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16. Abstract

The focus of this project is to develop an integrated hot-mix asphalt (HMA) mixture design method which balances both rutting and cracking requirements. The Hamburg Wheel Tracking Test (HWTT) and Overlay Tester (OT) devices were used to evaluate the rutting and cracking resistance of HMA mixtures, respectively. Eleven mixtures commonly used in Texas were designed following the current Texas Department of Transportation (TxDOT) mixture design process and then evaluated under the HWTT and the OT. It was found that the Dense-Graded and Superpave mixtures designed following current TxDOT mixture design procedures were rut resistant, but generally not crack resistant. However, all three Stone-Matrix Asphalt (SMA) mixtures were both rut and crack resistant. These observations are consistent with the past experience and field performance.

The balanced design procedure proposed in this project recommends minor changes to TxDOT's current mixture design procedure. Seven mixtures including dense-graded and Superpave mixtures were used to verify and demonstrate this balanced mixture design approach. It was found that a balanced HMA mixture could always be designed providing the aggregates used were not highly absorptive. Statistic analyses on the OT results showed that Performance-Grade (PG) of asphalt binder, effective asphalt content in volume (VBE), film thickness (FT), and surface area (SA) had significant impact on crack resistance of mixtures. Note that the influence of asphalt absorption by aggregates was included in the VBE and FT. The influence of air void content was not significant on crack resistance. Similarly, statistic analyses indicated that the following factors had significant influence on rutting resistance: 1) PG, 2) voids in the mineral aggregate, 3) FT, 4) SA, and 5) air void content.

Additionally, the minimum and maximum asphalt contents for different mixtures to pass the cracking and rutting criteria were recommended based on extensive laboratory testing results. The recommended values were preliminarily verified by field performance data from IH20, WesTrack, and National Center for Asphalt Technology test track. Furthermore, a simplified version of the balanced HMA mixture design procedure was also proposed. Instead of volumetric design, trial asphalt contents for different mixtures were recommended for performance evaluation in the simplified mixture design procedure. The procedure was verified in two case studies.

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INTEGRATED ASPHALT (OVERLAY) MIXTURE DESIGN, BALANCING RUTTING AND CRACKING REQUIREMENTS

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LIST OF ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
ANCOVA	Analysis of Covariance
ANOVA	Analysis of Variance
ASTM	American Standard Test Method
AV	Air Void
CalTrans	California Department of Transportation
DSR	Dynamic Shear Rheometer
DOT	Department of Transportation
EMAC	Estimated Minimum Asphalt Content
FT	Film Thickness
HMA	Hot-Mix Asphalt
HWTT	Hamburg Wheel Tracking Test
IDT	Indirect Tension Test
MEPDG	Mechanistic-Empirical Pavement Design Guide
N_{design}	Number of Gyrations for HMA Mixture Design
NCAT	National Center for Asphalt Technology
NCHRP	National Cooperative Highway Research Program
OAC	Optimum Asphalt Content
OT	Overlay Tester
PG	Performance Grade
RTFO	Rolling Thin Film Oven
SA	Surface Area
SGC	Superpave Gyratory Compactor
SHRP	Strategic Highway Research Program
SMA	Stone-Matrix Asphalt
SPT	Simple Performance Tests
SST	Simple Shear Test
TGC	Texas Gyratory Compactor
TxDOT	Texas Department of Transportation
TTI	Texas Transportation Institute

LIST OF ABBREVIATIONS (CONT'D)

VBE	Effective Asphalt Content in Volume
VFA	Void Filled with Asphalt, percent of VMA
VMA	Voids in Mineral Aggregates

CHAPTER 1

INTRODUCTION

Construction of a hot-mix asphalt (HMA) overlay is the most common method used by the Texas Department of Transportation (TxDOT) to rehabilitate existing asphalt and concrete pavements. Selecting the appropriate combination of aggregates and binder types is an important decision that TxDOT engineers make on a routine basis. However, this selection is a difficult balancing process, because for an HMA (overlay) mixture to perform well it must have a balance of both adequate rut and crack resistance performance. In fact, the goal of balancing HMA design has been pursued for a long time by various researchers and practitioners, but without much success (*1, 2, 3, 4*). One of the main reasons is the lack of performance-related tests and associated criteria for evaluating both rutting and cracking resistance. In the past, HMA designs, such as the Hubbard-Field and Marshall methods, focused on rutting resistance only. Since the early 1990s, the Superpave volumetric design method has been implemented by many state Departments of Transportation (DOTs). Three major problems have been reported with Superpave mixtures. One problem is that the Superpave mixtures are too dry and often associated with cracking distresses such as reflective and top-down cracking (*5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15*). The second problem is high permeability of the Superpave mixtures (*8, 16, 17, 18, 19, 20, 21*). Third, there is a problem of compactability in the field (*8, 22*). Low compactability is highly related to high air voids (AV) and permeability, and subsequently poor durability. In Texas, with the implementation of the Hamburg Wheel Tracking Test (HWTT), TxDOT now has the means to screen out mixtures that are susceptible to rutting and moisture damage. With regard to cracking, in an earlier TxDOT project (0-4467) the Texas Transportation Institute (TTI) demonstrated the value of the upgraded Overlay Tester (OT) for characterizing the cracking resistance of HMA mixtures (*5*). This study seeks to integrate the HWTT and the OT to develop a balanced HMA mixture design procedure.

