Question 9
What is the purpose of the kick back paddles on the auger?
a) They are used to distribute the mixture away from the center of the auger chamber
b) They are used to prevent segregation in the auger chamber
c) They are used to tuck mix back under the gear box of the paver
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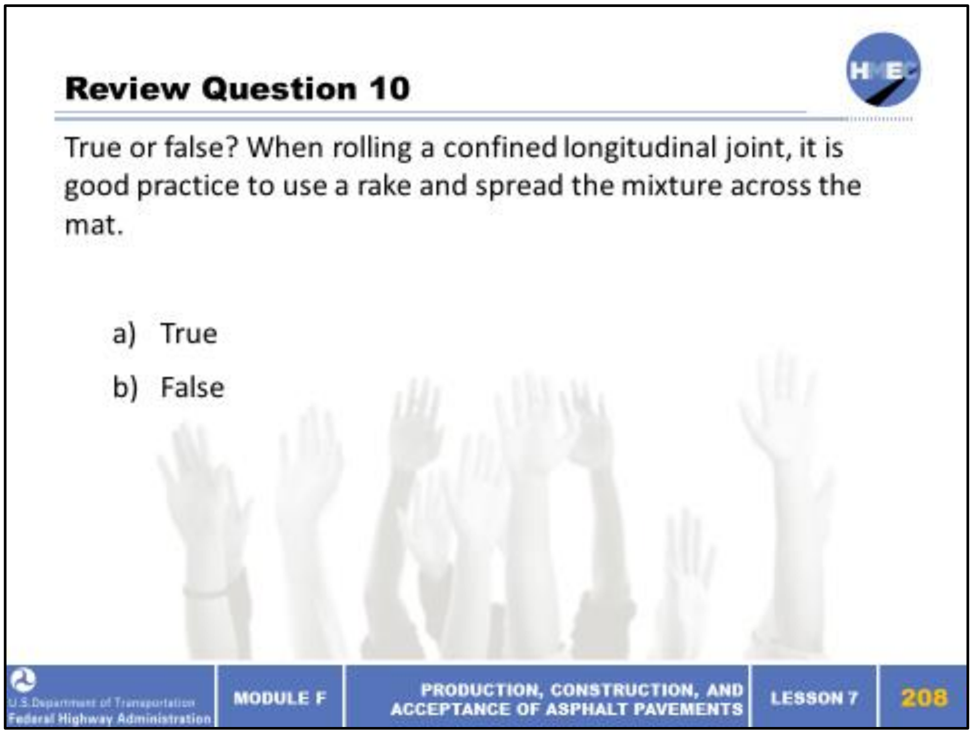
Slide 208


Review Question 10
True or false? When rolling a confined longitudinal joint, it is good practice to use a rake and spread the mixture across the mat.
a) True
b) False
U.S. Department of Transportation
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MODULE F
PRODUCTION, CONSTRUCTION, AND ACCEPTANCE OF ASPHALT PAVEMENTS
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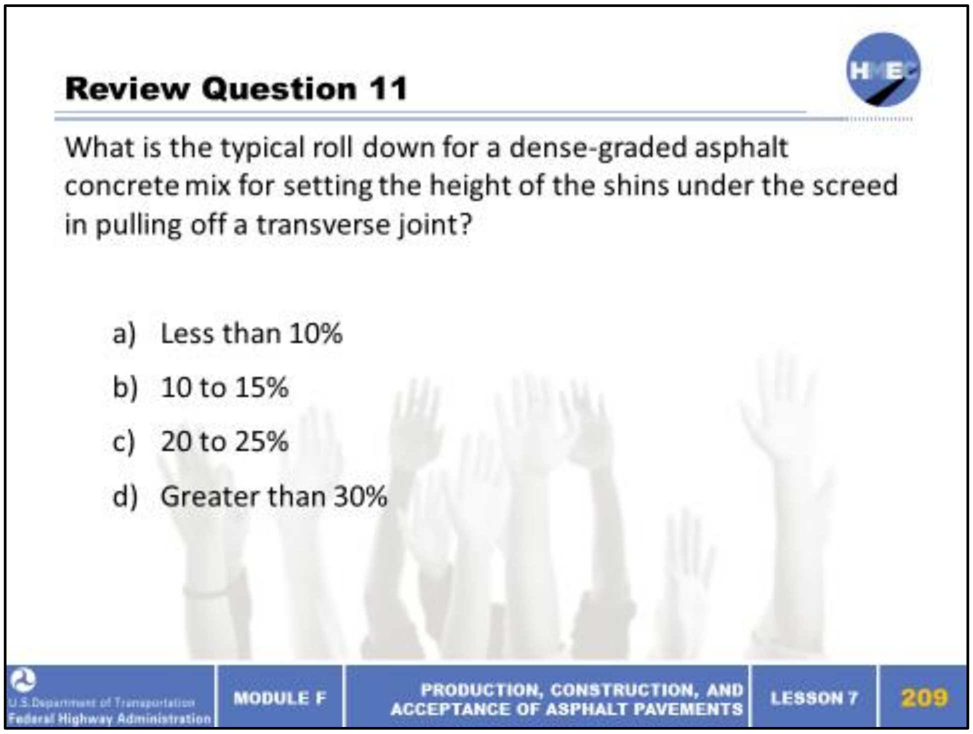
Slide 209


Review Question 11
What is the typical roll down for a dense-graded asphalt concrete mix for setting the height of the shins under the screed in pulling off a transverse joint?
a) Less than 10%
b) 10 to 15%
c) 20 to 25%
d) Greater than 30%
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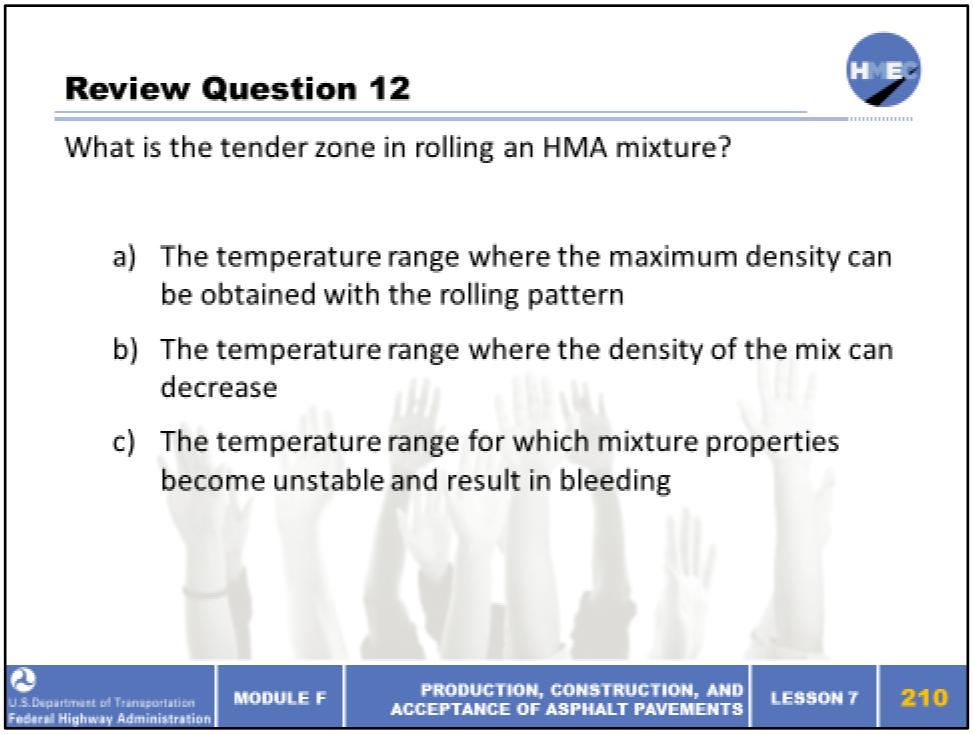
Slide 210


Review Question 12
What is the tender zone in rolling an HMA mixture?
a) The temperature range where the maximum density can be obtained with the rolling pattern
b) The temperature range where the density of the mix can decrease
c) The temperature range for which mixture properties become unstable and result in bleeding
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PRODUCTION, CONSTRUCTION, AND ACCEPTANCE OF ASPHALT PAVEMENTS
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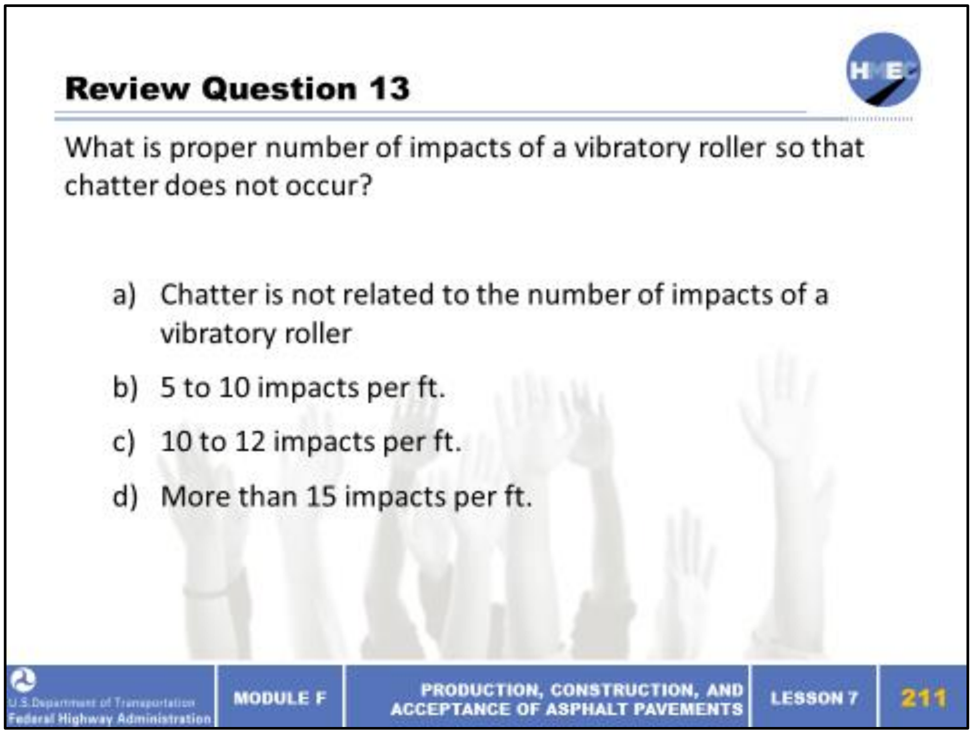
Slide 211


Review Question 13
What is proper number of impacts of a vibratory roller so that chatter does not occur?
a) Chatter is not related to the number of impacts of a vibratory roller
b) 5 to 10 impacts per ft.
c) 10 to 12 impacts per ft.
d) More than 15 impacts per ft.
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PRODUCTION, CONSTRUCTION, AND ACCEPTANCE OF ASPHALT PAVEMENTS
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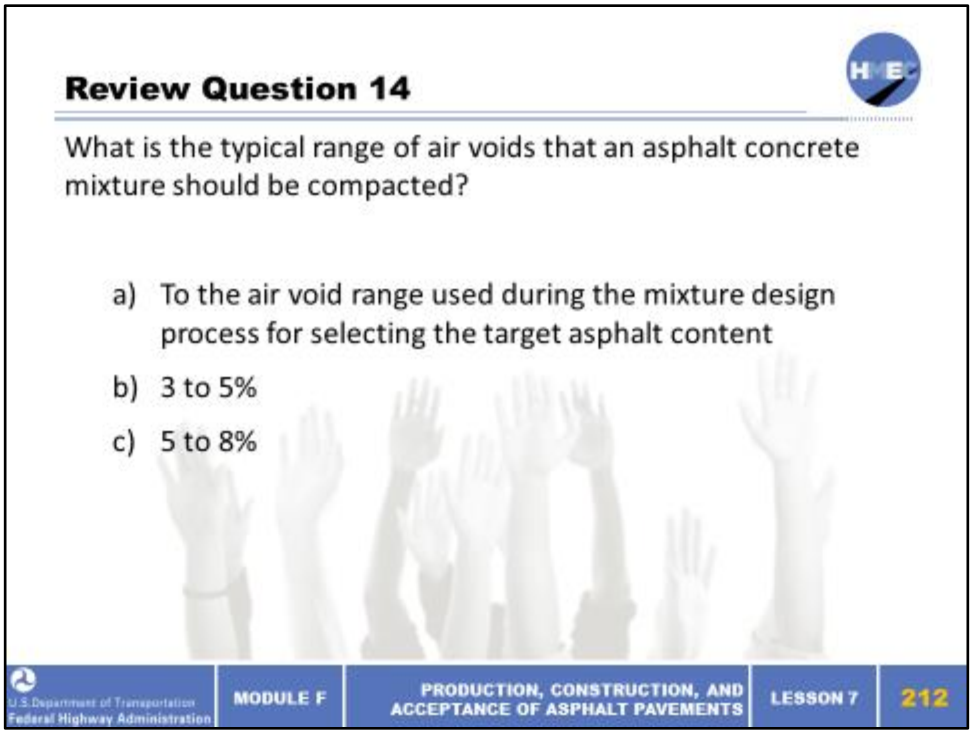
Slide 212


Review Question 14
What is the typical range of air voids that an asphalt concrete mixture should be compacted?
a) To the air void range used during the mixture design process for selecting the target asphalt content
b) 3 to 5%
c) 5 to 8%
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MODULE F
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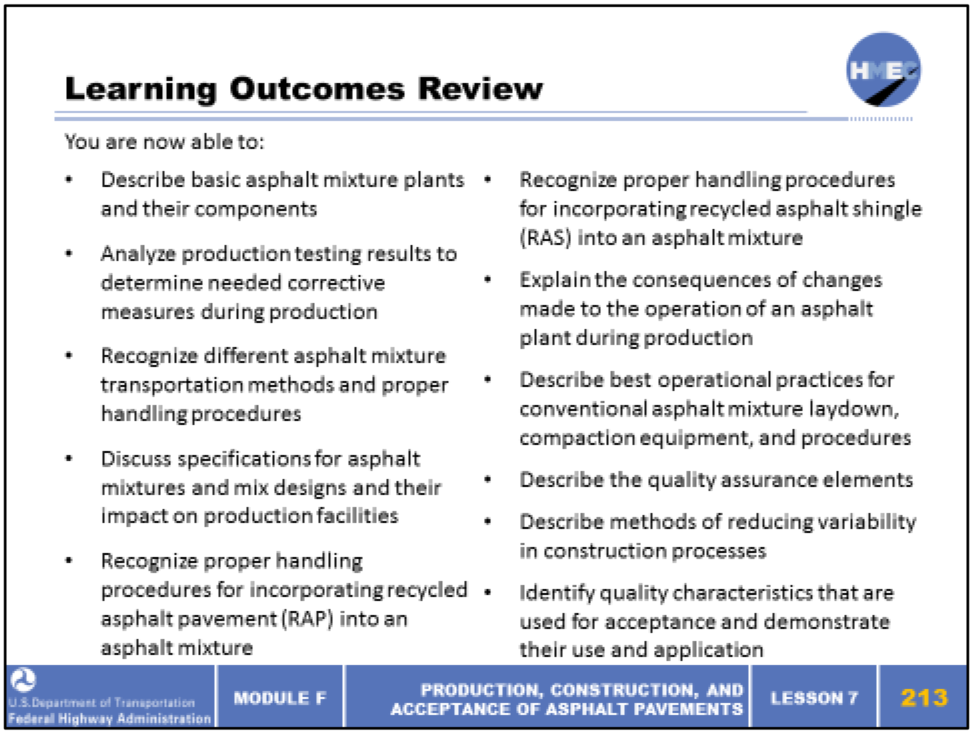
Slide 213


Learning Outcomes Review
You are now able to:
• Describe basic asphalt mixture plants and their components
• Analyze production testing results to determine needed corrective measures during production
• Recognize different asphalt mixture transportation methods and proper handling procedures
• Discuss specifications for asphalt mixtures and mix designs and their impact on production facilities
• Recognize proper handling procedures for incorporating recycled asphalt pavement (RAP) into an asphalt mixture
• Recognize proper handling procedures for incorporating recycled asphalt shingle (RAS) into an asphalt mixture
• Explain the consequences of changes made to the operation of an asphalt plant during production
• Describe best operational practices for conventional asphalt mixture laydown, compaction equipment, and procedures
• Describe the quality assurance elements
• Describe methods of reducing variability in construction processes
• Identify quality characteristics that are used for acceptance and demonstrate their use and application
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PRODUCTION, CONSTRUCTION, AND ACCEPTANCE OF ASPHALT PAVEMENTS
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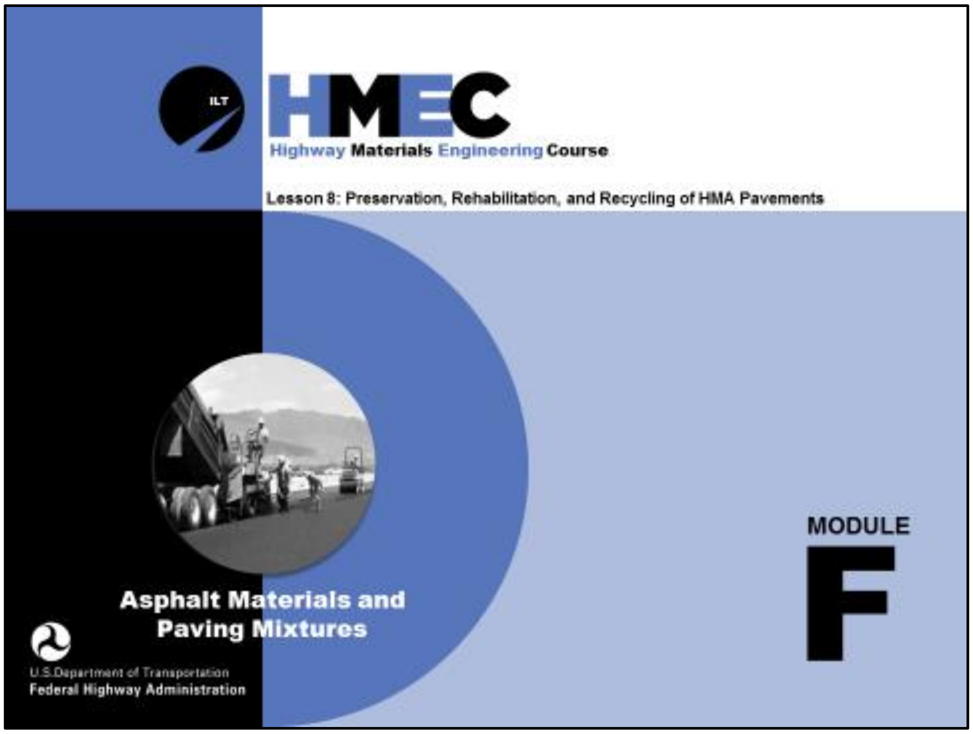
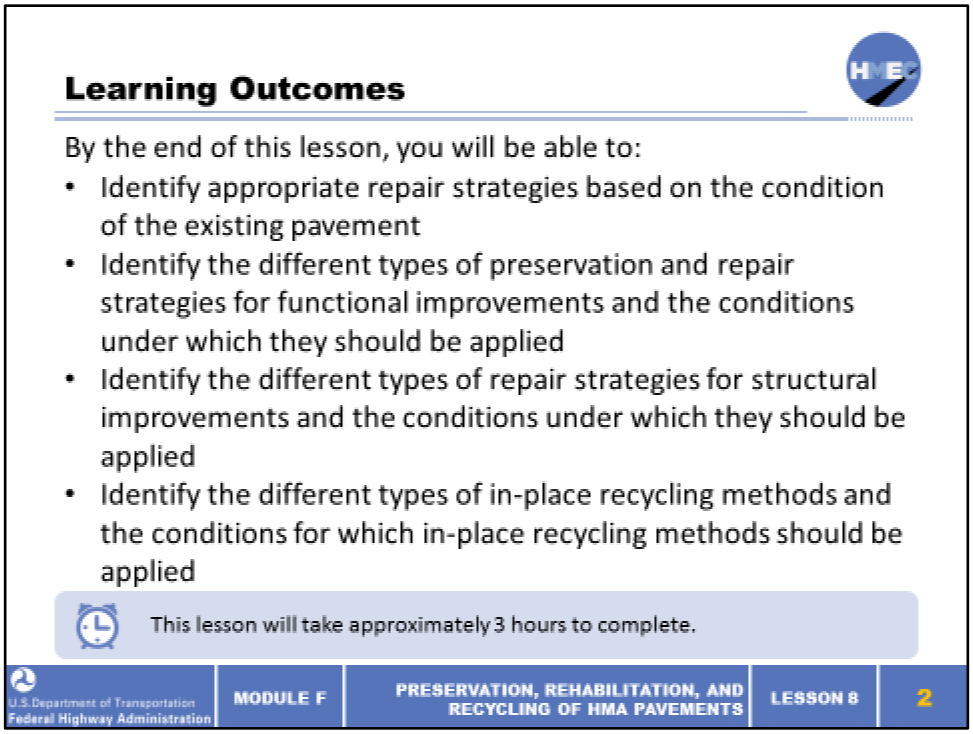
Slide 1


HMEC
Highway Materials Engineering Course
Lesson 8: Preservation, Rehabilitation, and Recycling of HMA Pavements
MODULE
F
Asphalt Materials and Paving Mixtures
U.S.Department of Transportation
Federal Highway Administration

Slide 2


Learning Outcomes
By the end of this lesson, you will be able to:
• Identify appropriate repair strategies based on the condition of the existing pavement
• Identify the different types of preservation and repair strategies for functional improvements and the conditions under which they should be applied
• Identify the different types of repair strategies for structural improvements and the conditions under which they should be applied
• Identify the different types of in-place recycling methods and the conditions for which in-place recycling methods should be applied
This lesson will take approximately 3 hours to complete.
U.S. Department of Transportation
Federal Highway Administration
MODULE F
PRESERVATION, REHABILITATION, AND RECYCLING OF HMA PAVEMENTS
LESSON 8
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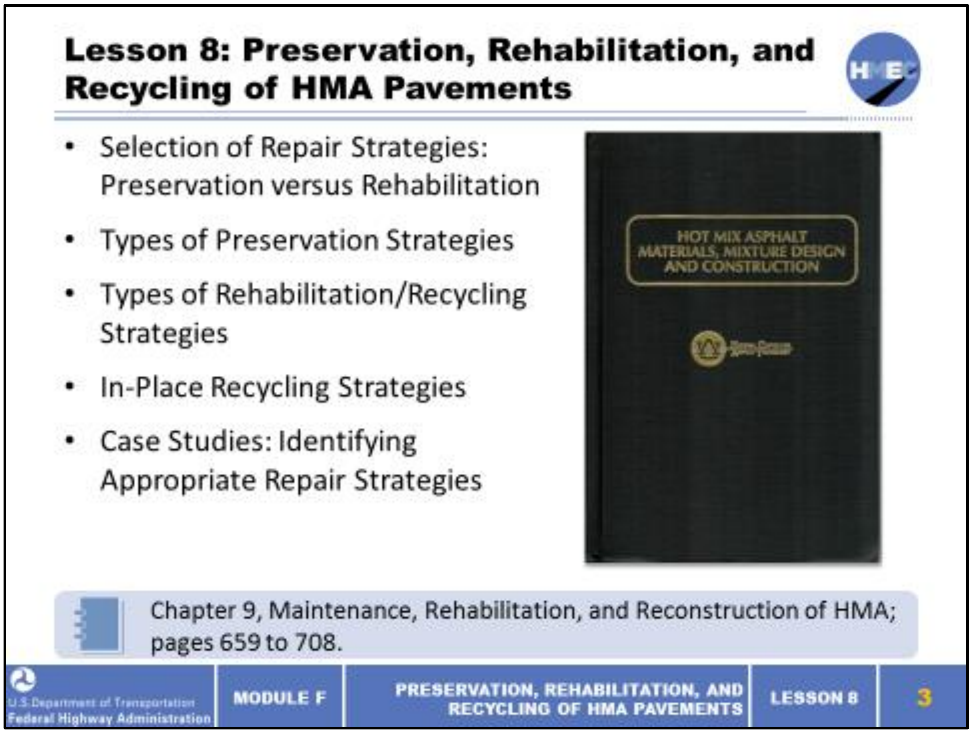
Slide 3