1.1 OBJECTIVES

The overall objectives for the first year of Project 0-5123 were to:

1) Propose a balanced HMA mixture design procedure incorporating both rutting and cracking requirements.
2) Demonstrate the impact of using the new procedure on HMA mixes currently designed in Texas.

1.2 REPORT ORGANIZATION

This report is organized into eight chapters. A brief introduction is presented in Chapter 1. Chapter 2 provides the literature review on HMA mixture properties, laboratory testing methods, and mixture design methods. Chapter 3 describes the research approach. A methodology for integrating the OT into current TxDOT HMA mixture design process is proposed. Eleven mixtures commonly used in Texas are used to demonstrate this methodology in Chapter 4. A balanced HMA mixture design procedure is recommended in Chapter 5. Seven HMA mixtures were designed following the proposed procedure. Chapter 6 investigates the lower and upper limits of the asphalt content within which HMA mixtures can pass both the rutting and cracking criteria. On the basis of these two limits, trial asphalt contents were recommended. In Chapter 7, a simplified version of the balanced HMA mixture design procedure is proposed. Guidelines for each component of the simplified mixture design procedure are also provided. Finally, Chapter 8 presents a summary of conclusions and recommendations from this project.

CHAPTER 2

LITERATURE REVIEW ON HMA MIXTURE DESIGN

Crawford traced the history of HMA mixture design dating back to the 1860s (*23*). Since the 1860s, different HMA mixture design methods have been developed at different places around the world. In this chapter, the significant mixture properties that relate to HMA concrete pavement performance/distresses and the associated laboratory test methods are discussed before the details of HMA design methods are presented. Several summary thoughts are provided at the end of this chapter.

2.1 SIGNIFICANT HMA MIXTURE PROPERTIES AND ASSOCIATED LABORATORY TESTS

Five major modes of distress generally considered in the design of HMA concrete (overlay) pavements are fatigue cracking, permanent deformation (rutting), reflective cracking, thermal cracking, and moisture damage. Minimizing these distresses requires consideration of a number of mixture properties. These mixture properties include:

- mixture stiffness;
- resistance to permanent deformation (rutting);
- cracking resistance (fatigue cracking, low-temperature cracking, reflective cracking, and top-down cracking);
- durability including aging hardening, moisture sensitivity, and permeability; and
- workability including compactability during the construction process.

Additionally, skid resistance is also a mixture property that needs to be considered in the design of surface mixtures. Skid resistance is not only related to the polishing characteristics of the aggregate but also to surface drainage conditions and the mixture's macro-texture.

2.1.1 Mixture Stiffness

The stiffness characteristics of HMA mixtures depend on time of loading and temperature. Mixture stiffness is typically represented by the following equation:

$$S_{mix}(t,T) = \frac{\sigma}{\varepsilon} \quad (1)$$

where:

$S_{mix}(t, T)$ = mixture stiffness at a particular time of loading, t, and temperature, T; and

σ, ε = applied stress and resultant strain, respectively.

The stiffness characteristic is needed to define the performance of an HMA mixture in a specific structure since it is required to determine the stresses and deformations in the HMA concrete layer due to loading and environmental effects. In pavement structures, mixture stiffness also influences the stresses and deformations of the other component layers (base, subbase, and subgrade), which, in turn, influence the performance of the HMA concrete layer. The stiffness characteristic is, for any pavement structural analysis and performance prediction, a critical parameter. Mixture stiffness is required to estimate the potential for fatigue cracking, rutting, low-temperature cracking, and reflective cracking.

Mixture stiffness, as shown in Table 1, can be measured in different loading modes, such as axial dynamic load test. However, there is general agreement that the stiffness of an HMA mixture is best characterized by dynamic modulus measurements made over a range of temperatures and loading frequencies, because dynamic modulus has the potential to simultaneously characterize the HMA visco-elastic property as a function of both loading time and temperature. Standard test methods (listed below) have been published by American Association of State Highway and Transportation Officials (AASHTO) and American Standard Test Method (ASTM). Recently, a hollow cylinder test for measuring the moduli of HMA mixtures was also introduced (*24*).

AASHTO TP62-03: Standard test method for determining the dynamic modulus of hot-mix asphalt concrete mixtures.
AASHTO T-320: Determining the permanent shear strain and stiffness of asphalt mixtures using the Superpave Shear Test (SST).
ASTM D 3497: Standard test method for dynamic modulus of asphalt mixtures.

Table 1. Methods for Measuring the Stiffness of HMA Mixtures.