Lesson 8: Preservation, Rehabilitation, and Recycling of HMA Pavements
Selection of Repair Strategies: Preservation versus Rehabilitation
Types of Preservation Strategies
Types of Rehabilitation/Recycling Strategies
In-Place Recycling Strategies
Case Studies: Identifying Appropriate Repair Strategies
HOT MIX ASPHALT MATERIALS, MIXTURE DESIGN AND CONSTRUCTION
Chapter 9, Maintenance, Rehabilitation, and Reconstruction of HMA; pages 659 to 708.
U.S. Department of Transportation
Federal Highway Administration
MODULE F
PRESERVATION, REHABILITATION, AND RECYCLING OF HMA PAVEMENTS
LESSON 8
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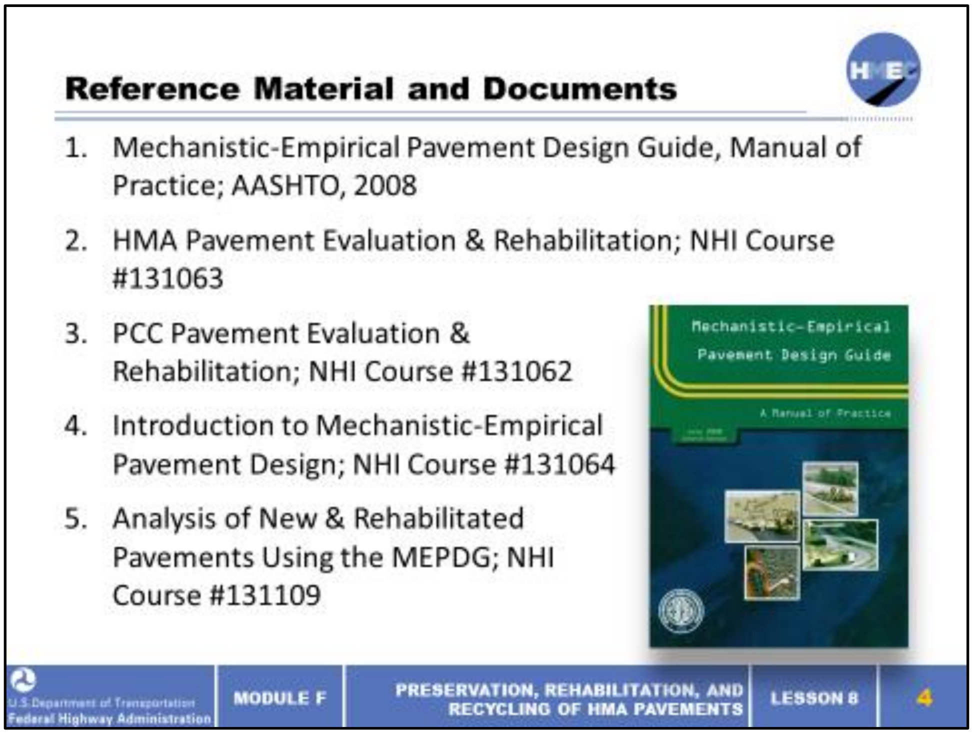
Slide 4


Reference Material and Documents
1. Mechanistic-Empirical Pavement Design Guide, Manual of Practice; AASHTO, 2008
2. HMA Pavement Evaluation & Rehabilitation; NHI Course #131063
3. PCC Pavement Evaluation & Rehabilitation; NHI Course #131062
4. Introduction to Mechanistic-Empirical Pavement Design; NHI Course #131064
5. Analysis of New & Rehabilitated Pavements Using the MEPDG; NHI Course #131109
Mechanistic-Empirical Pavement Design Guide
A Manual of Practice
U.S. Department of Transportation Federal Highway Administration
MODULE F
PRESERVATION, REHABILITATION, AND RECYCLING OF HMA PAVEMENTS
LESSON 8
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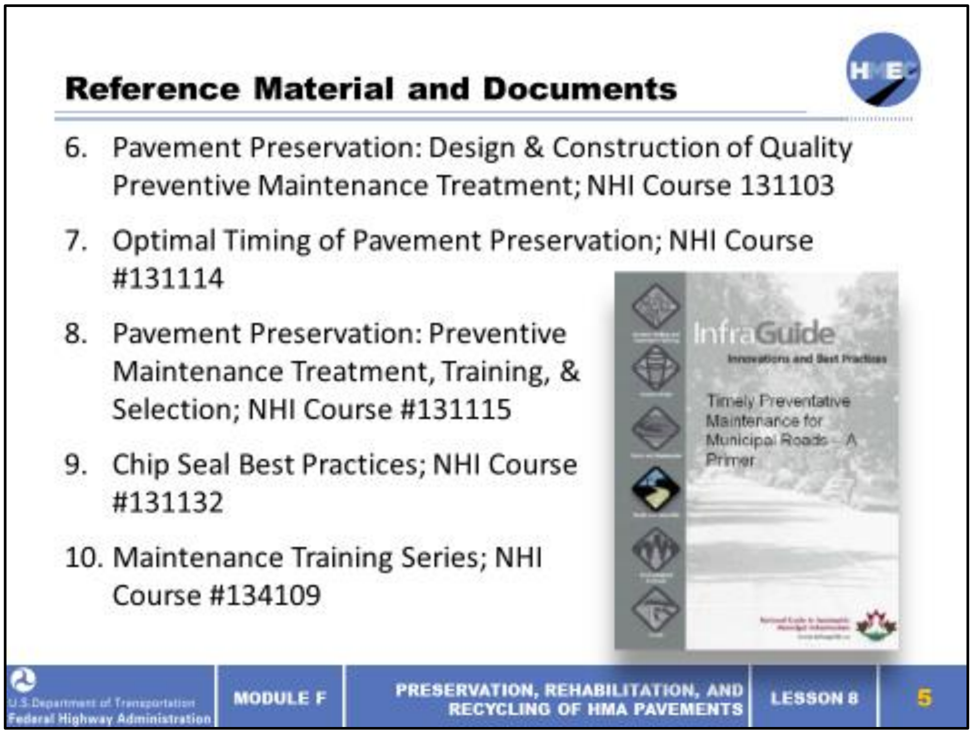
Slide 5



All of the documents and courses identified in this screen are directly applicable to this course.

Slide 6

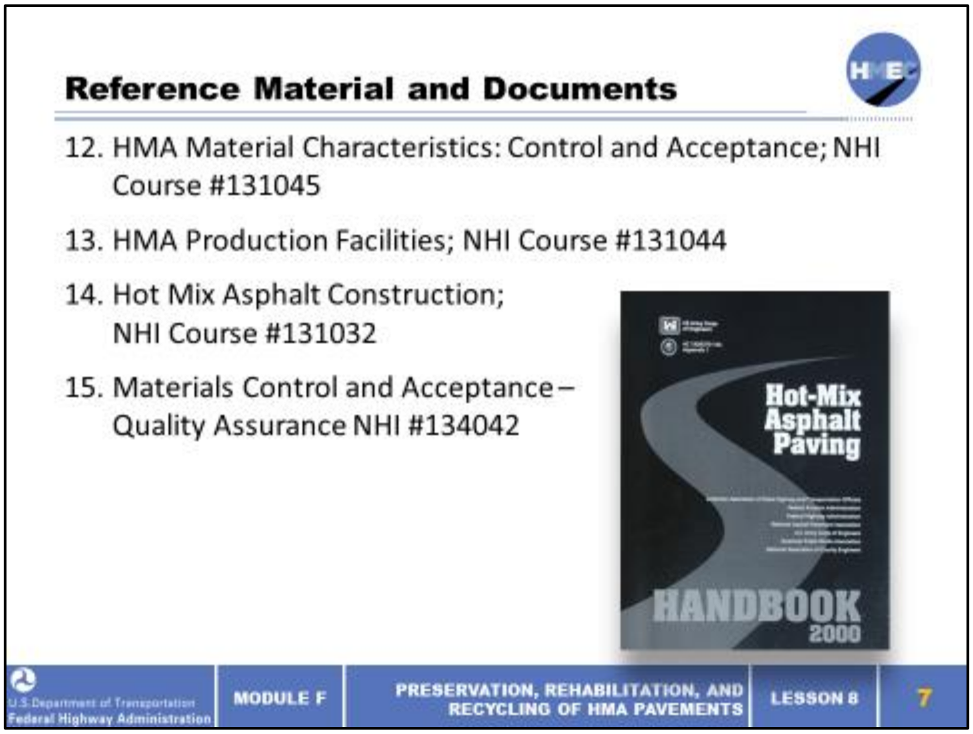
Reference Material and Documents

11. Pavement Preservation Treatment Series; NHI Course #131110

	Course Title	NHI #
a)	Introduction to Pavement Preservation	131110A
b)	Materials	131110B
c)	Crack Sealing & Filling & Joint Sealing	131110C
d)	Localized Pavement Repairs	131110D
e)	Chip Seals	131110E
f)	Fog Seals	131110F
g)	Slurry Seals	131110G
h)	Micro-Surfacing	131110H
i)	Thin Functional HMA Overlay	131110I
j)	Ultra Thin Bonded Wearing Surface	131110J
k)	Selecting the Right Treatment	131110K

U.S. Department of Transportation Federal Highway Administration | MODULE F | PRESERVATION, REHABILITATION, AND RECYCLING OF HMA PAVEMENTS | LESSON 8 | 6

Slide 7

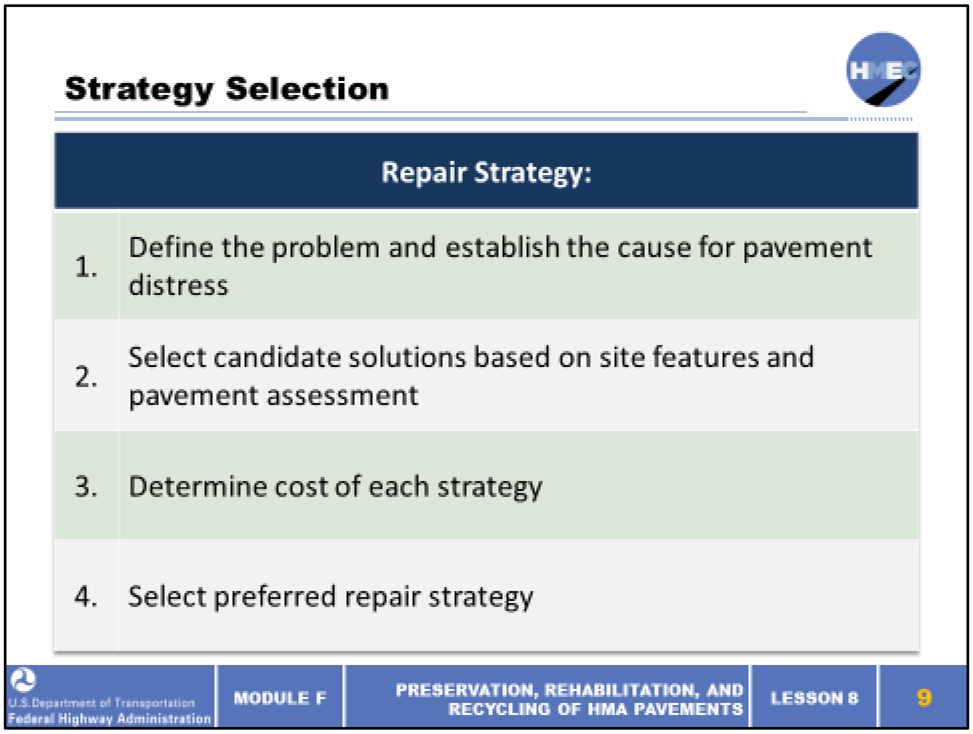


The Joint AASHTO/FHWA/Industry Training courses offered by the NHI, titled "Hot-Mix Asphalt Production Facilities" and "Hot-Mix Asphalt Construction," are comprehensive sources of training and reference materials concerning the topics discussed or included in this lesson. A revised version of the *Hot-Mix Asphalt Paving Handbook* was published by AASHTO in 2001. Numerous other publications were also used to develop this lesson.

Slide 8

The information covered within this lesson comes from many sources, which will be listed on the next set of screens, but was generally taken from Chapter 9, Maintenance, Rehabilitation, and Reconstruction of HMA of the NCAT textbook. Another major document for establishing the condition of the existing pavement is the 2008 AASHTO Mechanistic-Empirical Pavement Design Manual of Practice.

Slide 9



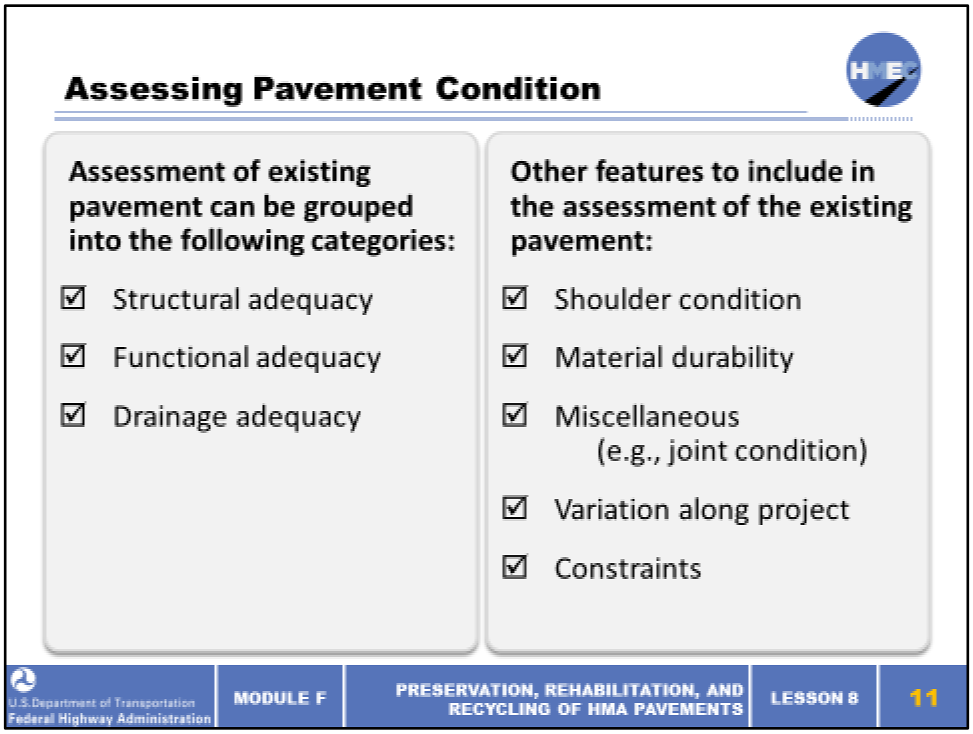
These four steps are included in the MEPDG Manual of Practice, which will be overviewed under Module E in terms of designing overlays and other rehabilitation strategies.

Slide 10

The use of pavement preservation shortly after construction to protect the wearing surface from early deterioration is more of an optimal timing issue which is not discussed within this lesson.

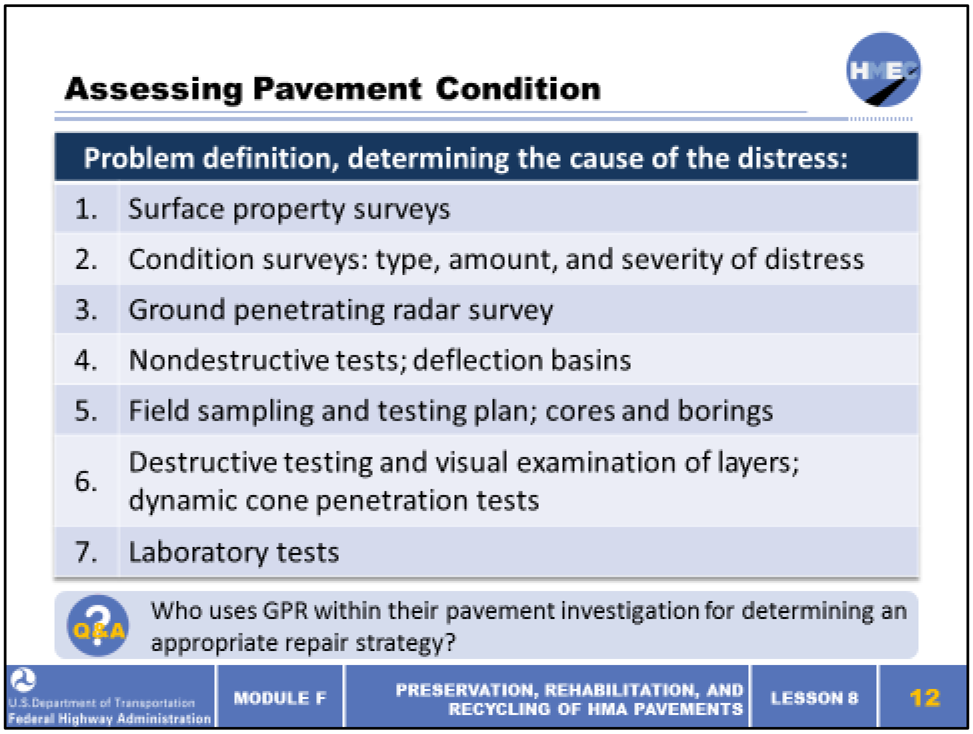
The 2008 AASHTO MEPDG Manual of Practice provides some guidance in the test equipment and activities required to characterize the in-place condition of the layers of the existing asphalt pavement. NHI course #131063 (HMA Pavement Evaluation and Rehabilitation) also provides detailed information and procedures that can be used to evaluate the condition of the existing pavement.

Slide 11



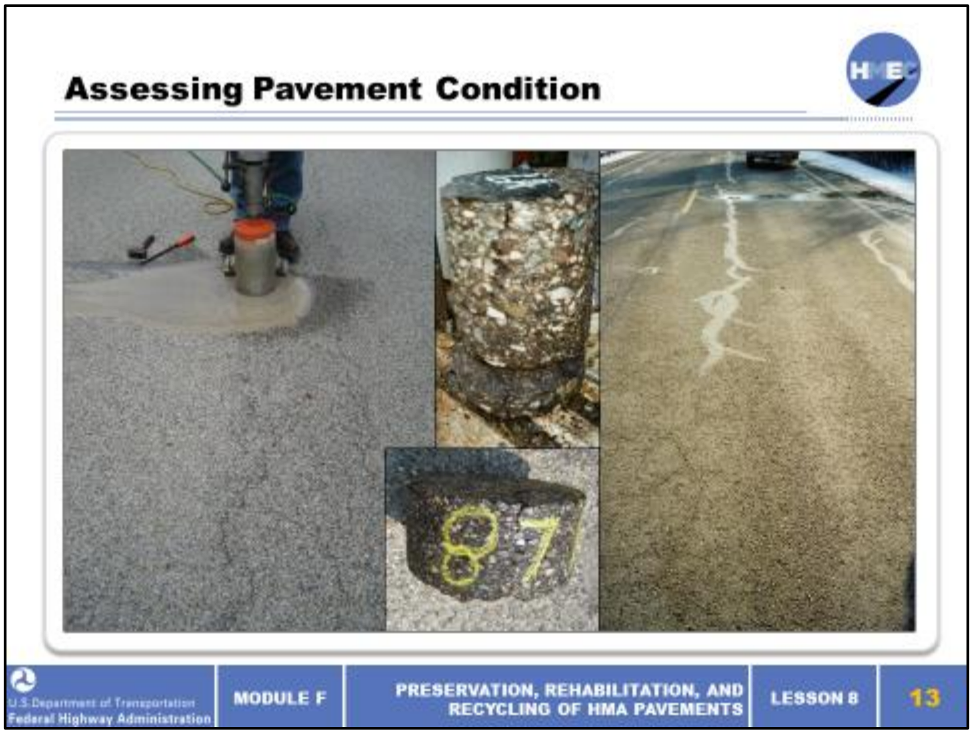
The steps to assess the pavement conditions for repairing the asphalt surface are included in the 2008 AASHTO MEPDG Manual of Practice. Module E provides additional discussion on these steps.

Slide 12



- Surface property surveys are used to determine the roughness and friction of the surface of the existing pavement.
- Condition surveys are used to define the type, amount, and severity of each distress.
- Ground penetrating radar (GPR) surveys are used to segment the subject roadway into different segments and identify subsurface features.
- Nondestructive deflection basin tests. It is expected that most of the agencies represented by the participants typically used deflection basins. These tests are used to back calculate the moduli of each pavement structural layer and to identify different pavement response design segments.
- Field sampling and testing consists of cores and borings. Cores are used to determine the layer thickness for back calculating layer moduli and identifying proper repair strategies. Cores are also used to visually inspect the different layers to identify any area or layer with stripping or moisture damage and if debonding has occurred between the bound pavement layers or lifts. Dynamic cone penetrometer (DCP) tests can be performed through the unbound layers after cores have been recovered. It is important to remove all water from the coring operation prior to conducting the DCP tests.
- Laboratory tests are used to measure the physical properties of the in-place layers.

Slide 13



Some within the industry use the GPR to determine the depth of cracking, but there is no consensus on the use of GPR to determine the depth of cracking. Another device that has been used to determine or evaluate the condition of the in-place asphalt concrete layers is the Portable Seismic Pavement Analyzer (PSPA). Both techniques have had successes as well as difficulty in determining the condition of the in-place layers and depths of cracks observed at the surface. The next screen shows the GPR and PSPA equipment.

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Slide 14

Module E provides more detail on how to interpret the results from the field investigation for deciding whether the existing asphalt pavement has adequate strength, needs additional structural support, or is inadequate for the truck traffic.

Slide 15

The 2008 AASHTO MEPDG Manual of Practice provides information and recommendations on developing an investigation program for determining the condition of the in-place asphalt pavement.

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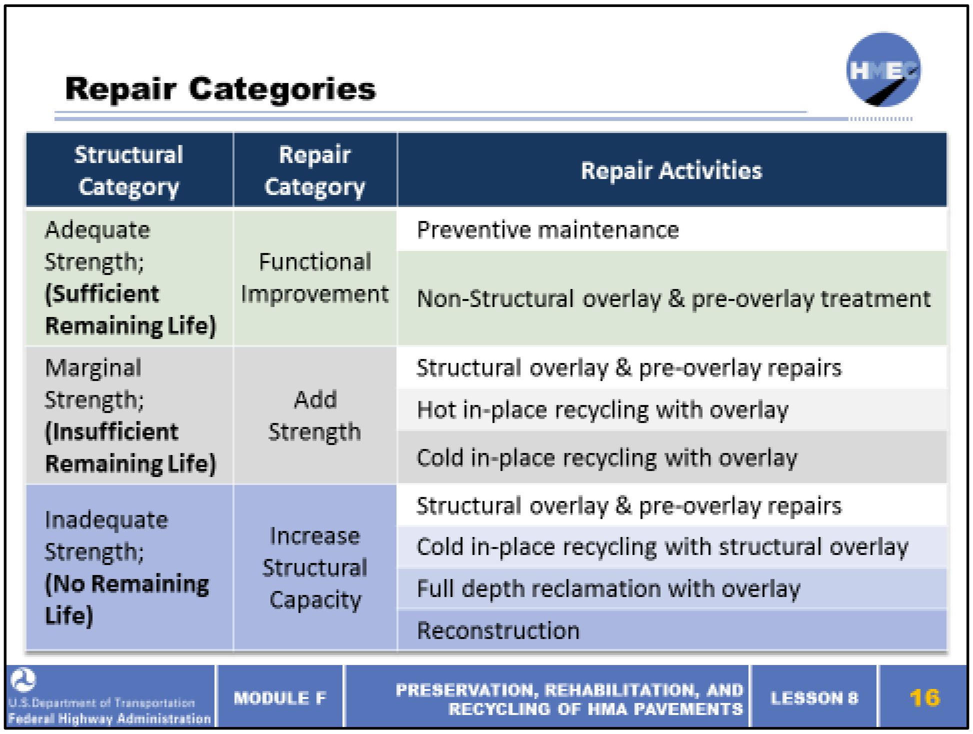
Slide 16



Structural Category	Repair Category	Repair Activities
Adequate Strength; **(Sufficient Remaining Life)**	Functional Improvement	Preventive maintenance
		Non-Structural overlay & pre-overlay treatment
Marginal Strength; **(Insufficient Remaining Life)**	Add Strength	Structural overlay & pre-overlay repairs
		Hot in-place recycling with overlay
		Cold in-place recycling with overlay
Inadequate Strength; **(No Remaining Life)**	Increase Structural Capacity	Structural overlay & pre-overlay repairs
		Cold in-place recycling with structural overlay
		Full depth reclamation with overlay
		Reconstruction

Some of the more common repair strategies for each group or category are identified for each repair strategy. Module E provides more detail on how to interpret the results from the field investigation for deciding whether the existing asphalt pavement has adequate strength, needs additional structural support, or is inadequate for the truck traffic.