Mode of loading	Form of load application	Stiffness measure	Sample geometry	Gauge length	Test conditions	Reference number
Axial (normal stress): compression	Creep	Compliance, creep modulus	D≥70 mm H/D≥1.5	=D	Friction reducing and treatment required	25
	Dynamic	Complex (dynamic) modulus				
	Repeated load	Resilient modulus				
Axial (normal stress): tension	Creep	Compliance, creep modulus	D≥75 mm H/D≥2.0	75 mm	Glued ends	26
	Dynamic	Complex (dynamic) modulus				
	Repeated load	Resilient modulus				
Shear (shear stress)	Creep	Compliance, creep modulus	D=150 mm H=50 mm or 38 mm	H	Glued ends	AASHTO T-320
	Dynamic	Complex (dynamic) modulus				
	Repeated load	Resilient modulus				
Diametral (indirect tensile stress)	Creep	Compliance, creep modulus	D=150 mm H=38 mm	50 mm	Steel loading strips, specimen loading frame	27
	Dynamic	Complex (dynamic) modulus				
	Repeated load	Resilient modulus				
Flexure	Dynamic	Complex (dynamic) modulus				28
	Repeated load	Resilient modulus				
Hollow cylinder	Dynamic	Complex (dynamic) modulus	150 mm outside 106 mm inside	50 mm	Internal pressure from pressure intensifier	24

Note: D = diameter, H = height

2.1.2 Resistance to Permanent Deformation (Rutting) − Stability

HMA mixtures need to resist rutting (accumulation of permanent deformation) under high tire contact pressures and large numbers of load repetitions. Rutting is caused by a combination of densification (decrease in volume and AV) and shear deformation (equal volume movement and increase in AV). For well-compacted HMA concrete pavements, past research (*29, 30*) indicated that shear deformation rather than densification is the primary rutting mechanism. Resistance to permanent deformation or shearing stress has been defined as a stability-related phenomenon. Because HMA mixtures must be designed with adequate stability to ensure adequate performance, stability is considered to be the core aspect of HMA mixture design with respect to rutting.

Stability is affected by type/grade and amount of asphalt binder, aggregate properties (such as absorption, texture, and shape of particle), gradation, compaction level, and temperature. Higher stability is promoted by using hard aggregates with rough surface textures, dense gradations, comparatively low asphalt binder contents, harder (stiffer) asphalts, and well-compacted mixtures as long as the air voids do not fall below a certain level.

At least three laboratory tests: Hubbard-Field (*31*), Marshall (*32*), and Hveem (*32*) tests, have been developed to characterize the stability of HMA mixtures. Necessary minimum values for measured stability have been established in different HMA mixture design methods (*31, 32*) to ensure adequate pavement stability. The minimum value established will, of course, depend on the type of stability test, weight and volume of traffic, and other factors such as climatic condition, type of underlying structure, and thickness of surfacing. *Because of the uncertainties of these factors and doubts about how to measure true pavement stability, there is, quite often, a tendency to design for maximum stability. Sometimes this is done at the detriment of other very important design factors, such as cracking resistance and durability.*

With the renewed interest in HMA mixture design generated by the Strategic Highway Research Program (SHRP), several new laboratory tests have recently been developed to characterize the permanent deformation properties of HMA mixtures. Sousa et al. (*29*) made an excellent review of available permanent deformation tests for HMA mixtures, as shown in Table 2. During the SHRP, the series of performance-based tests listed in Table 3 were also developed (*33*). Christensen et al. (*34*) summarized the latest developments after the SHRP, as described in Table 4.

In summary, permanent deformation tests have evolved from purely empirical tests (Hubbard-Field, Marshall, and Hveem tests) through simulation tests (such as HWTT, Asphalt Pavement Analyzer, and French wheel tracking test) to more fundamental tests (such as the Simple Performance Test [SPT] [*35*]). However, most of these fundamental SPTs, which include dynamic modulus, flow number, and flow time tests, are still under development and/or evaluation and are not yet ready for implementation within routine HMA mixture design procedures. Consequently, simulation tests, such as the HWTT, are considered to be the best option for routine HMA mixture design at the present time.

Table 2. Comparison of Various Test Methods for Permanent Deformation Evaluation (*29*).

Test method	Sample shape	Measured characteristics	Advantages and limitations	Field simulation	Simplicity	Overall ranking
Diametral static (creep)	D= 100 mm H= 62 mm	Creep modulus vs. time Permanent deformation vs. time	Easy to implement Field cores can be easily obtained Shear stress field not uniform State of stress is predominantly tension Equipment is relatively simple in static test For repeated and dynamic tests, the complexity of the equipment is similar to that of triaxial repeated and dynamic equipment	3	1	2
Diametral repeated		Resilient modulus Permanent deformation vs. cycles				
Diametral dynamic		Dynamic modulus Damping ratio Permanent deformation vs. cycles				
Hollow cylindrical	Wall thickness = 25 mm H= 450 mm External D= 225 mm	Dynamic axial modulus Dynamic shear modulus Axial damping ratio Shear damping ratio Axial permanent deformation vs. cycles Shear permanent deformation vs. cycles	Almost all states of stress can be duplicated Capability of determining damping as a function of frequency for different temperatures for shear as well as axial Sample preparation is tedious Expensive equipment Cores cannot be obtained from pavement	1	3	Not suitable for routine use
Simple shear static (creep)	D= 100 mm H= 62 mm	Shear creep modulus vs. time Shear permanent deformation vs. time	Shear stress can be directly applied to the specimen Cores can be easily obtained from existing pavement	2	2	1
Simple shear repeated		Shear resilient modulus Shear permanent deformation vs. cycles	Better expresses traffic conditions			
Simple shear dynamic		Shear dynamic modulus Damping ratio Shear permanent deformation vs. cycles	Capability of determining the damping as a function of frequency for different temperatures Equipment not generally available			

Table 3. Performance Test Included in the Superpave Mixture Analysis System (*33*).