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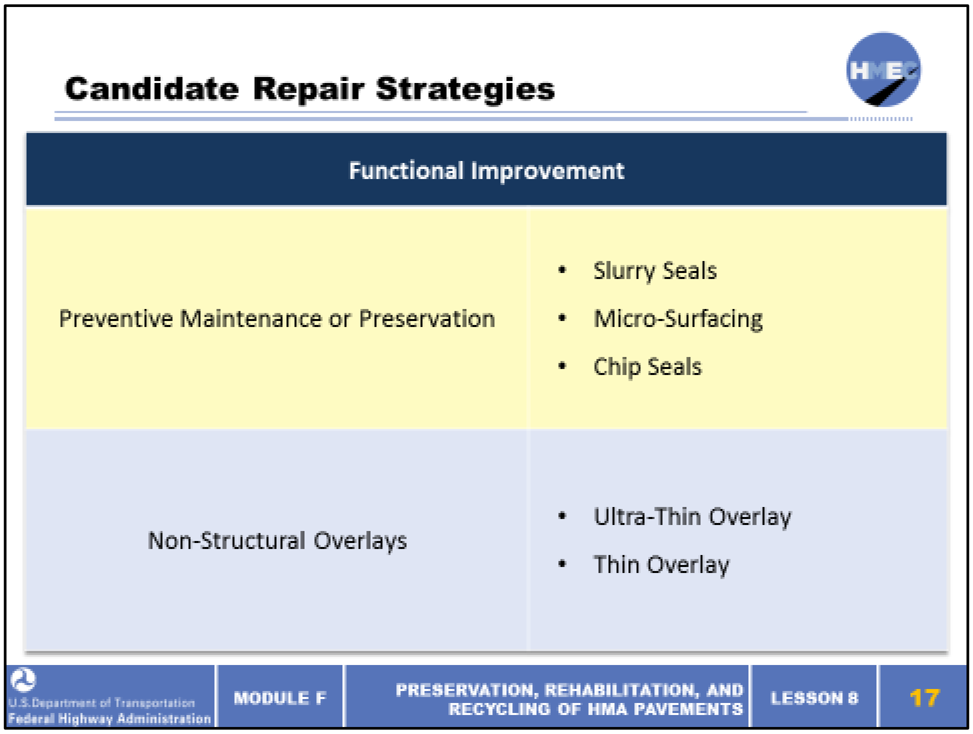
Slide 17


Candidate Repair Strategies
HMEC
Functional Improvement
Preventive Maintenance or Preservation
Slurry Seals
Micro-Surfacing
Chip Seals
Non-Structural Overlays
Ultra-Thin Overlay
Thin Overlay
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Candidate Repair Strategies

Structural Improvement		
Existing Surface Modification		• Mill & Remove Layers • Hot In-Place Recycling • Cold In-Place Recycling • Full Depth Reclamation
Structural Layer or Mixture	Overlay	• Thick Overlay
	Mix Modification	• Stone Mastic Asphalt • Polymer Modified Asphalt
	Interlayers or Reinforcement	• Stress/Strain Absorbing Membrane • Geo-grid

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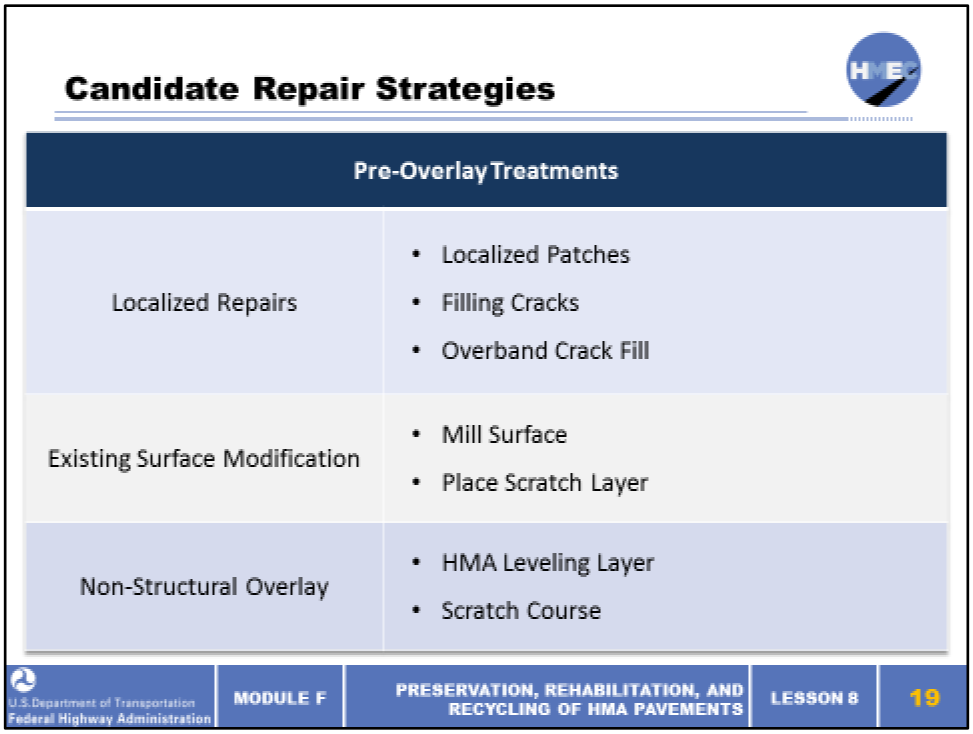
Candidate Repair Strategies
HMEC
Pre-Overlay Treatments
Localized Repairs
Localized Patches
Filling Cracks
Overband Crack Fill
Existing Surface Modification
Mill Surface
Place Scratch Layer
Non-Structural Overlay
HMA Leveling Layer
Scratch Course
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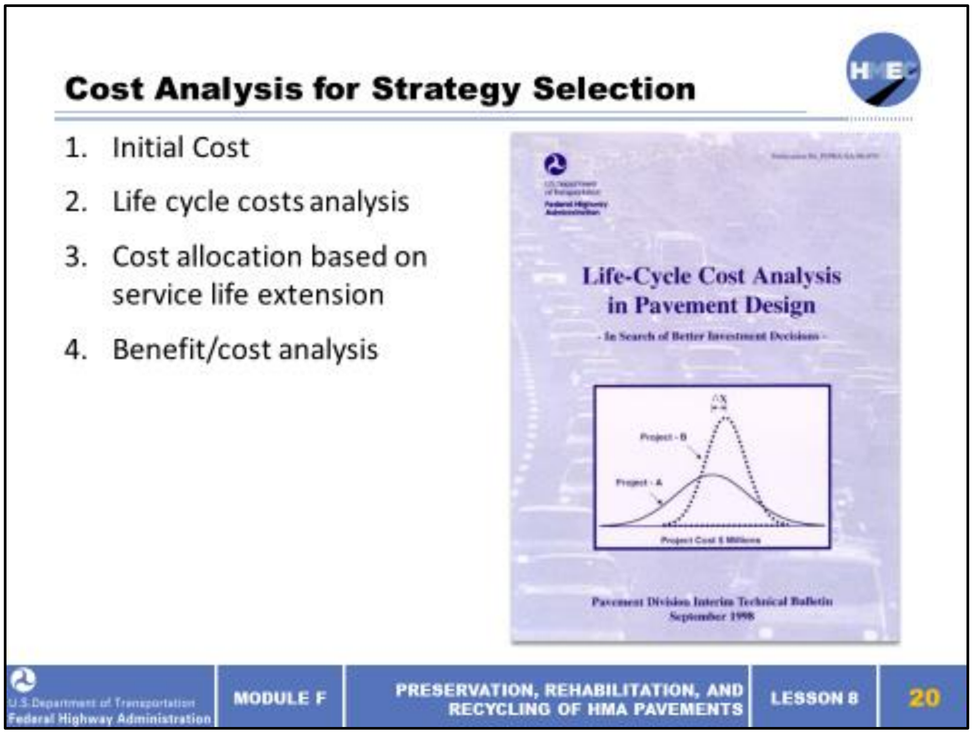


Four different techniques, which are listed in the screen and defined in the background information, can be used:

1. Initial cost of the strategy, which is probably the more common approach.
2. Life cycle cost analysis, which is defined in the report included in the screen.
3. Cost allocation based on the service life extension.
4. Benefit/cost analysis, which is the least common approach used.

The following defines and explains the different cost analysis techniques.

Initial Cost Approach: Selection of pavement rehabilitation strategy based solely on initial costs. Only considering initial construction costs will not identify or determine the optimal solution to minimize long-term costs for managing an agency's roadway network. Many agencies, however, use initial costs in their decision-making process. Initial cost by itself is not recommended for use within this guide and usually results in higher long-term costs, which are not sustainable as a general policy.

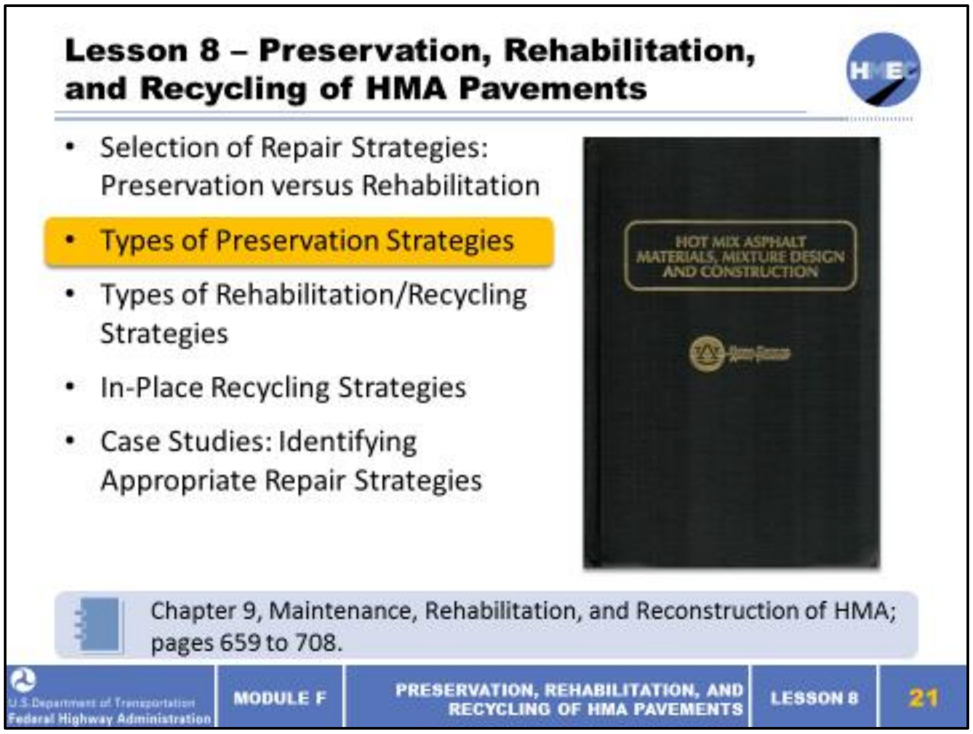
Life Cycle Costs Analysis Approach: The FHWA manual on Life Cycle Cost Analyses is used by many agencies in comparing and selecting new and rehabilitation alternate designs (FHWA-SA-98-079, Life-Cycle Cost Analysis in Pavement Design – Interim Technical Bulletin, 1998). The design strategy that minimizes life cycle cost is usually selected unless there are site conditions or non-monetary factors that override cost consideration. The LCC analysis approach is considered an appropriate method for evaluating and comparing different rehabilitation strategy options, but requires many input parameters—many of which may be unavailable and/or highly controversial.

Cost Allocation Based on Service Life Extension: This is a different approach for allocating costs and selecting pavement rehabilitation strategies based on service life extension ($/lane-mile-year). This approach is explained in the FHWA's technical brief entitled "A Quick Check of Your Highway Network Health," dated 2007 (Report #FHWA-IF-07-006).

Benefit/Cost Analysis: This approach considers both benefits to the user/agency and costs associated with those benefits. The process entails comparing benefit/cost ratio over the analysis period for various rehabilitation strategies. The preferred option is the one with the greatest benefit-cost ratio. Detailed information on the performance characteristics of individual treatments (performance curves and not just performance lives) is needed for this analysis, which is generally considered to be the more complicated one.

Note that once the cost analysis is completed, the repair strategy with the lowest cost and best constructability factors is then selected, which is the last activity. The different types of repair strategies are covered in the next set of screens or next section.

Slide 21



The information covered within this lesson comes from many sources, which will be listed on the next set of screens, but was generally taken from Chapter 9, Maintenance, Rehabilitation, and Reconstruction of HMA of the NCAT textbook. Another major document for establishing the condition of the existing pavement is the 2008 AASHTO Mechanistic-Empirical Pavement Design Manual of Practice.

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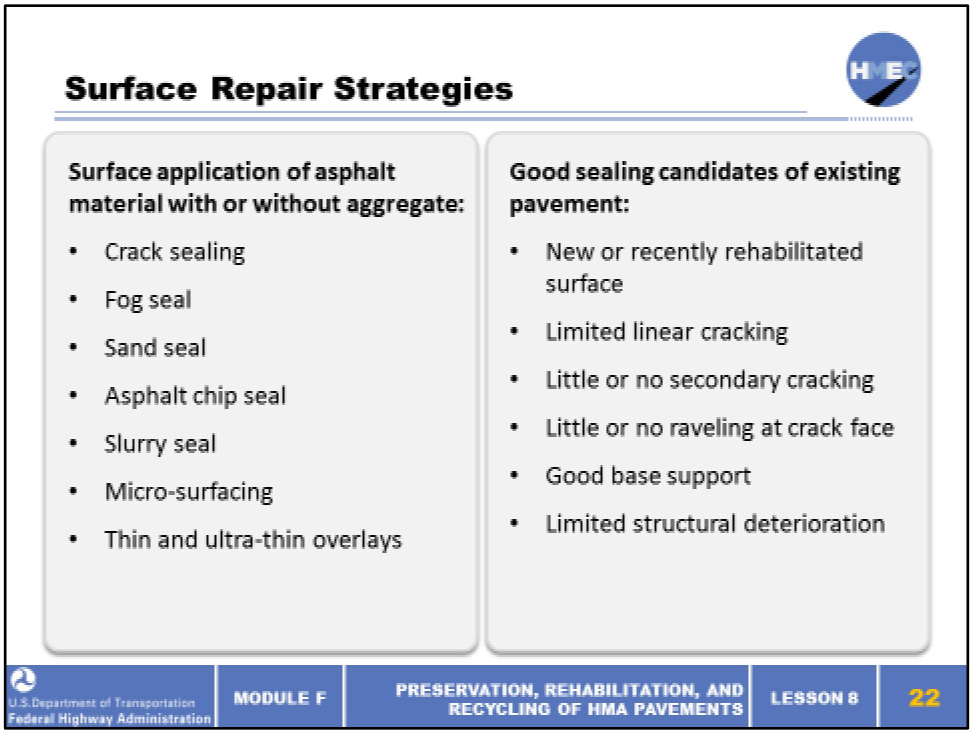
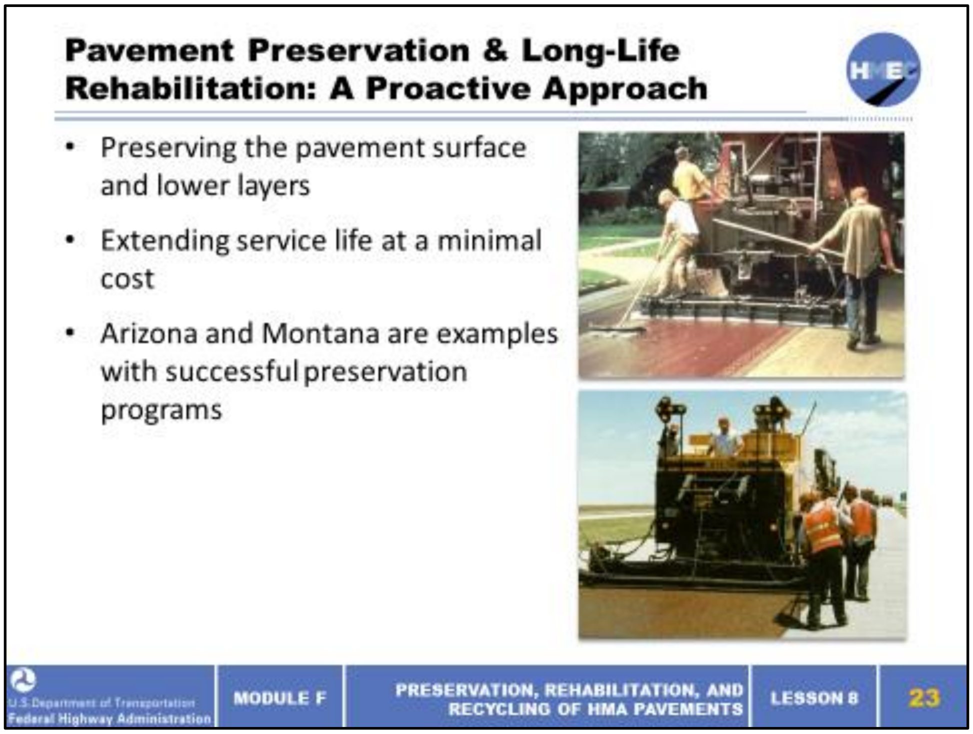


These surface strategies are those that are also more commonly used for pavement preservation early in the life of the asphalt pavement. Cracking sealing was included in the surface repair strategies because it is identified as a pavement preservation activity in the FHWA pavement preservation set of tools. However, it is usually applied as a part of other repair strategies.

Such mixtures are commonly in the range of 3/8- to 1-in. thick. Their purpose is to seal the pavement and to improve or protect the surface characteristics of the roadway, but they generally do not provide any structural enhancement from increased thickness. The types of surface treatments are listed on the screen, along with the conditions that should exist for a pavement to be a good crack sealing candidate.

Slide 23



The use of pavement preservation techniques or treatments applied to the pavement surface early in its life with little to no distress to extend the life of the pavement in comparison to the use of these same strategies except when pavement distresses start to occur.

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The purpose for crack sealing is to minimize the intrusion of moisture and incompressible materials. As a surface preparation technique, an additional purpose for crack sealing is to help reduce the rate of reflection cracking in an HMA overlay. However, the reduction in reflection cracking is minimal at best.

Sealant materials for HMA pavements typically fall under the thermoplastic category, i.e., they soften upon heating and harden upon cooling. These include asphalt cement, asphalt rubber, rubber asphalt, fiberized asphalt, and PVC coal tar. The process of crack sealing involves up to four basic steps:

1. Remove old sealant material.
2. Route the crack to provide a reservoir and better sealing capacity. This is normally not done as a surface preparation operation prior to overlay.
3. Clean dust and old sealant debris from the crack. Typically, this is done through compressed air blasting, which also helps to dry any moisture. However, many States are now using a compressed hot-air lance to clean, dry, and heat up the HMA around the crack to provide for better adhesion.

4. Apply sealant material. As a surface preparation technique, crack sealant is usually applied using a squeegee (lower right photograph on the screen).

Crack sealing and crack filling do not refer to the same process. Crack sealing refers to a process (as described above) that is intended to address problems with working cracks, i.e., transverse cracks that tend to open and close with changes in pavement temperature. Crack filling, on the other hand, refers to a less intensive process (employing lesser quality sealant materials) to address longitudinal and other cracks that do not open and close very much with changes in pavement temperature.

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Fog seals, sand seals, and scrub seals add asphalt to an existing pavement surface to improve sealing or waterproofing, prevent further stone loss by holding aggregate in-place, provide improved friction properties (sand seals and scrub seals), and simply improve the surface appearance. Rejuvenators are combinations of asphalt binder and proprietary chemicals that penetrate and improve the properties of the existing asphalt near the surface.

When used under the right conditions, surface seals are very effective at providing a waterproof surface that slows the aging process and improves surface characteristics. The keys to a successful fog, sand, or scrub seal are the application rate of the emulsion, the placement timing, and the care given to quality construction.

These treatments have traditionally been used on low-volume roads in rural areas; however, because of improvements in design, materials, and construction quality (as well as the increased emphasis on preventive maintenance), they are now being used on higher-volume roads. The Europeans have had good success with these treatments, and they are now becoming more popular in the US.

The primary benefit obtained from these treatments, especially when applied at the proper time, is the extension in pavement service life by slowing down the aging process.

The main similarity among the treatments is that they all involve the spray application of an asphalt emulsion using a distributor truck. The differences are in the types of emulsion used, the method by which the emulsion is massaged into the surface, and the use of a fine aggregate (sand).

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This application serves the same function as a fog seal but provides better surface friction. However, the surface appearance of a sand seal does not provide the delineation that a fog seal does. A sand seal is typically 2 mm to 5 mm (0.08- to 0.2-in.) thick.

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Fog seals are defined as a light application of a diluted asphalt emulsion (typically 8:1 or 9:1 mixture) placed primarily to seal the pavement, prevent raveling, enrich hardened asphalt, or provide delineation with the shoulder. Fog seals most commonly consist of a slow-setting emulsion that takes time to "break." Because of the required time to break, the pavement must typically be closed for approximately 2 hours after placement. Therefore, fog seals may be inappropriate for pavements in high-traffic urban areas.

More importantly, fog seals have the potential to temporarily reduce surface friction if excess asphalt is inadvertently applied to the pavement. Therefore, they are not recommended on high-speed roadways.

Fog seals are most effective on pavements in relatively good condition (i.e., little or no cracking or raveling). Fog seals have been found to last an average of 1 to 2 years, and repeated applications are expected to provide improved effectiveness. However, no formal studies have been conducted to evaluate the effect of fog seals on prolonging pavement life.

Fog seals are primarily used in the southwest US (dry arid climates) on lower volume roadways.

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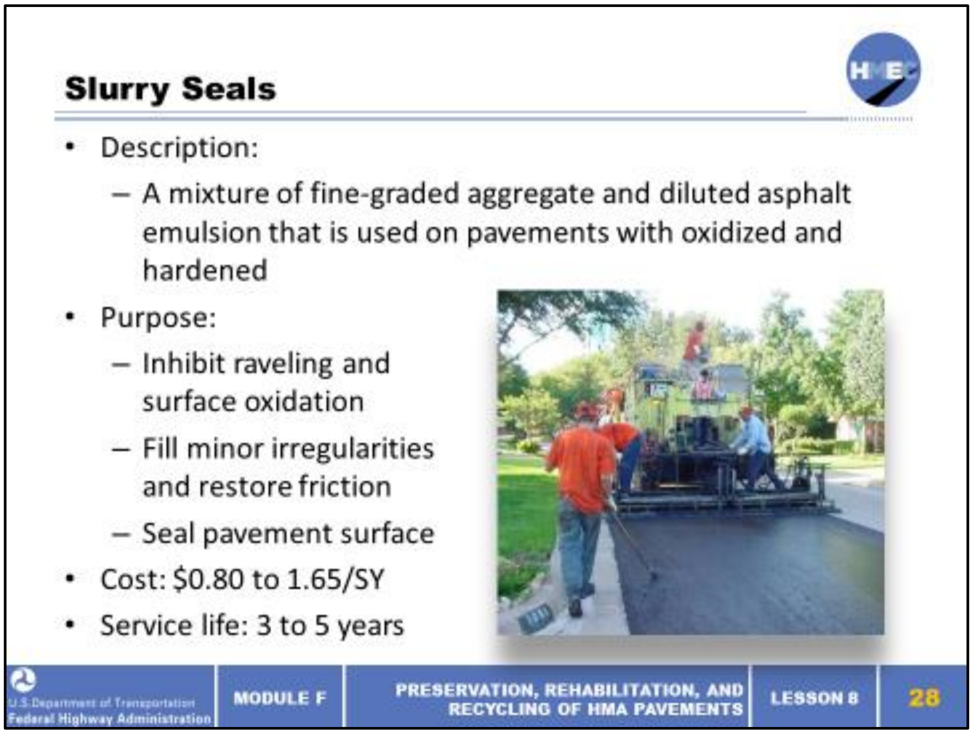

Slurry seals are appropriate for use when the primary deterioration is related to excessive oxidation and hardening of the existing asphalt. Aggregates must be clean, angular, durable, well-graded, and uniform (prefer 100% crushed). They generally require several hours to set before they can be opened to traffic, and are temperature and moisture sensitive. They are usually used on lower-volume roads (< 5,000 ADT) and placed during daylight hours.

Slurry seals are considered to have a nominal life of 3 to 5 years.

A slurry seal is a mixture of well-graded aggregate (fine sand and mineral filler) and diluted asphalt emulsion that is spread over the entire pavement surface with either a squeegee or spreader box attached to the back of a truck. (Geoffrey, D. N. 1996. “Cost-Effective Preventive Pavement Maintenance.” NCHRP Synthesis of Highway Practice 223. Transportation Research Board, Washington, DC)
There are three types of slurry seals as defined by the International Slurry Surfacing Association (ISSA), and these are differentiated based on traffic volume served and aggregate gradation (aggregate size):

• Type I is used to seal surface cracks on low-volume roadways (1/4-in. aggregate).

- Type II (the most common) is used to correct raveling and oxidation on roadways with moderate to heavy traffic levels (5/16-in. aggregate).
- Type III is used to fill minor surface irregularities and restore surface friction (3/8-in. aggregate).