Distress	Test	Device
Permanent deformation	Frequency sweep at constant height	Superpave Shear Tester
	Repeated shear at constant stress ratio	
	Simple shear	
	Uniaxial strain	
	Volumetric	
Fatigue cracking	Frequency sweep at constant height	Superpave Shear Tester
	Tensile strength	Indirect Tensile Tester
Low-temperature cracking	Creep compliance	Indirect Tensile Tester
	Tensile strength	

Table 4. Summary of Post SHRP Permanent Deformation Testing Research (34).

Test	Device	Demonstrated correlation with measured rutting	Criteria	Advantages	Disadvantages
Dynamic shear modulus	Superpave shear tester	High	Preliminary based on sensitivity analysis	Applicable to field cores and lab specimens	Specimen preparation Equipment cost and complexity
Repeated shear constant height	Superpave shear tester	High	Preliminary based on sensitivity analysis	Applicable to field cores and lab specimens Simulates traffic loading Large deformation test	Specimen preparation Equipment cost and complexity Available permanent deformation model not widely accepted
Dynamic modulus	Simple performance test system	High	AASHTO MEPDG	Compatible with the AASHTO MEPDG rutting model Active equipment development	Specimen preparation Cannot test field cores
Flow number	Simple performance test system	High	None	Simulates traffic loading Wide range of stress state possible Large deformation test Active equipment development	Specimen preparation Cannot test field cores
Flow time	Simple performance test system	High	None	Very simple test Wide range of stress state possible Large deformation test Active equipment development	Specimen preparation Cannot test field cores
High-temperature indirect tensile strength plus compaction slope	Superpave Gyratory Compactor plus AASHTO T283 indirect tension test	Potentially high based on correlation with repeated shear constant height test	Preliminary repeated shear constant height criteria	Uses existing equipment Applicable to quality control testing	Requires field verification
Gyratory shear resistance	Superpave Gyratory compactor with shear force capability	Fair	To identify unstable mixtures	Results available after compaction	Requires shear force measurement capability Can only identify mixture instability
Rapid performance	Superpave Gyratory compactor with indenter	Potentially high based on correlation with repeated load test	Preliminary based on sensitivity analysis	Uses existing equipment and low-cost indenter Applicable to quality control testing	Requires field verification
Wheel tracking	Asphalt pavement analyzer	High	Location, facility, and mix specific	Intuitive test	Equipment cost Extensive calibration to establish local criteria
	Hamburg Wheel Tracking Test	High	Yes	Very simple test Applicable to field cores and lab specimens	Equipment cost Extensive calibration to establish local criteria

Note: MEPDG = Mechanistic-Empirical Pavement Design Guide.

2.1.3 Cracking Resistance

Four types of cracking need to be considered in HMA design:

1) fatigue cracking (bottom-up),
2) top-down cracking,
3) low-temperature cracking, and
4) reflective cracking.

In Texas, low-temperature cracking and top-down cracking, when compared to fatigue cracking and reflective cracking, are of secondary concern. Therefore, only fatigue and reflective cracking resistances of HMA mixtures are discussed in detail in this report.

2.1.3.1 Fatigue resistance

Fatigue resistance is the ability of an HMA mixture to bend repeatedly under repeated loading without fracture. Considerable research (*1, 36, 37, 38, 39, 40, 41*) has been devoted to fatigue characterization of HMA mixtures. Response to repetitive loading is typically defined by the relationship in equation 2:

$$N_f = k_1 \left(\frac{1}{\varepsilon_t} \right)^{k_2} \left(\frac{1}{S_{mix}} \right)^{k_3} \tag{2}$$

where:

N_f = fatigue life of HMA mixture;
ε_t = tensile strain at the critical location within the HMA layer;
S_{mix} = HMA mixture stiffness; and
$k_{1\text{-}3}$ = experimentally determined constants.

For HMA mixtures with continuous graded aggregates, two major factors affect fatigue response: asphalt binder content and degree of compaction as measured by the AV content. Fatigue response is proportional to the applied tensile strain at the bottom of the HMA layer, which varies with pavement temperature and load magnitude. It is therefore necessary to use an accumulative damage concept. A reasonable hypothesis is the linear summation of load cycle ratios using Miner's hypothesis (*38*) as shown in equation 3:

$$\sum_{i=1}^{n} \frac{n_i}{N_i} \le 1 \tag{3}$$

where:

n_i = number of actual traffic load applications at strain level *i* and
N_i = number of allowable traffic load applications to failure at strain level *i*.

Significant efforts have been made to characterize fatigue response of HMA mixtures in the laboratory since the late 1950s. Tangella et al. (*42*) summarized laboratory tests used to characterize the fatigue properties of HMA mixtures before the SHRP, as presented in Table 5. During SHRP, Monismith and his associates (*40*) chose the bending beam fatigue test for characterizing the fatigue properties of HMA mixtures, after a comprehensive comparison among bending beam fatigue tests, flexural cantilever tests, and repeated diametral tests. After the SHRP was completed, Monismith and his associates (*43*) continued the bending beam fatigue test, focusing on the fatigue properties of HMA mixtures at high temperatures. They developed a recursive fatigue cracking model based on the Weibull proportional hazards model (*43*). Major efforts have also been made by Carpenter et al. (*44*) and under the current NCHRP 9-38 project (*45*) to investigate the existence of an endurance fatigue limit for HMA mixtures. The concept of an endurance fatigue limit is that an HMA mixture can withstand an infinite number of load repetitions if the strain level applied by traffic load is below the endurance fatigue limit. The endurance fatigue limit is an important concept for perpetual pavement design. However, the latest investigation under Project 0-4822 found that HMA mixture design, permeability, compactability, and debonding may be more critical to performance than other variables (*22*).

In addition to the bending beam fatigue test, alternative methods have been developed for evaluating fatigue cracking resistance. Witczak et al. (*35*) evaluated the dynamic modulus test and Indirect Tension Test (IDT) creep compliance test for characterizing fatigue resistance of HMA mixtures. However, there are not sufficient published data to support the conclusion that fatigue cracking is related to dynamic modulus or creep compliance. Another significant effort in developing a fatigue test is to use continuum damage theory to analyze the fatigue behavior of HMA mixtures (*41, 46, 47, 48, 49*). Additionally, Si (*50*) employed the pseudo-strain energy concept to study the fatigue properties of HMA mixtures with consideration of micro-damage and healing. Most recently, a laboratory test protocol and analytical methodology, which are also based on the pseudo-strain energy concept, have been developed under Project 0-4468 (*51*). This protocol is still under validation and evaluation. Therefore, no pseudo-strain energy-based fatigue model is currently available for routine implementation.

Table 5. Comparison of Test Methods for Cracking (42).

Method	Application of test results	Advantages	Disadvantages and limitations	Simulation of field conditions	Simplicity	Overall ranking
Repeated flexural test	Yes σ_b or ε_b ,S_{mix}	1. Well known, widespread. 2. Basic technique can be used for different concepts. 3. Results can be used directly in design. 4. Options of controlled stress or strain.	1. Costly, time consuming. 2. Specialized equipment needed.	4	4	I
Direct tension test	Yes (through correction) σ_b or ε_b ,S_{mix}	1. Need for conducting fatigue tests is eliminated. 2. Correlations exist with fatigue test results.	1. The correlations based on one million repetitions. 2. Temperature only at 10ºC. 3. Use of EQI (thickness of bituminous layer) for one million repetitions only.	9	1	I
Diametral repeated load test	Yes $4\sigma_b$ and S_{mix}	1. Simple in nature. 2. Same equipment can be used for other tests. 3. Tool to predict cracking.	1. Biaxial stress state. 2. Underestimates fatigue life.	6	2	II
Dissipated energy method	Φ, ψ, S_{mix} and σ_b or ε_b	1. Based on a physical phenomenon. 2. Unique relation between dissipated energy and N.	1. Accurate prediction requires extensive fatigue test data. 2. Simplified procedures provide only a general indication of the magnitude of the fatigue life.	5	5	III
Fracture mechanics tests-repeated tension	Yes K_I, S_{mix} curve (a/h - N); calibration function (also K_{II})	1. Strong theory for low temperature. 2. In principle the need for conducting fatigue tests eliminated.	1. At high temp., K_I is not a material constant. 2. Large amount of experimental data needed. 3. Only stable crack propagation is accounted for.	7	8	IV
Repeated tension or tension and compression test	Yes σ_b or ε_b ,S_{mix}	1. Need for flexural fatigue tests eliminated.	1. Compared to direct tension test, this is time consuming, costly, and special equipment required.	8	3	
Triaxial repeated tension and compression test	Yes σ_d, σ_c, S_{mix}	1. Relatively better simulation of field conditions.	1. Costly, and special equipment required. 2. Imposition of shear strains required.	2	6	
Repeated flexure test on elastic foundation	Yes σ_b or ε_b ,S_{mix}	1. Relatively better simulation of field conditions. 2. Tests can be conducted at higher temperatures since specimens are fully supported.	1. Costly, and special equipment required.	3	7	
Wheel track test (laboratory)	Yes σ_b or ε_b	1. Good simulation of field conditions.	1. For low S_{mix} fatigue is affected by rutting due to lack of lateral wandering effects. 2. Special equipment required.	1	9	
Wheel track test (field)	Yes σ_b or ε_b	1. Direct determination of fatigue response under actual wheel loads.	1. Expensive, time consuming. 2. Relatively few materials can be evaluated at one time. 3. Special equipment required.	1	10	

2.1.3.2 Reflective cracking resistance

Compared to fatigue cracking, there have been relatively few studies on reflective cracking. However, this issue is often critical for HMA overlay performance. In contrast to new flexible pavement applications, HMA overlays are relatively thin and frequently placed over a damaged structure. Both surface rutting and reflective cracking must be considered during the HMA overlay design process. To resist rutting, stiffer layers in terms of the elastic moduli are desired. In fact, most design options used to improve rut resistance. However, the same design options often reduce reflective cracking resistance. Developing an HMA overlay mixture design method that balances both rutting and reflective cracking resistance requirements is crucial to ensuring adequate performance of an HMA overlay. This balance of requirements is one of the fundamental objectives of this research project.