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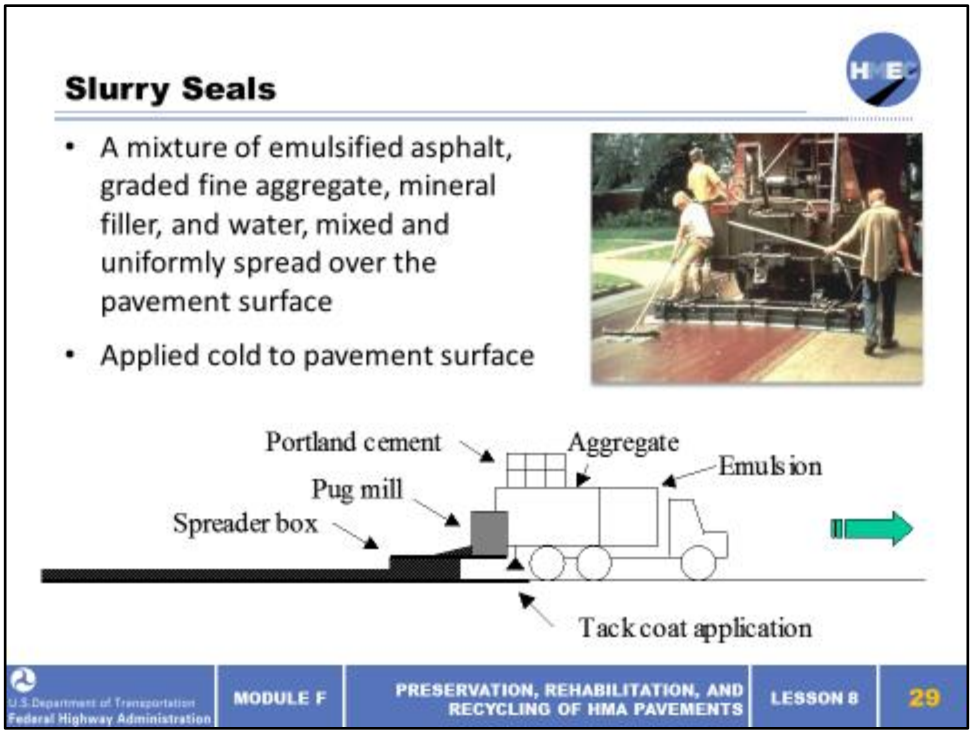

It is effective in sealing surface cracks, waterproofing the pavement surface, and improving skid resistance at speeds below about 64 km/h (40 mph). Different types of slurry seal are used and differ by the size of aggregate used. The thickness of the slurry seal is generally less than 9.5 mm (0.38 in.).

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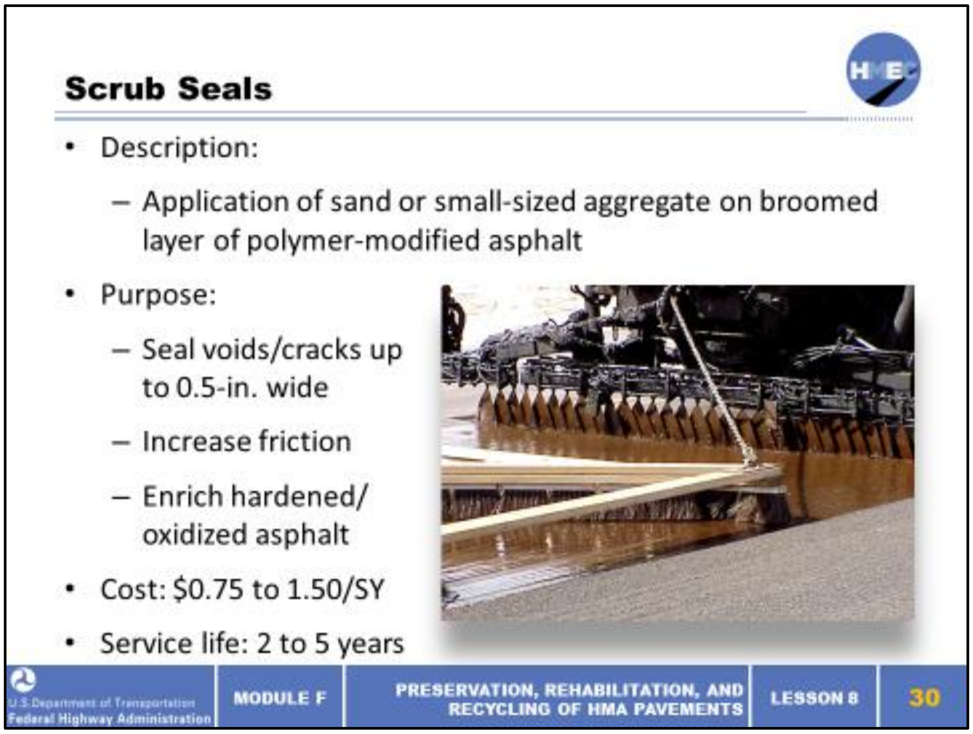

Scrub seals consist of an asphalt emulsion sprayed on the surface that is broomed, sanded, broomed again, and rolled. The brushes used are not exactly standardized. The emulsions used in the scrub seals are typically polymer-modified emulsions.

Scrub seals are a little more applicable to increased volume roadways than sand seals. The cost of scrub seals are about four to five times that of a fog seal. With the greater film thicknesses, however, significantly improved performance can be realized.

Basic definition of a scrub seal—asphalt emulsion sprayed on the surface, broomed, lightly sanded, and broomed again and rolled. Pioneered in the 1950s by Arizona and California, this treatment requires a little more attention in terms of design and construction detail than fog or sand seals.

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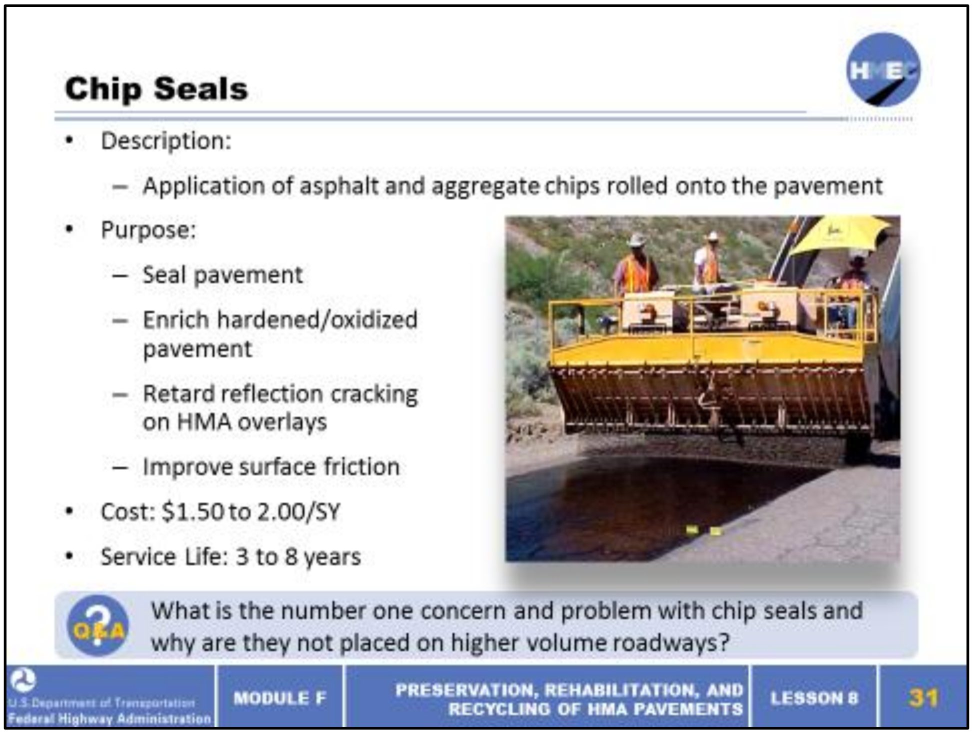

Chip seal or seal coats all involve the spray application of an asphalt emulsion. The differences in the types of seal coats result from the type of emulsion, the method by which the emulsion is worked into the surface, and whether an aggregate is broadcast over the emulsion.

A chip seal (also referred to by other names, including surface treatments, bituminous surface treatments, surface dressings, and seal coats) is defined as an application of asphalt (commonly an asphalt emulsion) directly on the pavement followed by an application of aggregate chips. The resulting treatment is then rolled to embed the chips in the binder.

There are also multiple applications of chip seals, in which two different-sized chips are placed in a built-up application. Chip seals have historically been used on low-volume roads (less than 2,000 ADT) as a wearing surface on untreated granular roadbeds; however, they have more recently been used on higher-volume roads (over 10,000 ADT) because of their ability to waterproof the surface, provide low-severity crack sealing, and improve surface friction. There is a complete family of treatments that falls into the “chip seal” category:

• Rubberized asphalt chip seals are commonly used in conjunction with HMA overlays to retard reflection cracking.

• Sand seals are used to enrich a dry, oxidized surface and to prevent the intrusion of moisture and air.
• Sandwich seals consist of large aggregate, spray of asphalt emulsion, and application of smaller aggregate. They are used to seal the surface and improve skid resistance.
• Cape seals are a chip seal covered with a slurry seal and are used to provide a dense waterproof surface with improved skid resistance.

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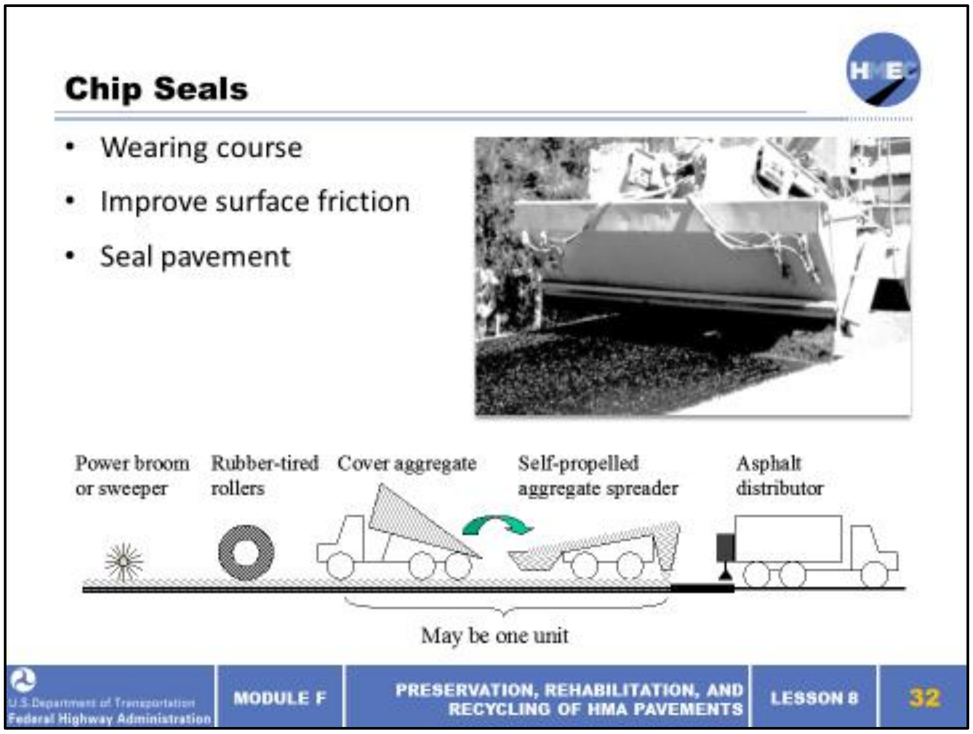

Existing pavements that exhibit the following distress are usually not good candidates for chip seals:

• Wide cracks experience large movements that will reflect through the chip seal.
• More severe alligator cracking and potholes indicate a localized structural problem that needs to be addressed by another technique.
• Deeper rutting or a very rough surface will not be corrected with a chip seal application.

Chip seals can make a pavement rougher, so a good candidate is one with a relatively smooth pavement surface. Rubber-modified chip seals have been used on more distressed pavements. As noted in the previous screen, chip seals and/or rubber-modified chip seals are typically used as a reflection cracking mitigation technique prior to placing a dense graded asphalt concrete overlay. This use of chip seals to mitigate reflection cracking is explained in the next section of this lesson.

The performance of chip seals is heavily dependent on the contractor's knowledge and experience in placing chip seals.

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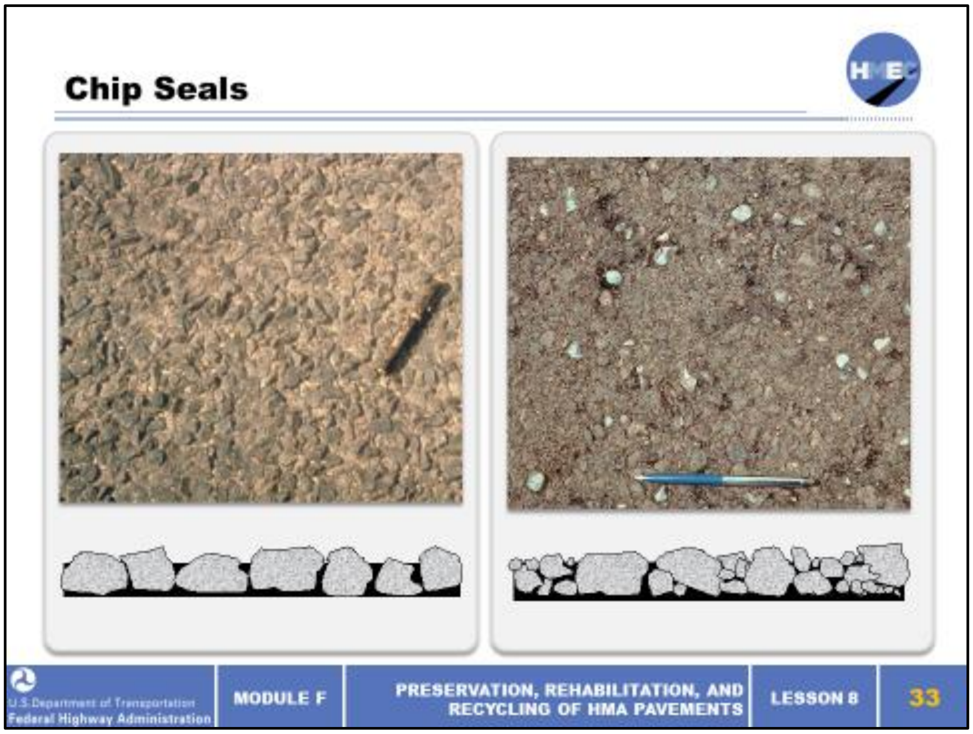

Note the difference in surface texture and/or chip retainage as shown in the screen.

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Chip Seal Variations

- Single chip seals
- Double or triple chip seals
- Cape seals
- Fabric and chip seals

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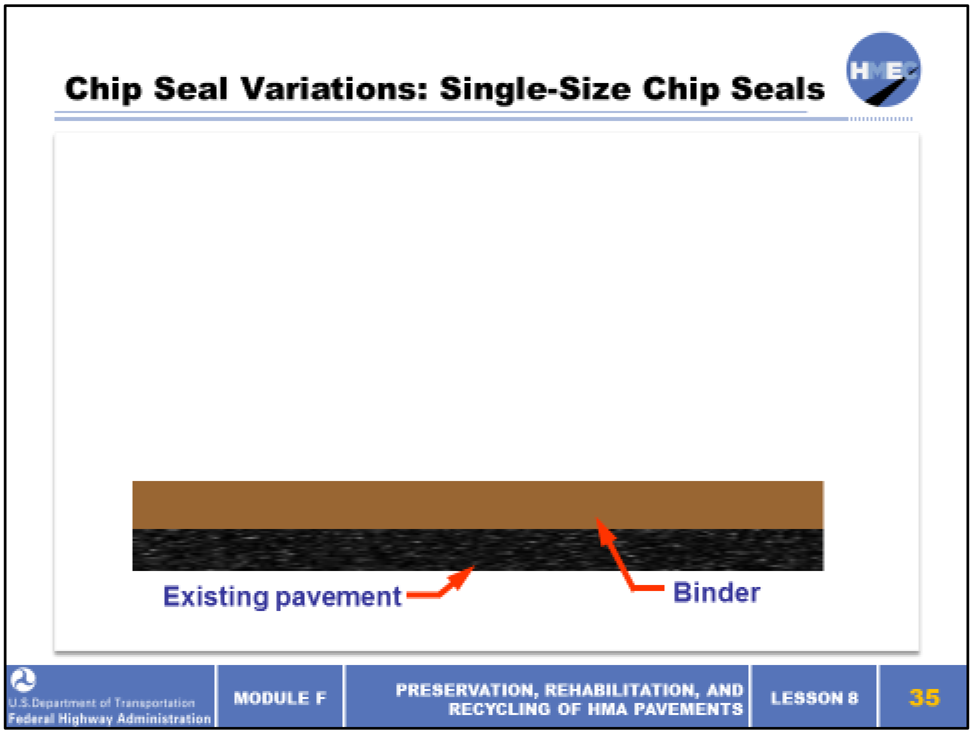

Single (conventional) chip seals consist of the application of an asphalt or emulsion directly on the existing pavement, followed by the application of an aggregate that is rolled with a pneumatic roller. Emphasize that the majority of this presentation focuses on single (conventional) chip seals.

Rubberized asphalt chip seals are similar to chip seals except that the asphalt binder is replaced with a blend of ground tire rubber (or latex rubber) and asphalt cement (Raza, H. 1992. An Overview of Surface Rehabilitation Techniques for Asphalt Pavements. FHWA-PD-92-008. Federal Highway Administration, Washington, DC). The rubber additive enhances the elasticity and adhesion characteristics of the binder. Rubberized asphalt chip seals are commonly used in conjunction with an overlay to retard reflection cracking.

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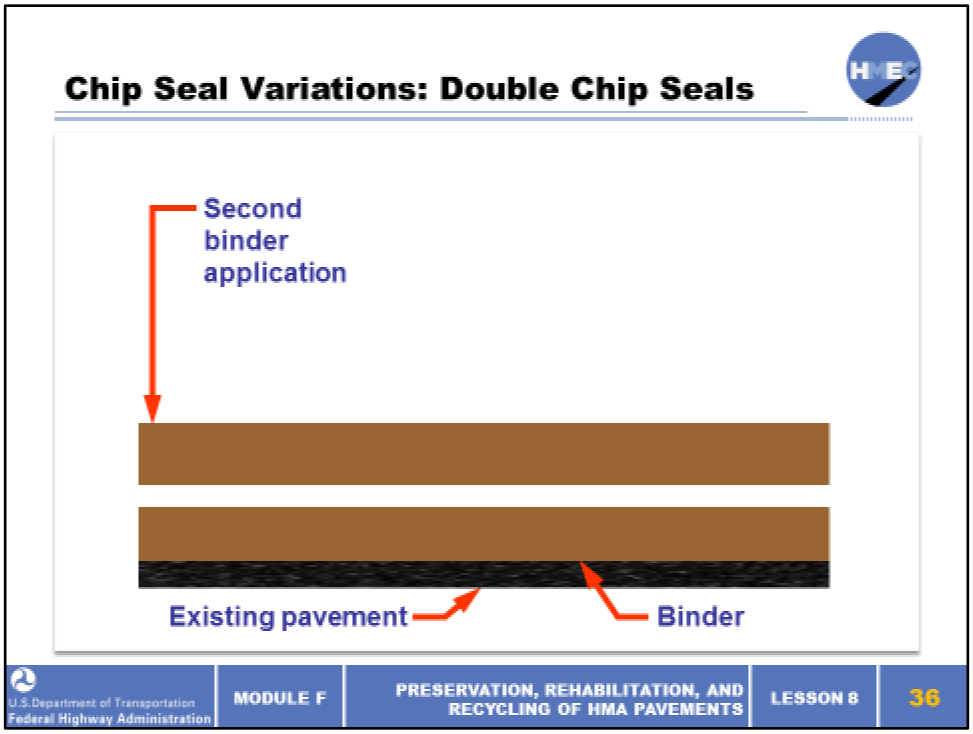

Double or triple chip seals are obtained through two or three applications of a chip seal operation over the same roadway, each subsequent layer being placed after the previous layer has cured. These multiple chip seals are dense-wearing, waterproofing applications that, in some instances, may approach thicknesses of 1 in. (Asphalt Institute. 1997. A Basic Asphalt Emulsion Manual. Manual Series No. 19, Third Edition. Asphalt Institute, Lexington, KY, and the Asphalt Emulsion Manufacturers Association, Annapolis, MD).

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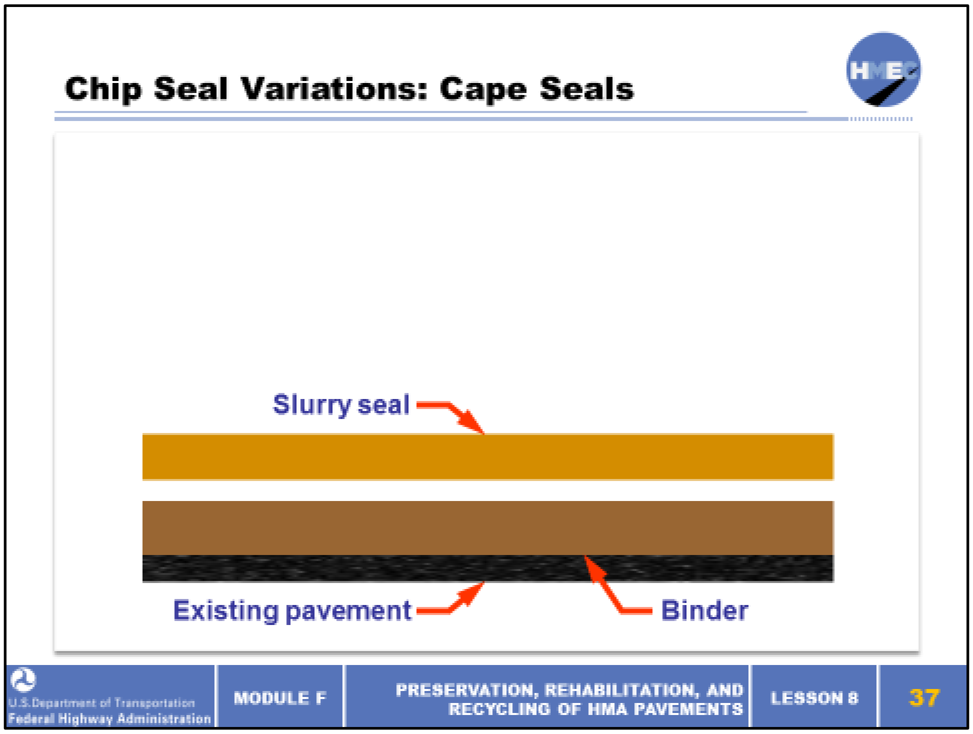

Cape seals are a chip seal covered with a slurry seal and are used to provide a dense, waterproof surface with improved skid resistance (Raza, H. 1992. An Overview of Surface Rehabilitation Techniques for Asphalt Pavements. FHWA-PD-92-008. Federal Highway Administration, Washington, DC). For cape seals, the application of the slurry seal also helps reduce stone loss.

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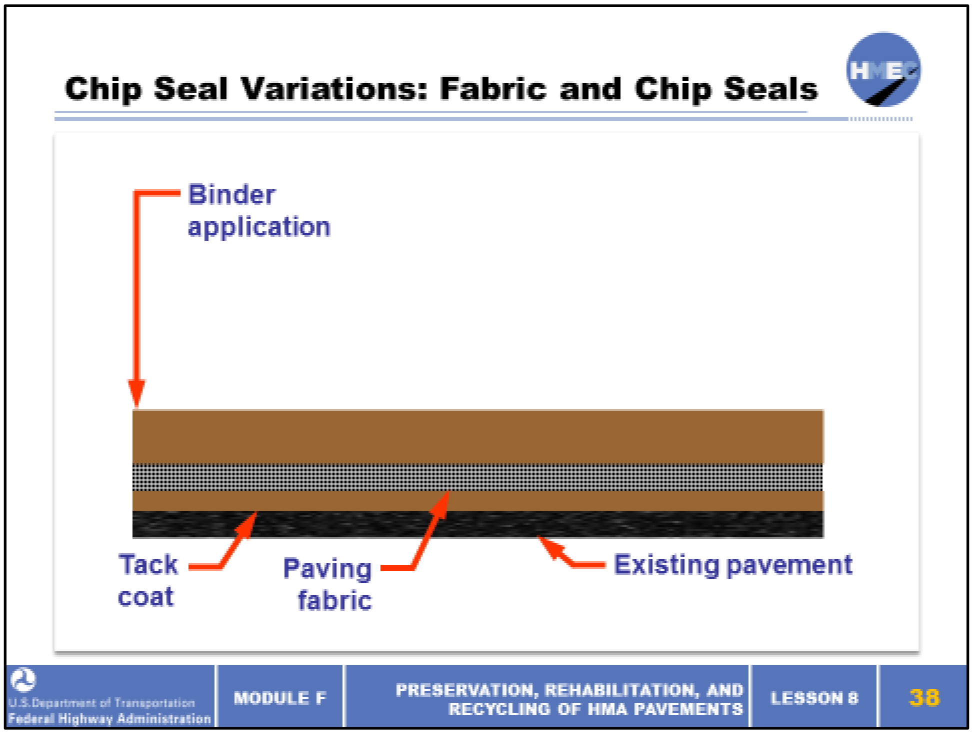

This screen is animated showing the different operations in placing a chip seal with a fabric that is typically used as a reflection cracking mitigation technique.