Since the late 1970s, TTI has used the OT to evaluate reflective cracking resistance of HMA concrete mixtures (*52, 53, 54, 55*). Recently, Zhou and Scullion (*5*) upgraded the OT system and established preliminary pass/fail criteria for reflective cracking resistance of HMA mixtures. The OT characterizes both fracture toughness and crack propagation, and is a simple, rapid performance test for reflective cracking evaluation, which can be implemented as a routine test for HMA mixture design. In several recently completed TxDOT studies the OT has been successfully used to design HMA overlays in the Houston, Atlanta, Fort Worth, and Wichita Falls Districts.

In summary, significant effort and development have been made in evaluating fatigue cracking resistance. Both flexural beam fatigue tests and advanced pseudo-strain energy-based fatigue tests have been used to characterize fatigue properties of HMA mixtures. In Florida, the IDT strength test and the dissipated creep strain energy concept were used to evaluate top-down cracking. The dissipated creep strain energy may, at least in theory, be related to fracture toughness. For reflective cracking, TTI developed the OT and preliminary pass/fail criteria. In contrast to other tests, the OT is a fundamental, rapid, and simple performance-related test that characterizes both fracture toughness (first cycle) and crack propagation (following cycles). Poor-performing HMA mixtures can be identified within minutes because they often fail at a relatively small number of load cycles. Therefore, the OT was selected to characterize cracking resistance in this project.

It should also be noted that fatigue cracking and reflective cracking are also highly related to pavement structural thickness. It is necessary to integrate both HMA design and structural thickness design together to obtain an optimized asphalt overlay mixture and asphalt overlaid pavement structure that will perform adequately in the field. Thickness design issues will be studied in Year 2 of this project.

2.1.4 Durability − Age Hardening and Permeability

Durability can be considered to be the resistance of an HMA mixture to environmental effects and to the abrasive action of traffic. HMA pavements in contact with air may be affected by oxidation, volatilization, or both, causing aging and hardening of the asphalt binder. Also, for an HMA mixture to be durable, it must resist stripping of the asphalt binder from the aggregates caused by the action of water. Note that the water action is accelerated under the action of traffic loading. Since moisture damage can cause premature failure of HMA pavements, it is discussed separately in the next section.

2.1.4.1 Age hardening and film thickness

Several researchers have recommended the use of binder film thickness (FT) as a parameter to evaluate HMA mixture durability. Campen and his associates *(56, 57)* found that with increasing aggregate surface area, additional asphalt binder is needed to create a durable HMA mixture, and that the voids in mineral aggregates (VMA) are largely independent of the aggregate surface area. The recommended binder FT for optimum fatigue resistance and durability was proposed to be between 6 and 8 microns *(57)*. Other researchers have also linked asphalt binder FT to the durability of HMA mixtures. Goode and Lufsey *(58)* used the concept of bitumen index (pounds of binder per square foot of aggregate surface area) rather than asphalt binder FT. It should be noted that the bitumen index is interchangeable with asphalt binder FT. Goode and Lufsey recommended a minimum bitumen index (0.00123) to ensure durability, which, according to Kandhal et al. *(59)* on review of Superpave VMA requirements, corresponds to a minimum FT of 6 microns. Stephens and Santosa *(60)* published research on asphalt binder age versus AV content for several Marshall-type HMA mixtures. They found a moderately strong relationship between age hardening and AV content, and also recommended that in-place AV be limited to a maximum of 9 percent to prevent premature age hardening in dense-graded HMA mixtures. In 1996, Kandhal and Chakraborty *(61)* demonstrated again that age hardening increased with decreasing asphalt binder FT. They suggested that a minimum asphalt binder FT of 9 to 10 microns would prevent premature aging in Superpave mixtures. Generally, thinner binder films are more susceptible to oxidation and consequently display poor durability properties compared to thicker binder films, due to ease of air infiltration in the compacted mixtures.

The thickness of the asphalt binder film around a particular aggregate is a function of the diameter of the aggregate and the percent asphalt binder in the mixture. Most often, asphalt binder FT is calculated rather than measured. The current technique for calculating FT is based on the surface area factors recommended by Hveem *(32)*. Asphalt binder FT is normally calculated using equation 4 *(62)*:

$$FT = \frac{V_{asp}}{SA \times W} \tag{4}$$

where:

FT = average film thickness;
V_{asp} = effective volume of asphalt binder;
SA = surface area of the aggregates, estimated based on aggregate gradation and surface area factors proposed by Hveem *(32)*; and

W = weight of aggregate.

It is also well known that calculated FT is substantially influenced by the amount of fine aggregates, specifically the fines passing the No. 200 sieve, because fine aggregates have much larger surface area factors. Furthermore, FT calculated in Equation 4 is an average value. Different sizes of aggregates may have variable asphalt binder FT around them. In order to consider the FT distribution among an HMA mixture, Masad and his research partners (*63, 64*) employed digital imaging techniques to directly measure the FT distribution. The approach, used to measure FT on two-dimensional images of an HMA mixture, is described as follows:

1) A binary image of the internal structure of an HMA mixture is captured at a certain resolution. Aggregates are represented in white color, while the asphalt domain appears in black (Image A, Figure 1).
2) An image of straight lines is created (Image B, Figure 2).
3) The two images (A and B) are combined using the logic operator (AND). Using this operator, images are compared and points that have the same color at the same location are retained on the resulting image (Image C, Figure 3). Therefore, the resulting image will contain straight lines with lengths that are equal to the asphalt FT in Image A.
4) Image analysis is used to measure the FT in Image C. The distribution of different FT is plotted in terms of the total area and the area percent for each FT.

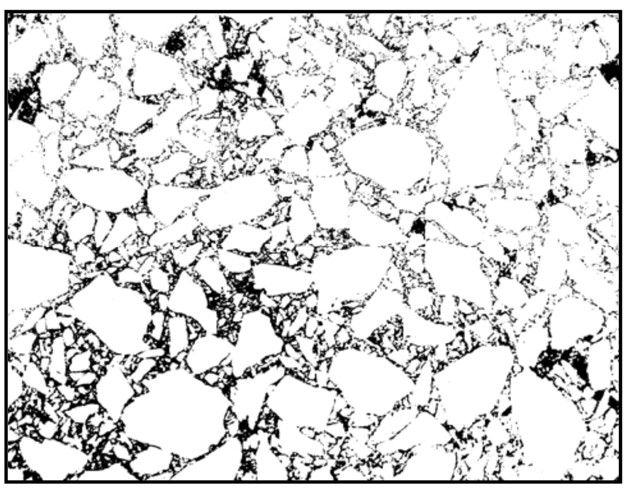


Figure 1. Image A: Binary Image of the Internal Structure of Asphalt Concrete (Black: Asphalt Film, White: Aggregates) (*64*).

Figure 2. Image B: Binary Image of Straight Lines (*64*).

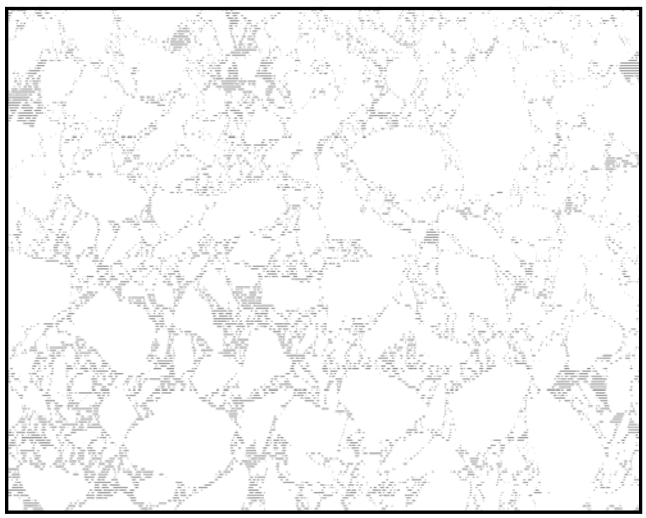


Figure 3. Image C: Result of Combining the Image of the Internal Structure with the Image of Straight Lines (*64*).

The above procedure was used to estimate the asphalt FT distribution in a Wisconsin mixture at three different resolutions (*64*). The analysis results are presented in Figures 4, 5, and 6. The results indicate that the dominant asphalt binder FT in the images is in the range of 350 to 700 microns, contradictory to the theoretical estimation of FT in the range of a few microns. It is postulated that the difference between the image analysis measurements of FT and the theoretical estimation is due to the absence of the majority of small particles from the image. Those small particles fill the area between the large particles and yield thinner asphalt FT in the HMA mixture than the one calculated using image analysis. It also can be noted that the asphalt FT distribution depends upon the selected image resolution. For the purpose of comparison among different HMA mixtures, a resolution should be fixed. This same approach will be used later to determine the asphalt binder FT of three HMA mixtures: one mixture that performed well in the OT and others that did poorly. Additional discussion on the FT is presented in Chapter 6 of this report.

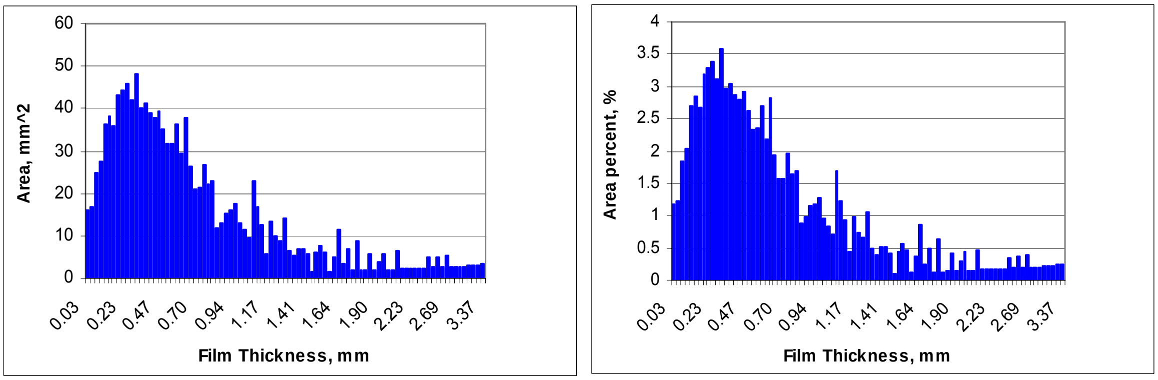


Figure 4. Distribution of Asphalt Binder FT at Resolution = 29.3 microns/pixel (*64*).