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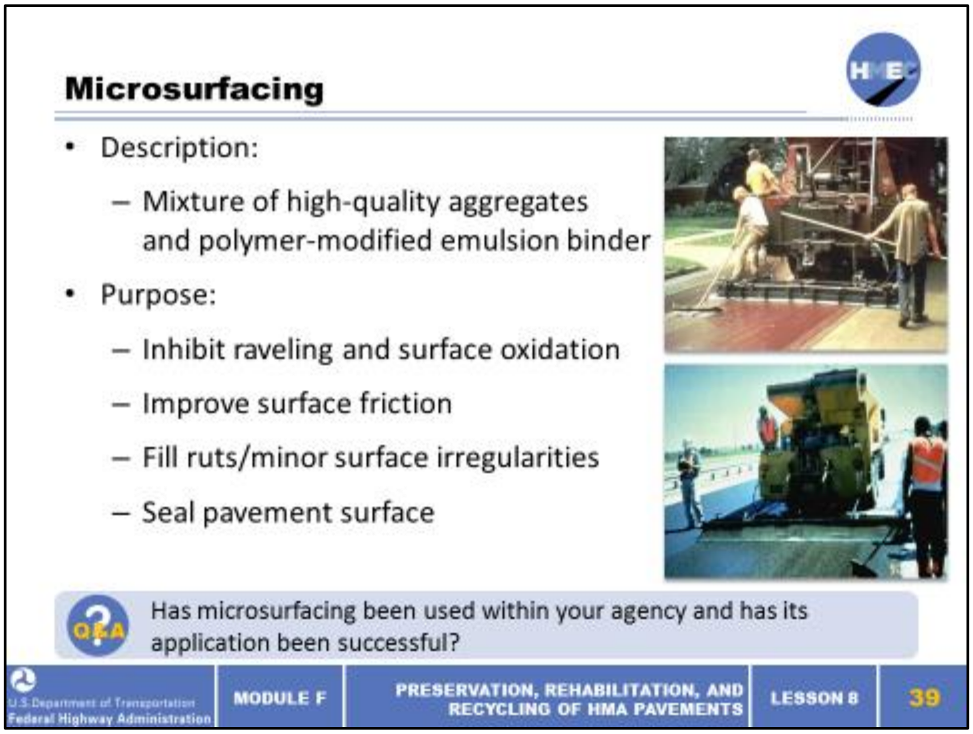

Developed in Europe, microsurfacing is a term used to describe the application of a polymer-modified slurry seal, with latex rubber being the most commonly used polymer. Microsurfacing materials consist of asphalt and latex mixed with aggregate, fillers, and other additives, and is a modification of the slurry and sand seal (Pederson 1986). It has been used as a wearing surface and for rut-filling. Ralumac is probably the most widely known example of this process in the United States (Asphalt Institute 1986).

Microsurfacing is primarily used to correct or inhibit raveling and oxidation of the pavement surface; however, it is also effective in improving surface friction, sealing the pavement surface, and filling minor surface irregularities and wheel ruts 1.25-in. deep in a single pass.

The use of a CSS-1hp binder is common. The “CSS” indicates a catatonic, slow-setting emulsion; the “1” is its relative viscosity (a “-2” is more viscous than a “1”), the “h” signifies that a harder grade of base asphalt was used in the production of the emulsion, and the “p” stands for polymer-modified.

Some other facts regarding the use of microsurfacing:

• Microsurfacing has successfully been used on both low- and high-volume roadways.
• Microsurfacing has been found to perform well for four to seven years and even longer, depending on the condition of the existing pavement.

There are two generally accepted aggregate gradations: Type II and Type III. The choice depends on the type of application.

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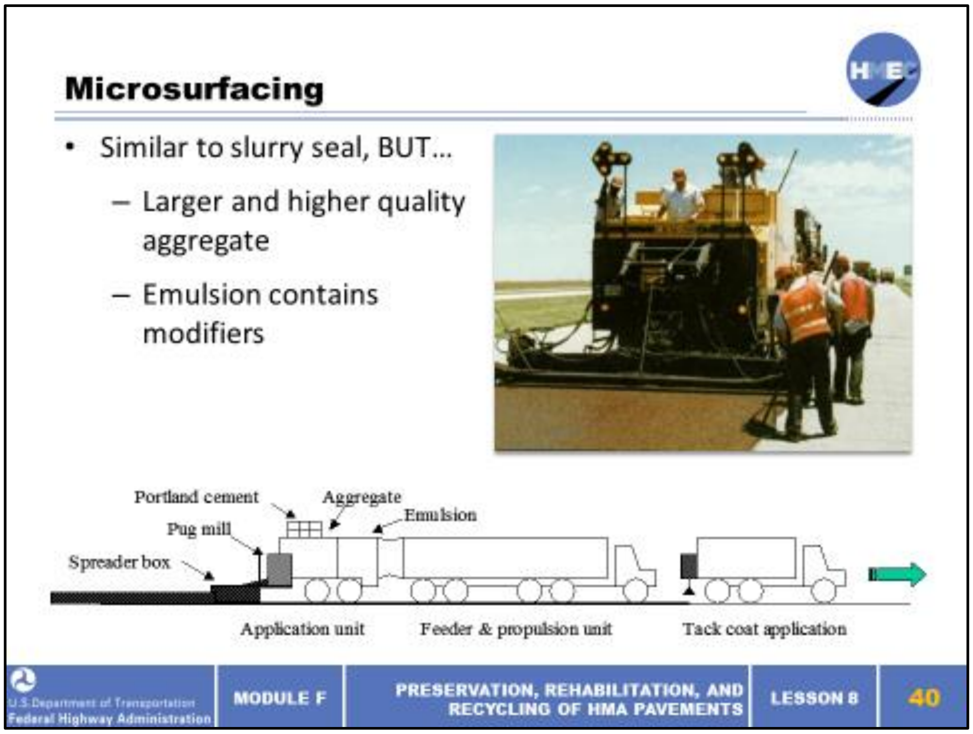

The schematic at the bottom of the screen shows the different equipment and materials used to place the microsurfacing. The instructor should review and explain the different equipment and placement operation.

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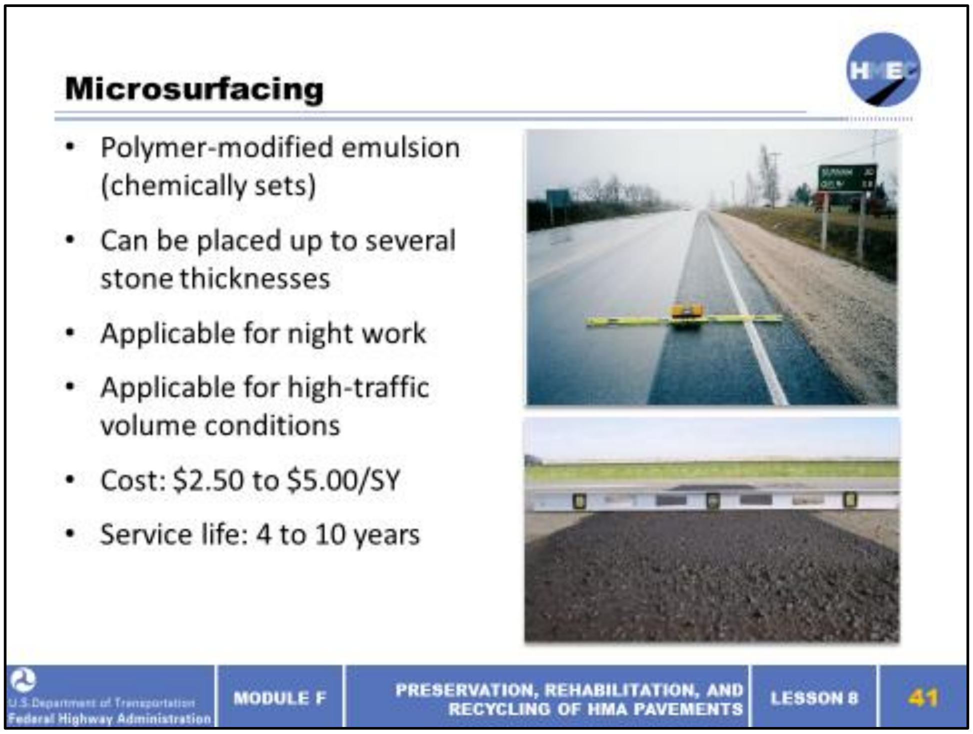

Microsurfacing is essentially a high-quality, engineered slurry seal. While microsurfacing is always polymer modified, slurry seals may or may not be polymer modified. Other differences in the two treatments are listed below:

- Microsurfacing has tighter controls on the aggregate quality and gradations.
- Microsurfacing chemically sets; slurry seals typically environmentally set.
- Microsurfacing is designed to be placed up to several stone thicknesses; slurry seals are designed to have a monolayer stone thickness.
- Microsurfacing can be applied at night.
- Microsurfacing may be used to fill ruts, as it can be placed in thicker layers.
- Microsurfacing also has quick setting properties that make it ideal for high traffic-volume conditions where long lane closures are not possible.
- Microsurfacing can also be placed in a broader range of temperature and weather conditions than conventional slurry seals, which means that it is better suited to night work and extended construction seasons.
- It is also possible to apply an additional microsurfacing on a microsurfaced pavement. More than one additional application is usually not recommended.

CSS-1h-p emulsions are the most widely used for microsurfacing.

A Type I gradation is commonly too fine for microsurfacing applications, although it is occasionally used on airfield projects. A Type II and Type III gradation are typically used in microsurface applications. A Type III gradation is coarser than the Type II gradation. Selection of the gradation depends on whether a single or double application of the microsurface will be placed, as well as the depth of any minor depressions or rut depths that need to be filled. The following shows the typical gradations:

Sieve Size	Type I	Type II	Type III
3/8	---	100	100
#4	100	94 to 100	70 to 90
#8	90 to 100	65 to 90	45 to 70
#16	60 to 90	45 to 70	28 to 50
#30	40 to 65	30 to 50	19 to 34
#50	---	18 to 30	7 to 18
#200	10 to 20	5 to 15	5 to 15

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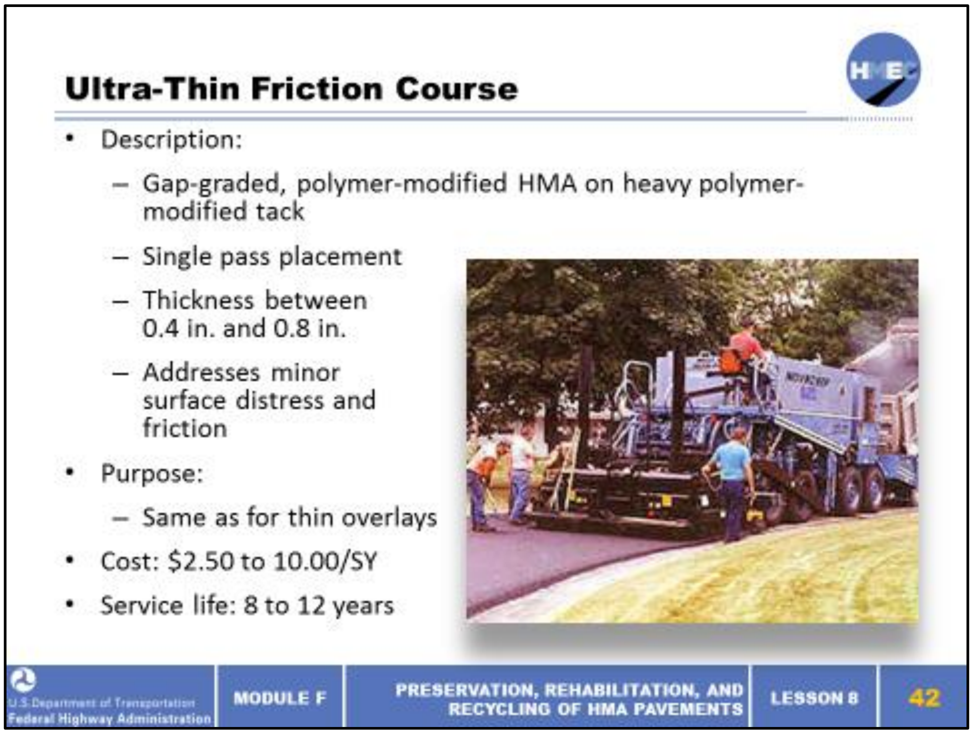

The effectiveness of an ultra-thin friction course is very similar to the effectiveness of a thin HMA overlay.

An ultra-thin friction course is an alternative to chip seals, microsurfacing, or thin HMA overlays as it effectively addresses minor surface distresses and increases surface friction. Some interesting facts with the use of ultra-thin friction courses are listed below:

• This technique is widely used in France and was introduced in the United States in the late 1990s.
• In the US, this product is marketed under the NOVACHIP name.
• The short-term performance of ultra-thin friction course projects has been promising. Texas and Pennsylvania are monitoring the performance of these installations, and they report excellent performance after three years.
• The installation cost for ultra-thin friction courses is as low as about $2.50 per square yard and may go much higher; it is at least 50% more than a thin, dense-graded HMA overlay.

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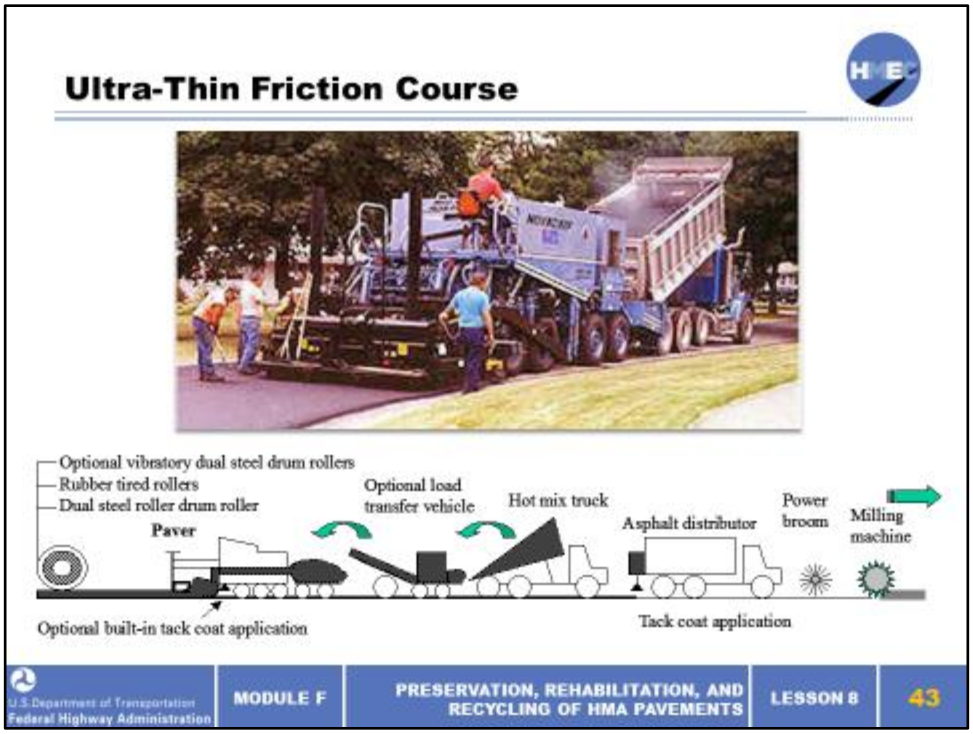

A schematic of the equipment and application process is shown in the screen.

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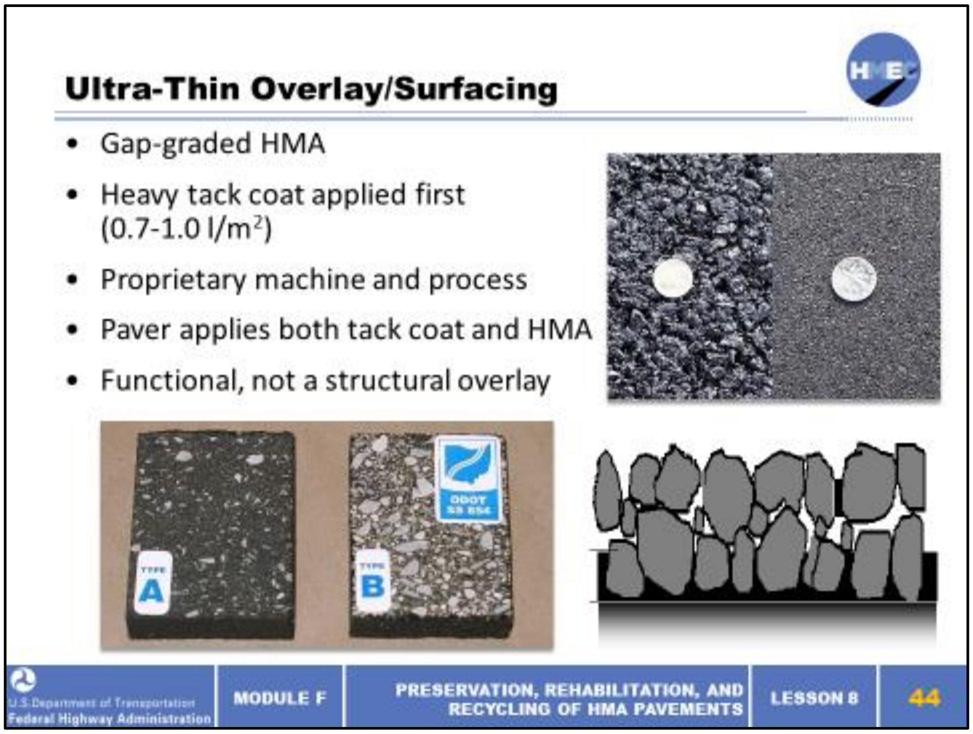
Slide 44



Ultra-thin overlay advantages:

• Can be placed in one pass, without milling.
• Quick construction and return to traffic mean shorter user delays and lower user delay costs.
• Coarse aggregate matrix results in reduced backspray for greater visibility in wet weather as well as good skid resistance.
• Special NovaBond polymer modified asphalt membrane for superior bonding to and protection of existing surface.
• A spray paver is used in this application so there is not tracking of the tack coat.
• Ultra-thin lift means maintaining overhead clearances, curbs, and drainage profiles, as well as lower costs.
• Construction by specially trained, reliable, quality contractors.
• Long-lasting performance.

The NOVACHIP is a specific type of ultra-thin overlay that is a proprietary product that was mentioned in the previous slide.

Slide 45


Example Projects – Comparison
Ultra-Thin Friction Course
Microsurfacing
U.S. Department of Transportation
Federal Highway Administration
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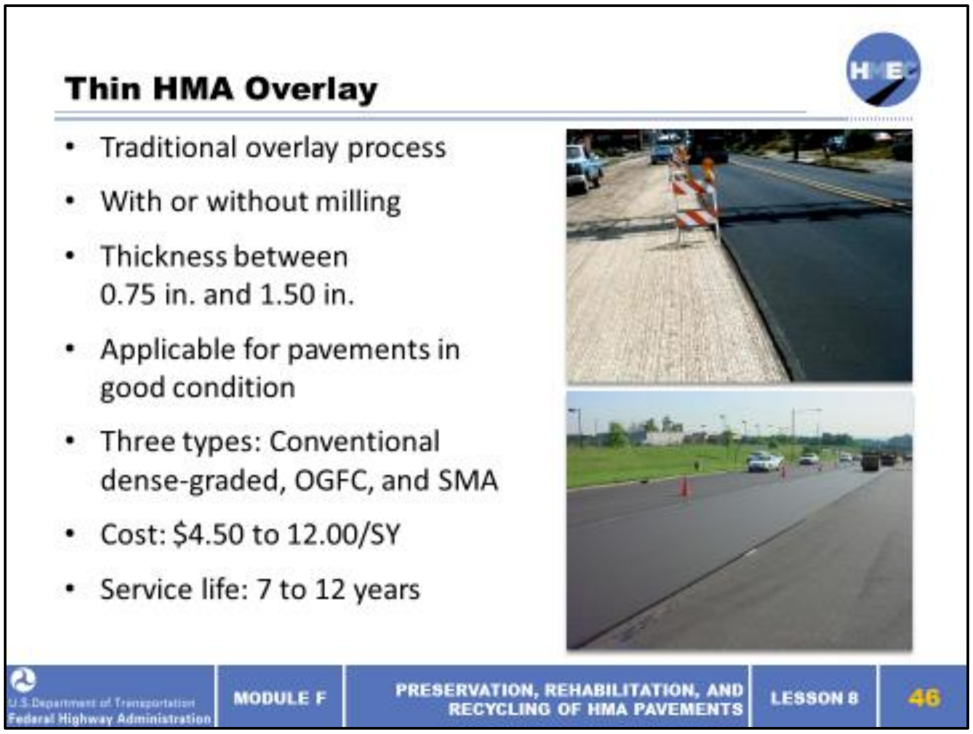
Slide 46



The combination of cold milling and the application of a thin HMA overlay is a viable option for improving ride ability and surface friction, reducing hydroplaning and tire splash (using an open graded friction course), and improving the profile, crown, and cross slope of an existing pavement. The process begins with the removal of a portion of the existing pavement surface (using carbide-tipped cutting bits) to a specified depth (typically 0.75 in. to 1.50 in.). Cold milling is generally not suitable for pavements with significant deterioration or distress.

Benefits associated with thin HMA overlays include the following:

• Improved ride quality
• Improved surface friction
• Enhanced appearance
• Sealed surface
• Reduced splash and spray
• Balancing the industry

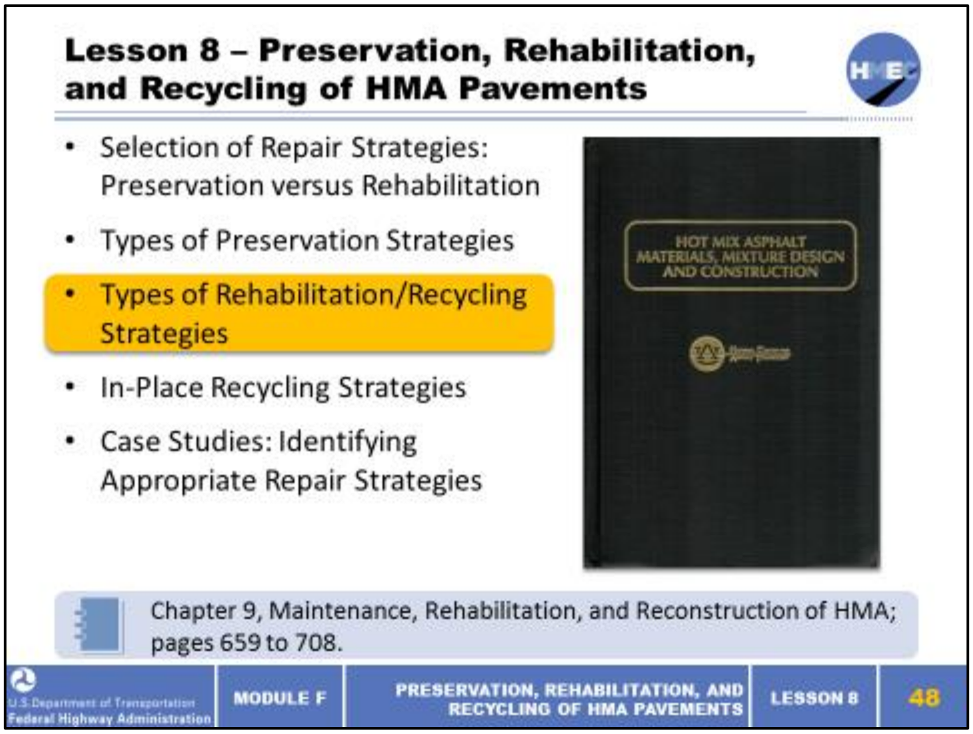
The following three different types of thin HMA overlays are available:

• Conventional dense-graded—Aggregate is uniformly distributed throughout the full range of sieve sizes.
• Open graded friction course (OGFC)—Aggregate particles are uniformly graded (or predominantly a single size). Additional benefits provided by an OGFC are reductions in hydroplaning and tire splash and spray. Note that this technique may also be used on thin overlays of PCC pavement.
• Stone matrix asphalt (SMA)—Aggregates are gap-graded, meaning that they contain coarse fractions and fine aggregate sizes, but no medium aggregate sizes. SMA also uses a high percentage of mineral filler (8% to 10%), and stabilizing additives such as fibers and polymers are added to control segregation and draindown.

Slide 47

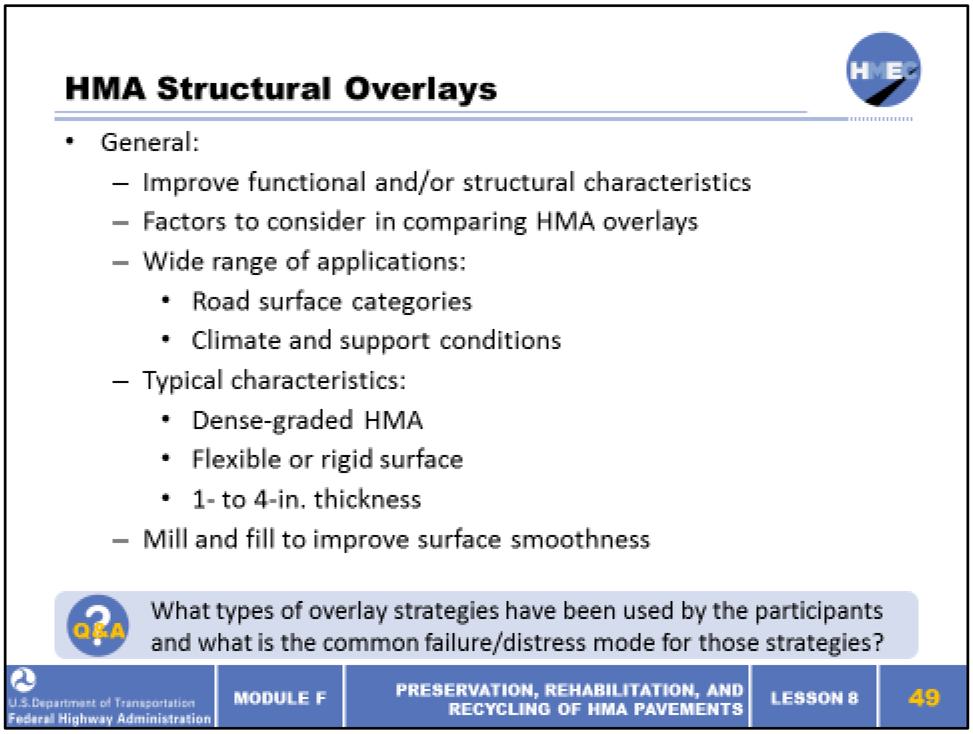
The schematic at the bottom of the screen shows the different equipment and materials used to place the thin overlay.

Slide 48



The information covered within this lesson comes from many sources which will be listed on the next set of screens, but was generally taken from Chapter 9, Maintenance, Rehabilitation, and Reconstruction of HMA of the NCAT textbook. Another major document for establishing the condition of the existing pavement is the 2008 AASHTO Mechanistic-Empirical Pavement Design Manual of Practice.

Slide 49



The general purpose of an HMA overlay is to improve the functional or structural performance of an existing pavement or road surface. Overlays can be very effective at addressing problems with existing surface deficiencies and/or increasing the load-carrying capacity of the pavement.

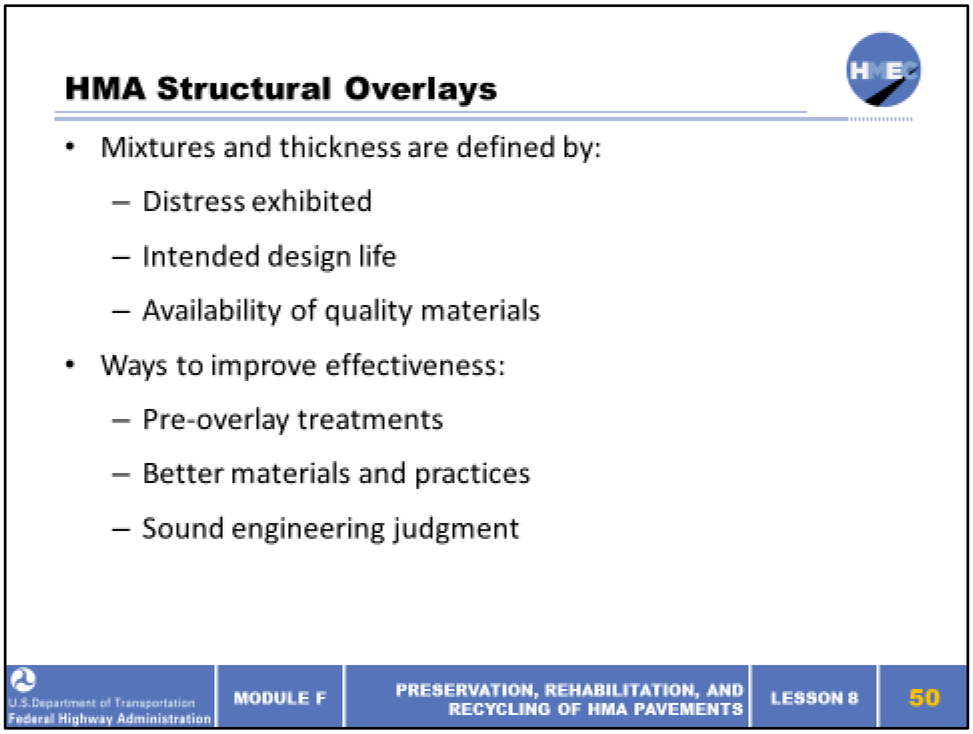
HMA overlays have a wide variety of applications ranging from upgrading low-volume roads to improving heavily-trafficked freeways and highways. In addition, with proper attention to traffic and environmental effects during structural and mix design, HMA overlays can be applied successfully over a wide range of climate and support conditions.

The most common HMA overlay is constructed with dense-graded, hot-mixed asphalt. The important consideration is the total thickness of the overlay in determining the mixtures or layers included in the overlay strategy. Depending upon the problem in the existing pavement, and on the purpose of the overlay, the thickness of HMA overlays may range from 1 to 4 in. or more.

HMA overlays may often be applied in conjunction with cold milling. The use of cold milling is an effective means of restoring cross slope, maintaining curb lines, and preparing the existing pavement to receive the overlay. The removal of a portion of the existing structure must be accounted for in determining the required overlay thickness.

The fact that overlays have a wide variety of applications does not mean that they have no limitations or that their effectiveness is always assured. Many overlays fail to provide the performance level and useful life expected because the condition of the existing pavement was not properly addressed and considered in the overlay design. The following screens focus on HMA overlays of existing flexible pavements

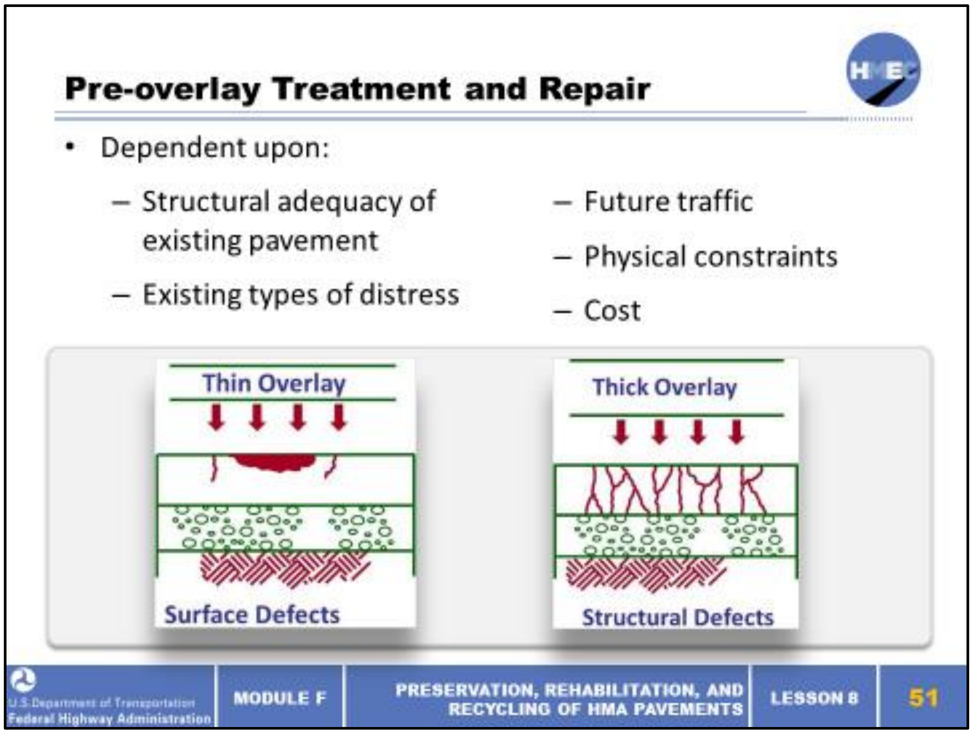
Slide 50



Despite their sensitivity to these factors, agencies have been able to maximize the effectiveness of HMA overlays as a rehabilitation option by exercising various types of pre-overlay treatments (e.g., geotextiles, heater-scarification, cold-milling, leveling courses, chip seals, crack seals, etc.), repairs (i.e., patching) and through the use of better quality materials and construction practices.

Consequently, if sound engineering judgment is exercised during design, it is possible to minimize the limitations of HMA overlays and simultaneously improve their effectiveness. Module E provides a summary on the use of the MEPDG for designing HMA overlays under different conditions.

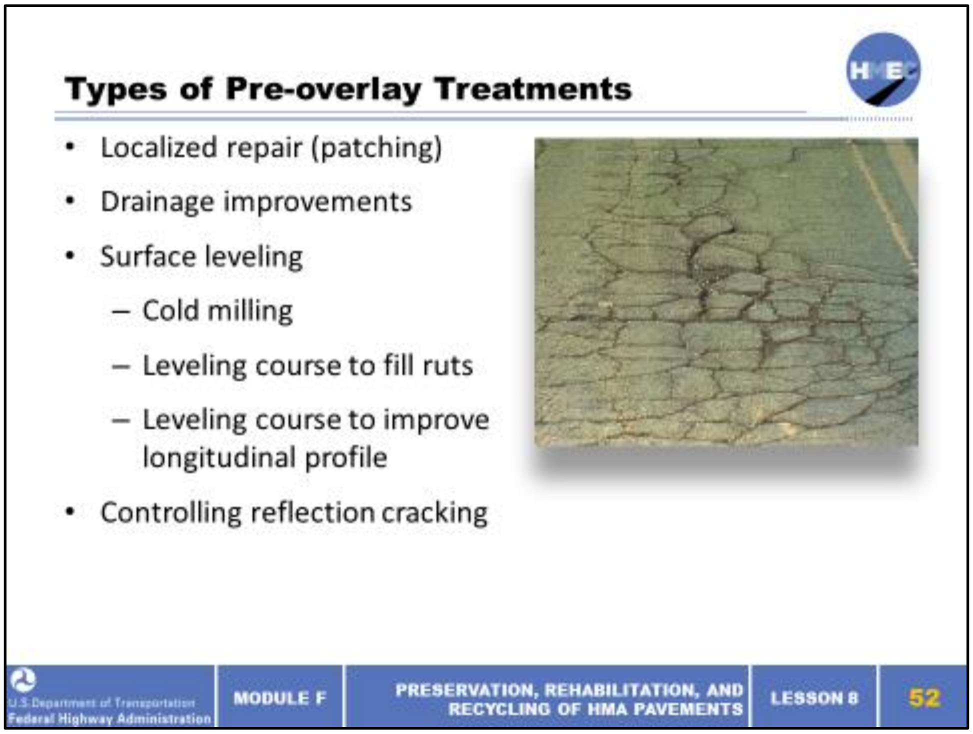
Slide 51



Certain performance indicators can be used to determine whether a pavement is structurally or functionally deficient. The amount of repair and treatment that is performed to a pavement prior to overlay is probably the single most important factor that affects the future performance of the overlay.

The amount and type of pre-overlay restoration needed on an existing pavement must be carefully determined by considering the factors listed on the screen.

Slide 52



Any drainage issue or deficiency should be corrected, regardless of which repair technique is selected—varying from structural overlays to reconstruction. A common pre-overlay treatment is to level the surface prior to placing the overlay and other repair strategies. The surface leveling is typically achieved by the following treatments, as noted on the screen:

• Transverse surface irregularities in flexible pavements (permanent deformation, crown problems, and so on) can be corrected by milling the surface to remove the irregularities by removing the unstable asphalt concrete surface layer prior to overlay, or by filling the ruts with a stable leveling course that is properly compacted prior to placement of the overlay.
• Milling is by far the easiest and most commonly used method. Long wavelength longitudinal profile irregularities (e.g., settlements and heaves) can be corrected with a leveling course overlay.
• An average HMA overlay thickness of about 0.80 in. is used for estimation purposes for a typical pavement to provide the required level up and surface profile.

Selecting a structural overlay usually implies a surface with various types and severities of cracking. One of the issues with using structural overlays are cracks in the existing surface, which can reflect through the overlay. Thus, controlling or mitigating the reflection cracks is a key issue for the repair strategy.

The next set of screens provides the different mitigation techniques that have been used to retard and/or delay reflection cracks.

Slide 53

Drainage Improvements/Corrections

- Drainage survey
- Identify moisture-/drainage-related distresses
- Develop solutions that address moisture problems

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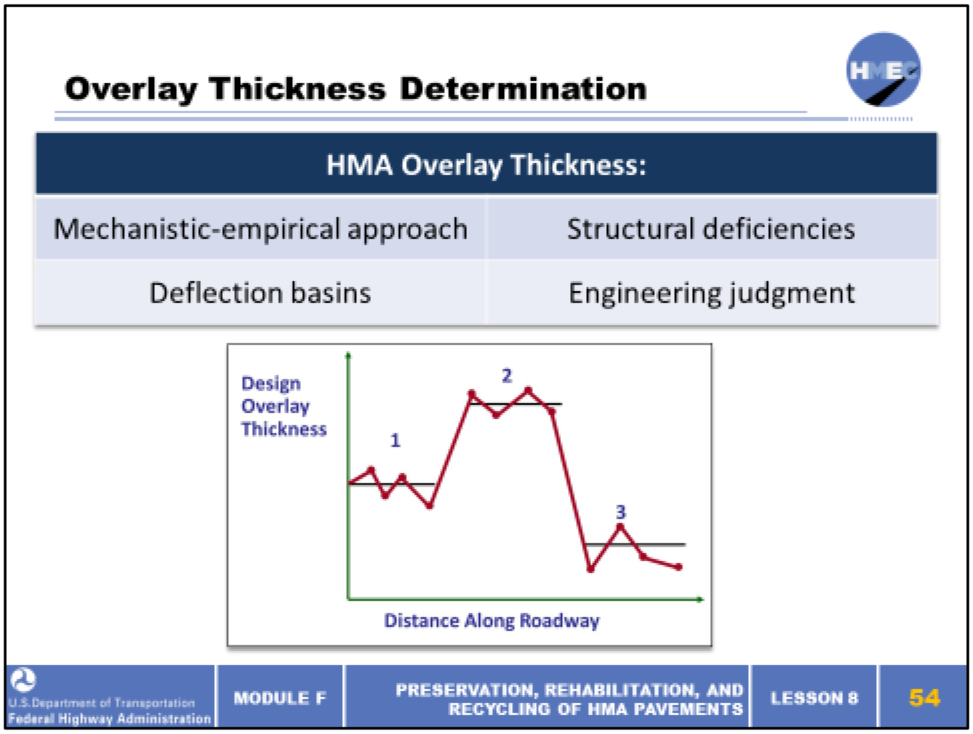
It is always advisable to conduct a thorough drainage survey, identify drainage-related distresses, and develop solutions that address these distresses as part of the overlay design process. Improving poor subsurface drainage conditions will have a beneficial effect on the performance of the overlay. Removal of excess water from the pavement cross section will reduce erosion and increase the strength of the base and subgrade soil, which in turn will reduce deflections. In addition, stripping in HMA layers may be slowed by improved drainage.

An important point to note is that most overlay design procedures assume that the existing pavement has good drainage.

Slide 54



Overlay thickness design procedures either assume that adequate pre-overlay repair and reflective crack control actions are taken, or they permit different levels of repair to be considered. If pre-overlay and reflective crack control treatments are inadequate, then the overlay will likely fail prematurely and not provide the designed structural service.

Due to variations in the structural integrity of the pavement along the roadway, the structural requirement (i.e., overlay thickness) may also vary.

As a result, the use of deflection basin measurements and analysis of the deflection basins through backcalculation of elastic layer moduli become very valuable. Module E overviews the use of deflection basins in determining the damage of the in-place asphalt layers in determining the thickness of the required overlay. More importantly, the use of deflection basin measurements can be used to stratify the project or determine the design segments. It can also identify areas with structural deficiencies or subsurface issues along the project. This can be used to strategically locate borings and cores to explain those differences.

Slide 55

Reflection Cracking Mechanisms

1	Thermal expansion & contraction of joints & cracks	Thermal Induced	Crack initiates in tension & propagates in tension & shear
2	Differential vertical deflections across joints & cracks	Traffic Induced	Crack initiates & propagates in shear
3	Curling of PCC slabs; PCC temperature gradients	Thermal Induced	Crack initiates in tension & propagates in tension & shear

Q&A What type of reflection cracking mitigation techniques have been used by the participants and what were their success?

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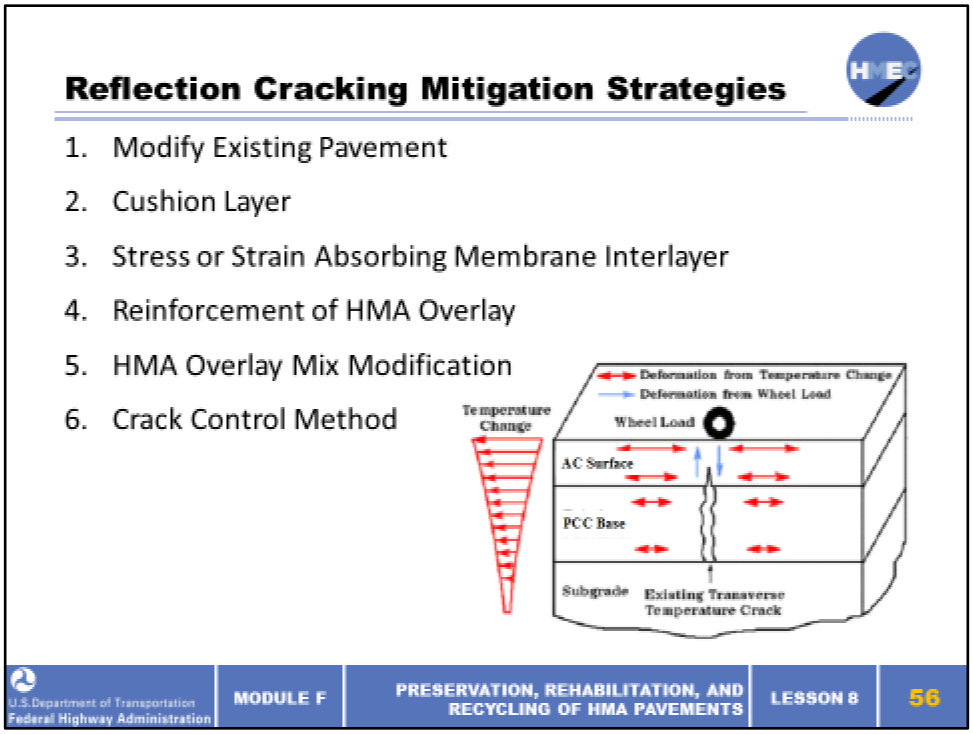
The mechanisms of reflection cracking are listed on this screen. It is important to point out the difference between reflection cracks occurring from horizontal and differential vertical movements.

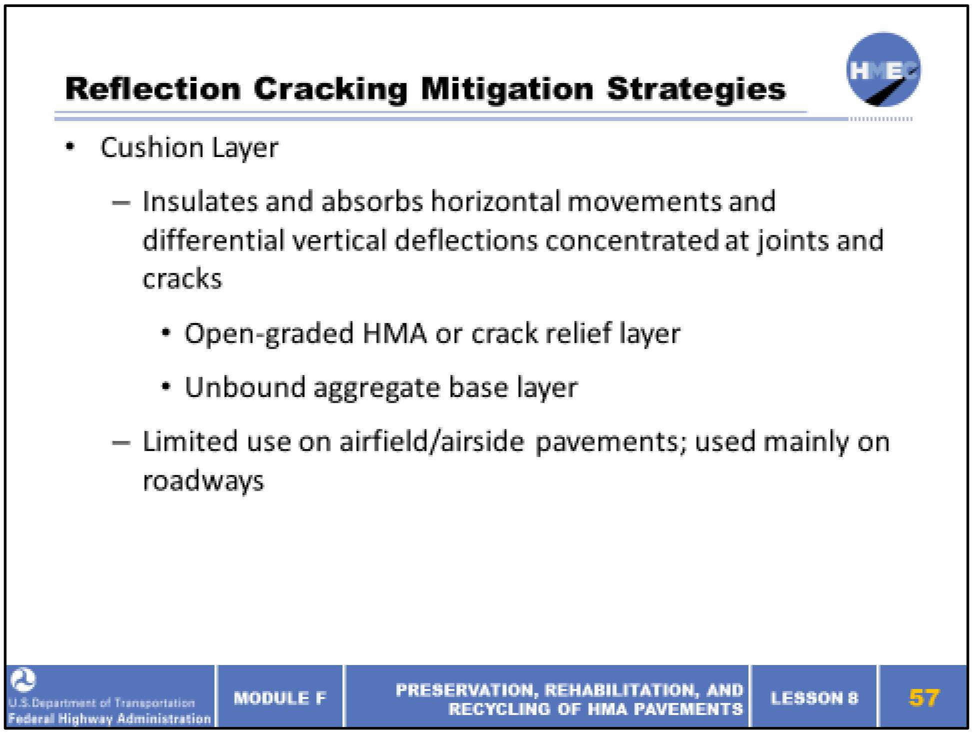
Slide 56



Although reflection cracking tends to be a more serious problem on overlays of existing rigid pavements, it can also be a significant problem in HMA overlays on flexible pavements with active transverse cracks.

The following sets of screens summarize and define each reflection cracking mitigation strategy.

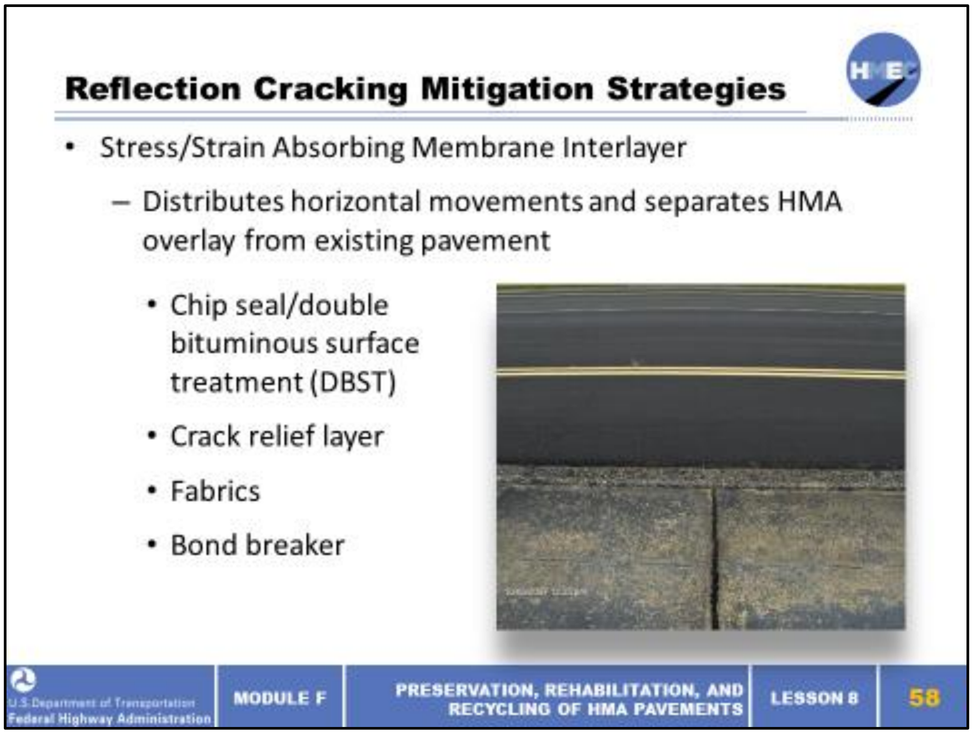
Slide 57



The success of this strategy has had mixed results but has not been used extensively in overlays over flexible pavements.

A couple of reasons for this limited use in the increased thickness above the existing HMA surface and the increased potential for moisture damage or stripping in the existing HMA layers. For example, it is important to note that a disadvantage of using open-graded HMA or crack relief layers above existing dense-graded mixtures is that the open-graded mixture can hold water for longer periods of time. If the existing dense-graded layers are susceptible to moisture damage, this can result in accelerated stripping of the asphalt in the existing dense-graded mixture.

Slide 58

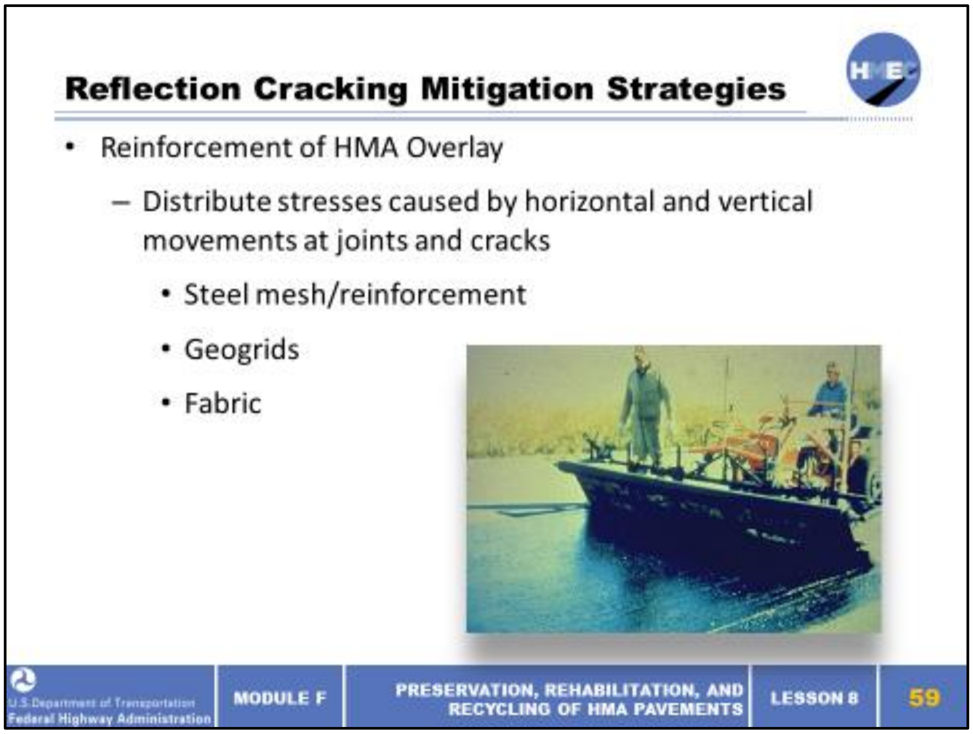


Chip seals and crack relief layers have been the better methods for mitigating the occurrence of reflection cracks.

A key issue with bond breakers (layer of sand, dust, or other similar materials) require thicker overlays because of slippage cracks that can occur in areas with horizontal stresses. The use of fabrics has had mixed results.

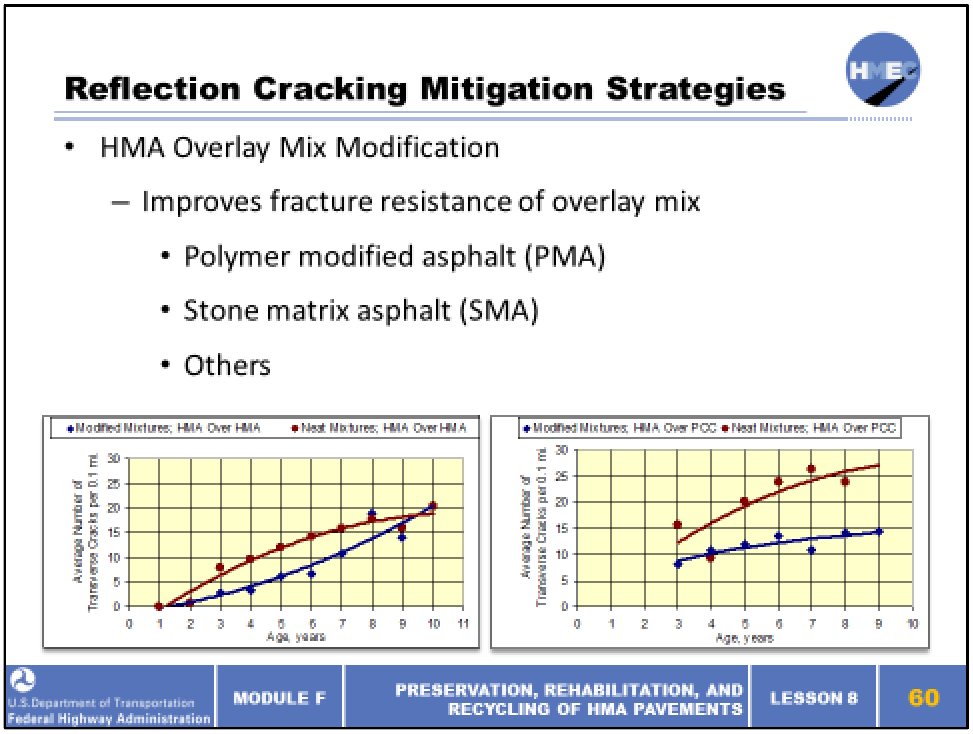
In the slide, DBST is a double bituminous surface treatment.

Slide 59



Some of the reinforcing materials has been relatively good in reducing the severity of the reflection cracks (geogrids), while some of the fabrics have not been very effective in mitigating reflection cracks of HMA overlays.

Slide 60



This method has been less effective in mitigating reflection cracks of HMA overlays.

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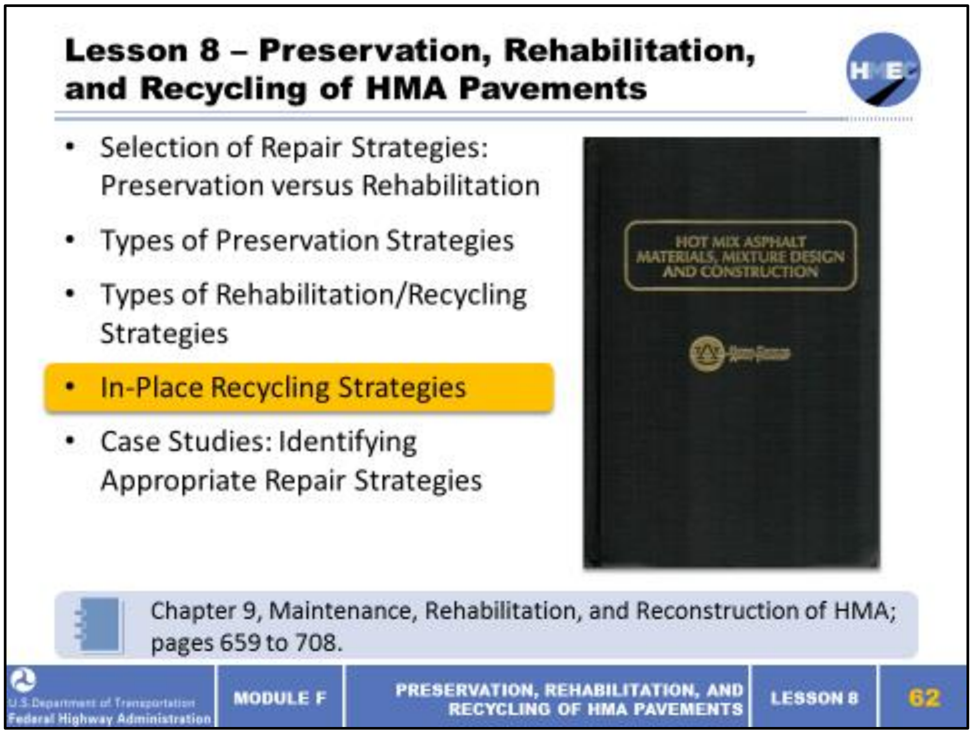
The crack control method or saw and seal method had been very effective in reducing the deterioration of reflection cracks in overlay of JPCP. Point out that sawing the HMA above the joints in a JPCP is actually a crack but the sealer placed in the sawed geometry helps prevent deterioration of the sawed joint.

One of the critical issues with the performance of the saw and seal method is making sure the HMA is sawed directly above the joint in the JPCP. If the HMA is not sawed above the joint, a secondary crack will develop above the joint and significant deterioration of the HMA will occur.

Slide 62



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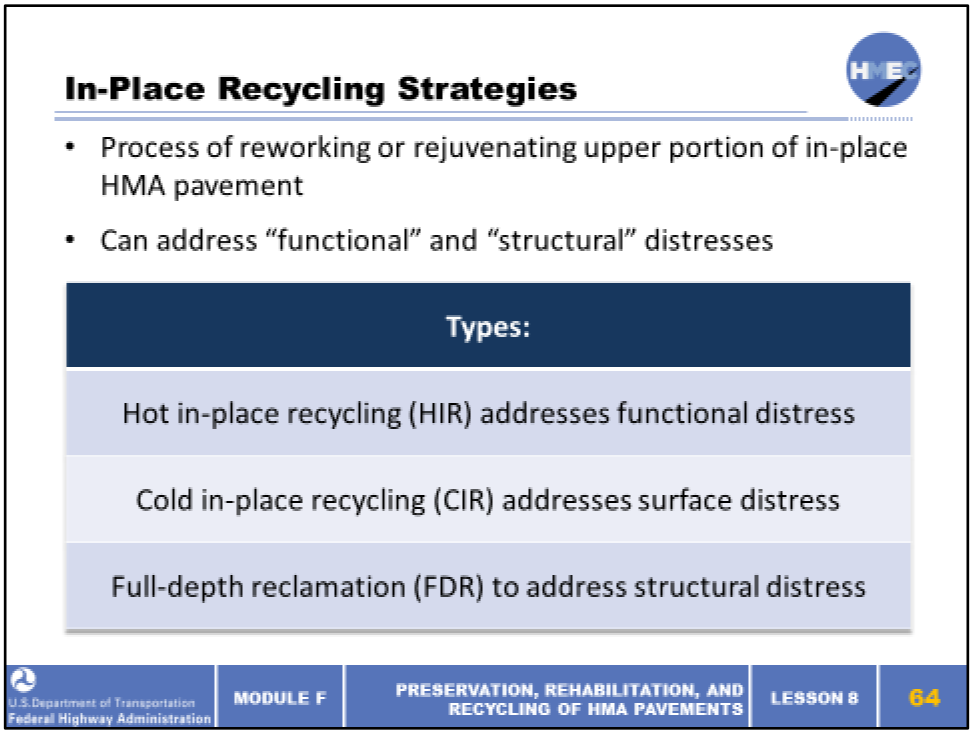
Slide 63

Because of their similarities, this series of screens covers both cold in-place (CIR) and hot in-place recycling (HIR).

Slide 64



CIR and FDR treatments are normally associated with rehabilitation and reconstruction, i.e., when the pavement is in relatively poor condition, while HIR treatment is normally associated with the rehabilitation of a pavement that has adequate structural strength but functional deficiencies. The primary goal of this section is to describe how these treatments can be used within a pavement repair program. FDR is used when the pavement exhibits significant structural distress, CIR is used when the pavement exhibits surface related distresses or top down cracking, while the HIR method can be used on pavements exhibiting only functional related distress.

This part of the lesson is divided into three sections for HIR, CIR, and FDR. The recycling treatments referenced here can be generally described as treatments that "rework" or "rejuvenate" the upper portion of an HMA pavement surface. Some of the more common reasons for applying a recycling treatment are listed below:

• The primary purpose of recycling is to address distresses that are limited to the upper portion of the pavement. This reworking of the upper portion of the surface layer addresses surface distresses such as minor cracking, corrugations, raveling, bleeding, low surface friction, and rutting.

• Improve profile, crown, and slope.

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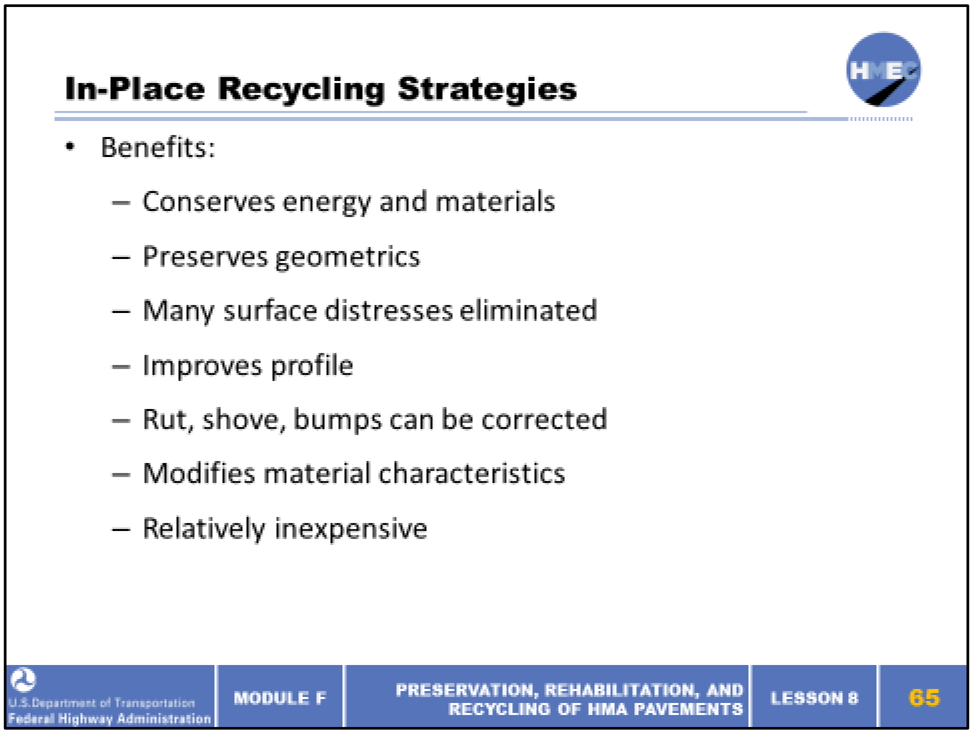

There are a number of distinctive benefits of using in-place recycling instead of other techniques. In general, the following list of benefits applies to both CIR and HIR:

• Conserves energy and materials.
• Preserves pavement geometrics.
• Many surface distresses can be eliminated (e.g., ruts, shoves, raveling, bumps). Pavement should, however, be structurally sound with a good draining base.
• Some profile improvement is possible.
• Ruts, shoves, bumps can be corrected.
• Aggregate gradation and asphalt content can be modified.
• Relatively inexpensive, minimal hauling.

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The discussion of recycling will be kept general in this course. For example, the details of how you determine which HIR recycling method to use (i.e., surface recycling, remixing, or repaving) is based on a number of factors, including the types of distresses that are present on the current pavement.

The details of all recycling methods can be found in the Basic Asphalt Recycling Manual noted above. There is also an NHI course on recycling: 131050, Asphalt Pavement Recycling Technologies.

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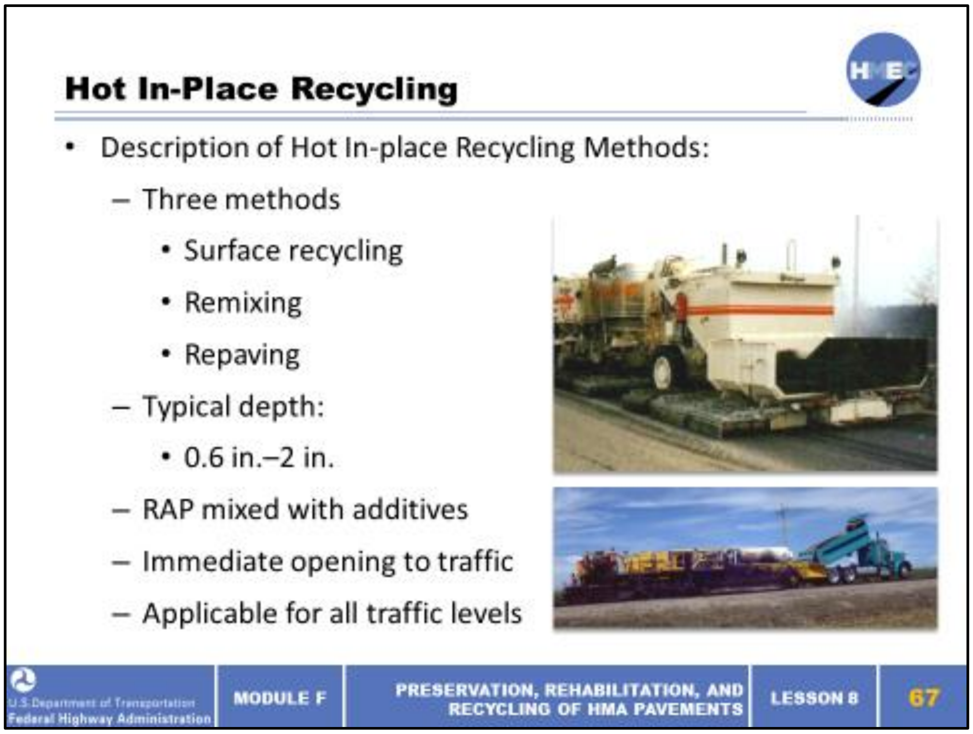
Slide 67



The primary purpose of HIR is to correct surface distresses such as raveling, corrugation, rutting, bleeding, friction loss, and minor cracking. Distresses attributed to subgrade or base failures cannot be remedied by HIR. HIR can also be used to restore profile, crown, and cross slope.

There are three types of HIR techniques available:

1. Surface Recycling or Heater-scarification is the earliest form of HIR and is a simple process in which the surface of the pavement is heated, scarified with a set of scarifying teeth, mixed with a recycling agent, and then leveled and compacted.
2. The remixing technique removes a portion of the existing pavement and then mixes it with controlled amounts of virgin mix and/or rejuvenating agents in an on-board pugmill. The resultant mixture is then placed as the new surface course.
3. The repaving technique heats the existing pavement and mills or scarifies it to a depth of 0.75 to 1 in., and then mixes in a recycling agent. This recycled material is then placed as a leveling course and is then followed with a hot-mix wearing surface (either immediately or at some later time).

The following summarizes the steps/major activities or details in using the HIR method:

• Scarifiers associated with surface recycling have a depth of impact in the range of 0.6 in.to 1 in.
• Hot milling equipment associated with remixing and repaving has a depth range of 1 in. to 2 in.
• The recycled material is typically mixed with some additives (i.e., rejuvenators, virgin aggregates, additional asphalt), then relaid and recompacted.
• HIR uses applied heat to soften the existing HMA surface.
• The milling depth depends on the specific process selected for the project.
• Scarifiers are used to remove the top layer of the existing HMA surface.

Some of the disadvantages in using the HIR method are listed below:

• Does not work well with pavements with too much variation in HMA mix.
• Any paving fabrics or interlayers in the anticipated treatment depth plus 25% cannot be recycled with HIR. Also, the presence of an interlayer needs to be assessed further depending on its characteristic and location.
• Deep ruts greater than one-half of the anticipated HIR treatment depth impose limitations on the type of HIR process.
• Depending on the top size of aggregate used in surface lift, large stone mixes may not be effectively recycled using all HIR processes.
• Presence of rubber in the surface lift or rubberized seal coats will require special attention in the mix design process. It will also cause some difficulties during construction because it has such a high affinity for the rubber in the tires of the HIR and compaction equipment.
• Moisture content significantly influences the production rates of the various HIR processes.

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A schematic of the equipment used to complete HIR is shown in the screen.

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Some of the characteristics associated with CIR include:

- No heat is used for CIR (hence the name).
- RAP is sized, mixed with additives (e.g., asphalt binder, emulsion, rejuvenator, and/or virgin aggregate), and relaid.

The following summarizes the major activities or details in using the CIR method:

- Cold milling equipment is used to remove only the top layer of the existing HMA surface.
- Typically resurfaced with a surface treatment or an HMA overlay.
- Residual moisture is approximately 16% after construction. Depending on ambient conditions, several weeks may be needed to allow moisture to evaporate before final surface is placed.
- CIR is effective at correcting surface distresses and some materials-related problems.
- Curing time is one of the main reasons CIR is applicable more to low-volume roads.

Some of the specific advantages in using CIR include the following (some additional advantages are listed on the following screen):

• Minimal hauling;
• Minimal air-quality problems; and
• Capability for widening.

Some of the specific disadvantages of using CIR include:

• Curing time;
• Control of moisture content;
• Requires wearing surface;
• Limited maximum size of RAP; and
• Utility appurtenances.

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A key at this point is the material properties and how those properties are measured for use in structural design. Point out some of the differences in support between the use of an additive and no additive. Another point to emphasize is the difference between the use of emulsion and Portland cement in terms of reflection cracks. The amount of Portland cement selected should be sufficient to increase the stiffness of the FDR layer but not to the level where shrinkage cracks develop that will reflect to the surface.

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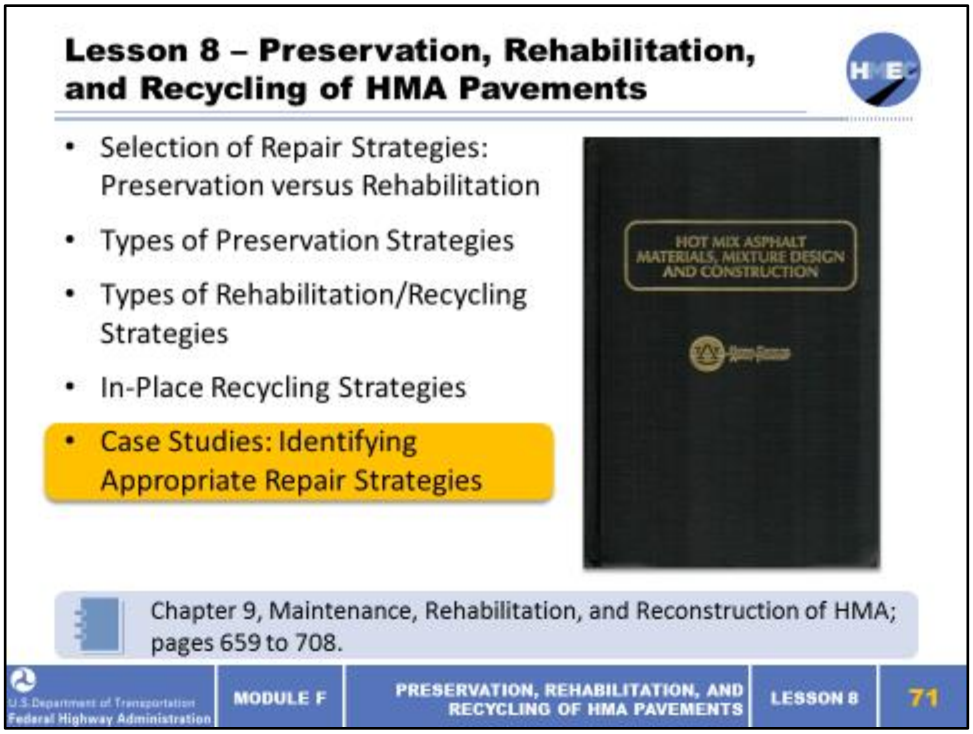

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Slide 72


Case Study: Appropriate Repair Strategy?
What repair strategies are considered an option for the different pavement conditions and why?
Q&A
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Case Study: Appropriate Repair Strategy?
What repair strategies are considered an option for the different pavement conditions and why?
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Case Study: Excessive Distress—No Remaining Life?
What repair strategies are considered an option for the different pavement conditions and why?
Q&A
U.S. Department of Transportation
Federal Highway Administration
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Case Study: Appropriate Repair Strategy?
What repair strategies are considered an option for the different pavement conditions and why?
U.S. Department of Transportation
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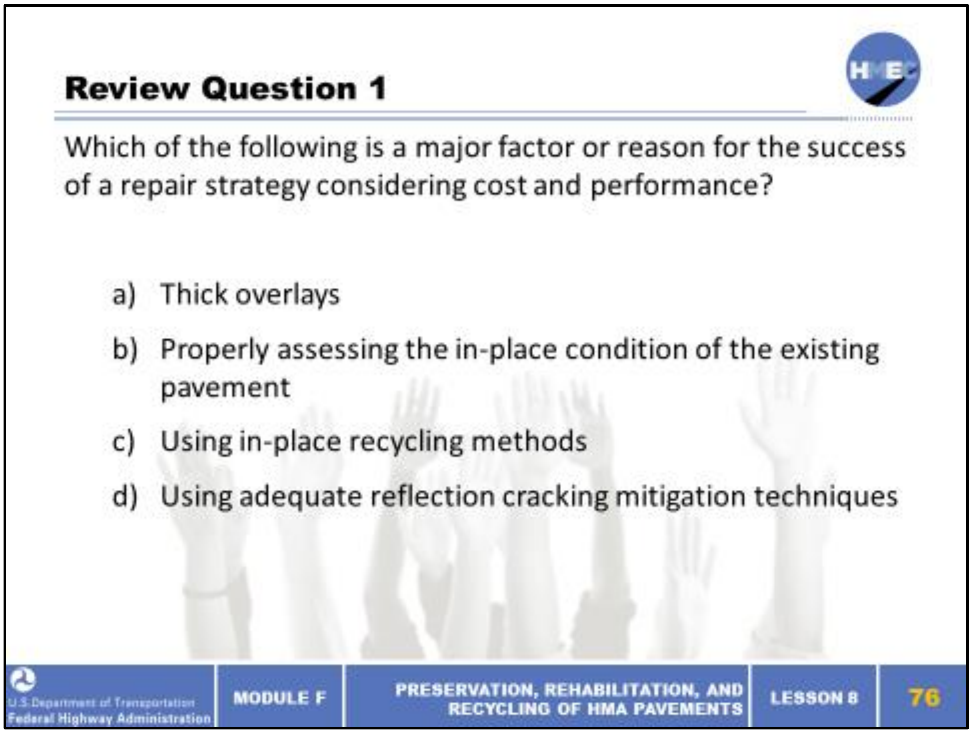
Review Question 1
Which of the following is a major factor or reason for the success of a repair strategy considering cost and performance?
a) Thick overlays
b) Properly assessing the in-place condition of the existing pavement
c) Using in-place recycling methods
d) Using adequate reflection cracking mitigation techniques
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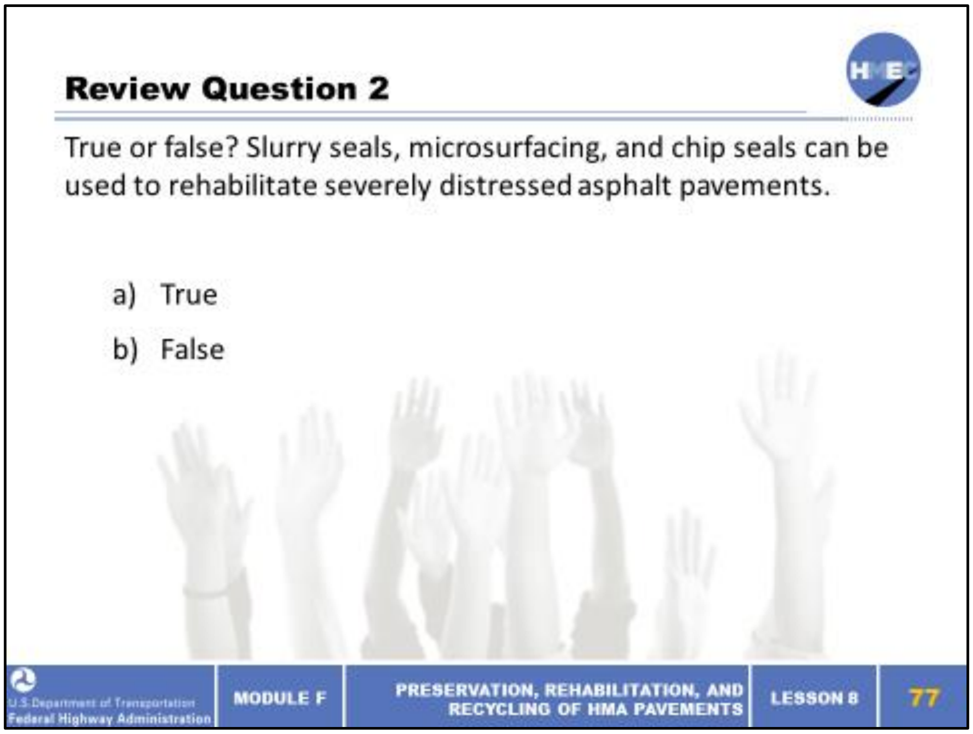
Review Question 2
True or false? Slurry seals, microsurfacing, and chip seals can be used to rehabilitate severely distressed asphalt pavements.
a) True
b) False
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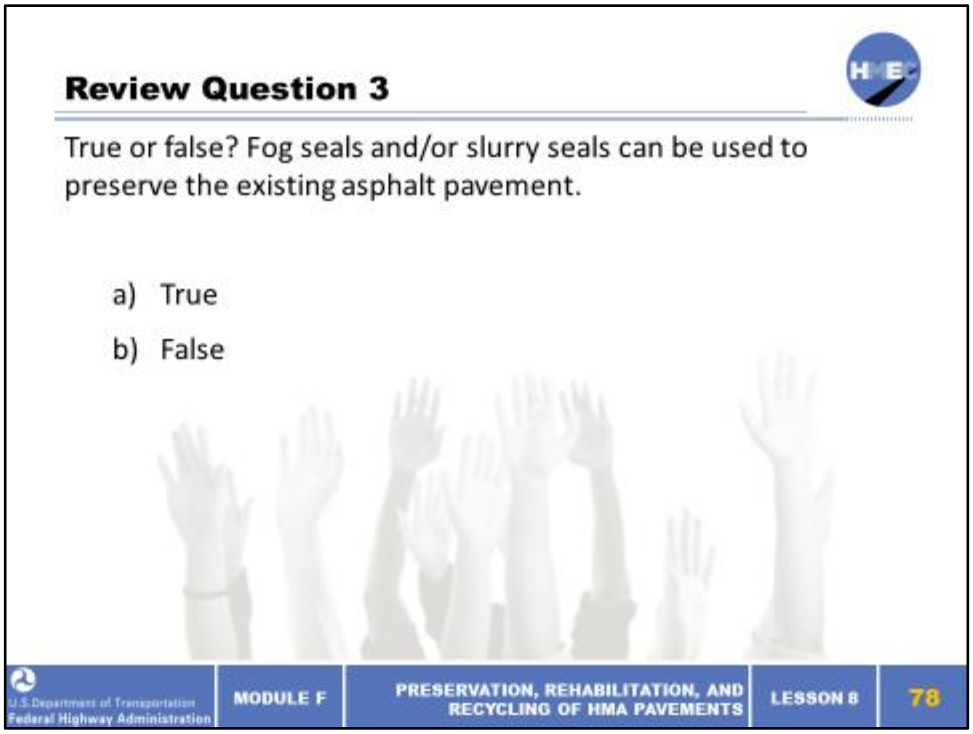
Review Question 3
True or false? Fog seals and/or slurry seals can be used to preserve the existing asphalt pavement.
a) True
b) False
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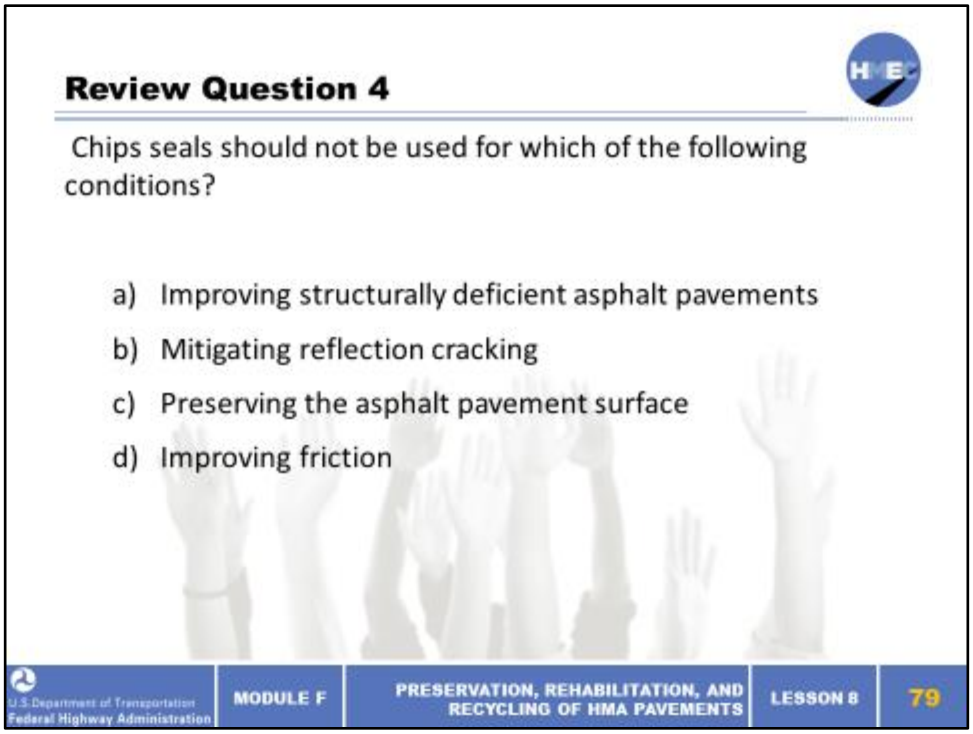
Review Question 4
Chips seals should not be used for which of the following conditions?
a) Improving structurally deficient asphalt pavements
b) Mitigating reflection cracking
c) Preserving the asphalt pavement surface
d) Improving friction
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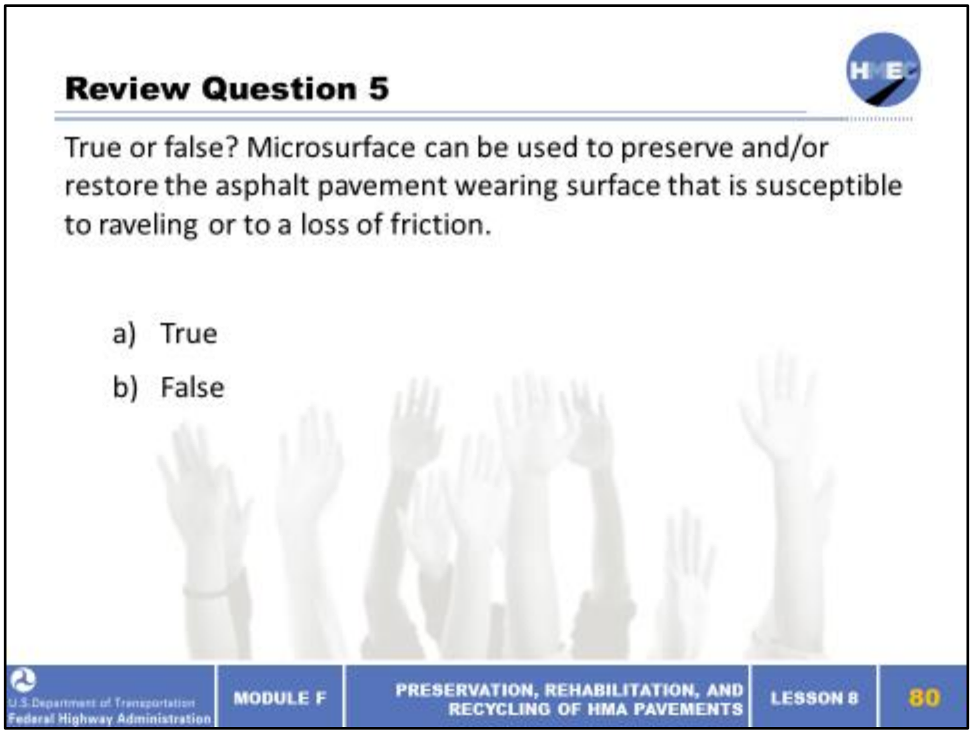
Review Question 5
True or false? Microsurface can be used to preserve and/or restore the asphalt pavement wearing surface that is susceptible to raveling or to a loss of friction.
a) True
b) False
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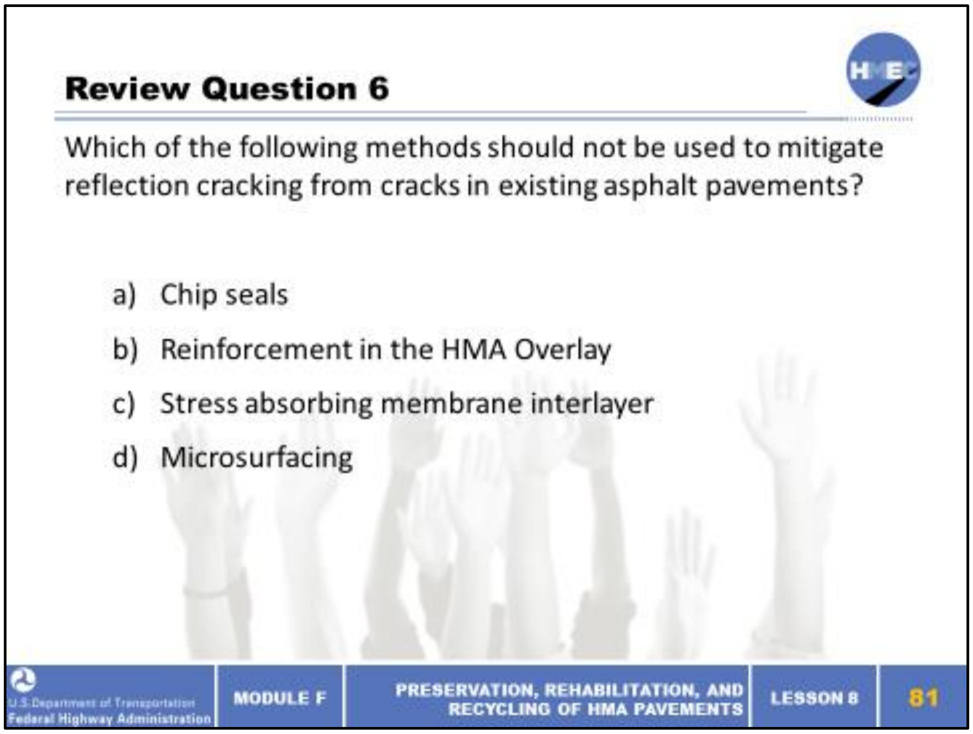
Review Question 6
Which of the following methods should not be used to mitigate reflection cracking from cracks in existing asphalt pavements?
a) Chip seals
b) Reinforcement in the HMA Overlay
c) Stress absorbing membrane interlayer
d) Microsurfacing
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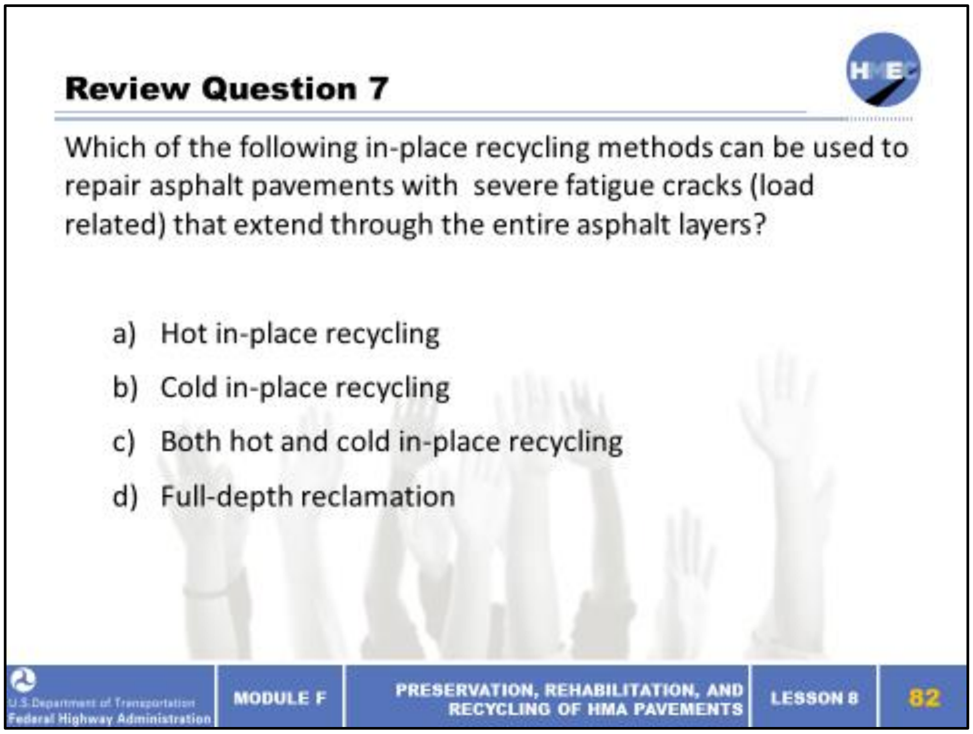
Slide 82


Review Question 7
Which of the following in-place recycling methods can be used to repair asphalt pavements with severe fatigue cracks (load related) that extend through the entire asphalt layers?
a) Hot in-place recycling
b) Cold in-place recycling
c) Both hot and cold in-place recycling
d) Full-depth reclamation
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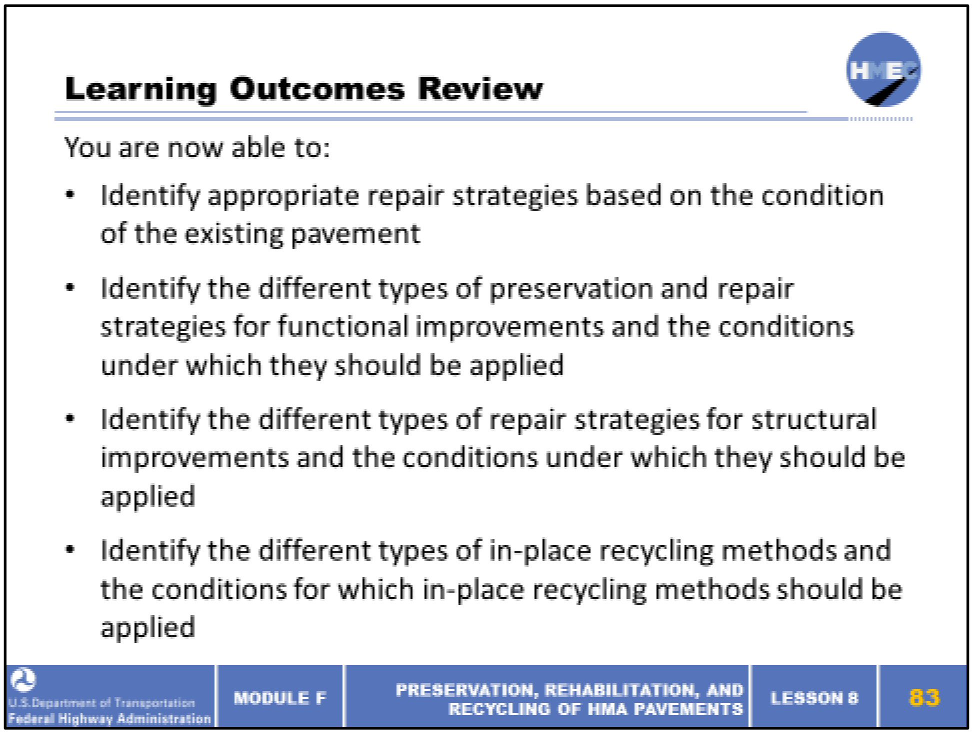
Slide 83


Learning Outcomes Review
You are now able to:
• Identify appropriate repair strategies based on the condition of the existing pavement
• Identify the different types of preservation and repair strategies for functional improvements and the conditions under which they should be applied
• Identify the different types of repair strategies for structural improvements and the conditions under which they should be applied
• Identify the different types of in-place recycling methods and the conditions for which in-place recycling methods should be applied
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Federal Highway Administration
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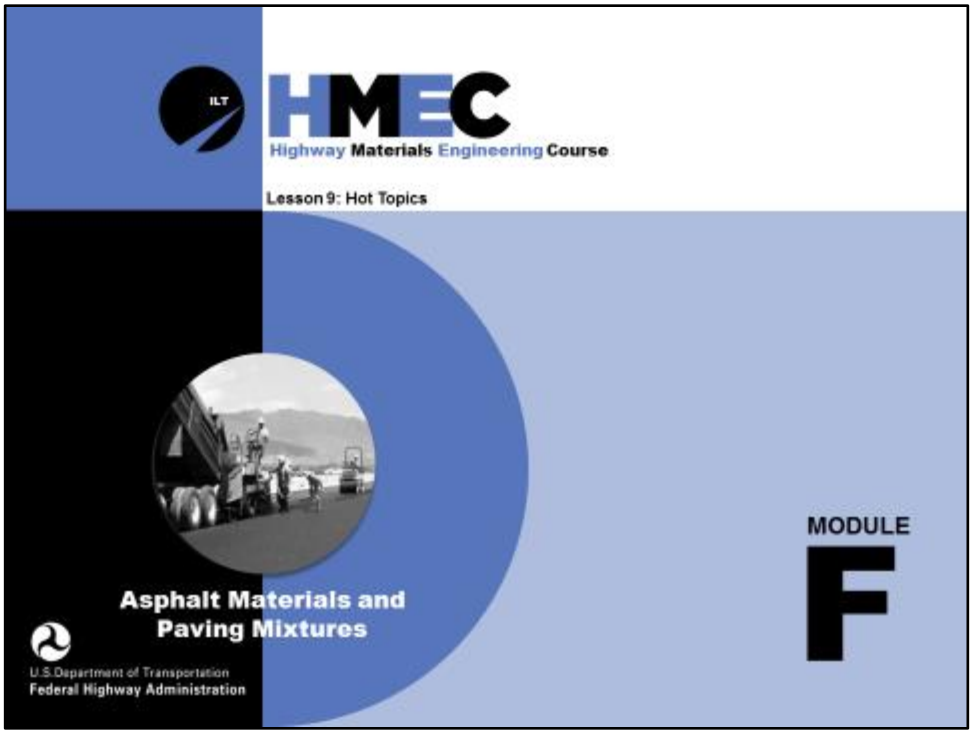
Slide 1


ILT
HMEC
Highway Materials Engineering Course
Lesson 9: Hot Topics
MODULE
F
Asphalt Materials and Paving Mixtures
U.S.Department of Transportation
Federal Highway Administration

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Appendix A: Acronyms

The following are acronyms referenced throughout the course that are important agencies or organizations:

Acronym	Proper Name
AASHTO	American Association of State Highway and Transportation Officials
ACAA	American Coal Ash Association
ACI	American Concrete Institute
ACPA	American Concrete Paving Association
AI	Asphalt Institute
ASTM	American Society for Testing and Materials
AWS	American Welding Society
CFR	Code of Federal Regulations
DOT	U.S. Department of Transportation
EPA	Environmental Protection Agency
FHWA	Federal Highway Administration
NACE	National Association of Corrosion Engineers
NAPA	National Asphalt Pavement Association
NCAT	National Center for Asphalt Technology
NCHRP	National Cooperative Highway Research Program
NEPCOAT	North East Protective Coating
NHI	National Highway Institute
NRC	National Recycling Coalition

Acronym	Proper Name
NRMCA	National Ready Mixed Concrete Association
NSA	National Slag Association
NSBA	National Steel Bridge Alliance
NTPEP	National Transportation Product Evaluation Program
OSHA	Occupational Safety and Health Administration
RCSC	Research Council on Structural Connections
SSPC	Society for Protective Coatings
TRB	Transportation Research Board
USGS	U.S. Geological Survey

Appendix B: Resources

Additional information regarding Module F can be found in the following sources.

National Asphalt Pavement Association (NAPA)

www.hotmix.org/index.php?option=com_content&task=view&id=192&Itemid=316

FHWA Distress Identification Manual for Long-Term Pavement Performance Program (FHWA Publication Number FHWA-RD-03-031)

https://www.fhwa.dot.gov/publications/research/infrastructure/pavements/ltpp/reports/03031/

NCAT Hot Mix Asphalt Materials, Mixture Design and Construction, 2nd edition or later

http://www.ncat.us/info-pubs/ncat-textbook.html

Hot Mix Asphalt Paving Handbook, Corps of Engineers FHWA/NAFA

http://www.fhwa.dot.gov/publications/focus/01jul/handbook.cfm

MS-4 Asphalt Handbook, Asphalt Institute

https://www.asphaltinstitute.org/

MS-22, Construction of Hot Mix Asphalt Pavements

MS-24 Asphalt Binder Testing Manual, Asphalt Institute

HMA Pavement Smoothness: Characteristics and Best Practices for Construction (FHWA Publication FHWA-IF-02-024)

https://www.fhwa.dot.gov/pavement/pub_details.cfm?id=33

Basic Asphalt Recycling Manual (BARM)

http://www.cdrecycling.org/assets/concrete-recycling/1-124-barm1.pdf

Basic Asphalt Emulsion Manual, AI MS 19

http://www.asphaltmagazine.com/news/detail.dot?id=7910cd49-b588-4253-b016-56c003c6a731

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