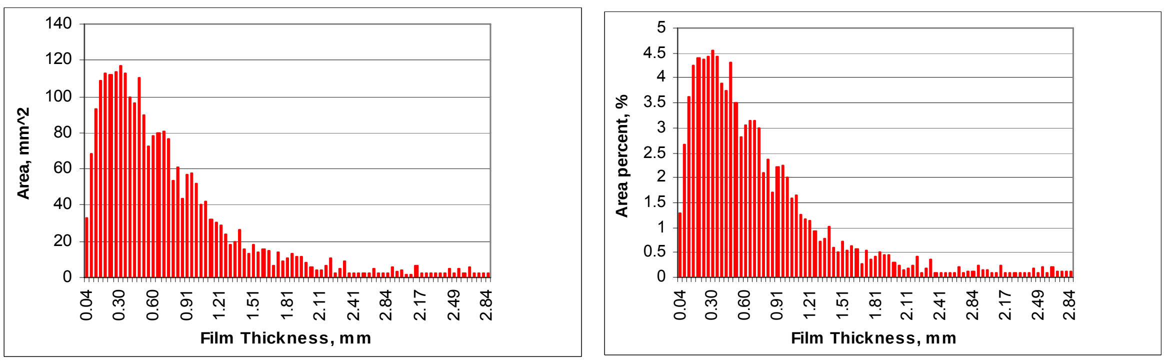


Figure 5. Distribution of Asphalt Binder FT at Resolution = 43.1 microns/pixel (*64*).

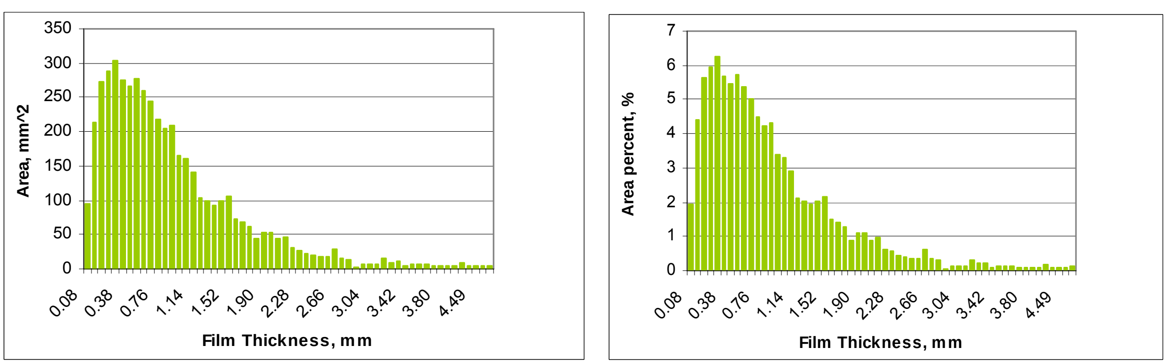


Figure 6. Distribution of Asphalt Binder FT at Resolution = 76.0 microns/pixel (*64*).

2.1.4.2 Permeability

Since the implementation of Superpave mixtures, permeability has become a significant issue for HMA design, especially for coarse Superpave mixtures. The permeability issue has been recently discussed by many researchers and pavement engineers (*16, 17, 18, 19, 20, 21*). Permeability is also related to age hardening and stripping; the more air and water entering an HMA layer, the greater the potential for age hardening and moisture damage. In 1998, Choubane et al. studied the permeability of Superpave wearing course mixtures in Florida (*16*). They found that coarse-graded mixtures designed according to the Superpave volumetric design exhibited significantly higher permeability than fine-graded mixtures designed according to the Marshall system. They also recommended that Superpave mixtures must be compacted to a minimum of 94 percent of maximum theoretical specific gravity, representing a maximum AV content of 6 percent, to ensure low permeability and good durability. *In fact, the permeability issue is closely related to compactability of HMA mixtures. The current trend of using coarse and dry Superpave mixes significantly reduces compactability and increases the potential of permeability problems.*

The most recent studies on permeability of HMA mixtures were undertaken by the NCHRP 9-25/31 team (*65, 66*). They found that permeability is a function of AV content and aggregate surface area. Permeability decreases with decreasing AV content and increasing aggregate surface area. Another finding from NCHRP 9-25/31 is that permeability values of laboratory molded specimens are significantly lower than those of field cores, and tend to be variable. Therefore, they recommended that "for the purpose of HMA mixture design and mixture selection, it is probably more practical to rely upon models for estimating permeability, rather than measuring permeability in the laboratory." Based on the field permeability data published by Choubane et al., Florida DOT (*16*), NCHRP 9-25/31 recommended the following model to predict the HMA mixture permeability:

$$k = 0.00108(V - 1.53S_a + 1.87) \tag{5}$$

where:

- k = coefficient of permeability, in cm/s;
- V = in-place AV content, percent; and
- S_a = aggregate specific surface (surface area), m^2/kg.

Generally, durability can be improved by: 1) increasing asphalt contents, 2) using dense aggregate gradations, and 3) ensuring adequate compaction. All of these factors interactively contribute to ensure that HMA mixtures will be impervious to air, water, and water vapor. In general, an impervious or low-permeability mixture is desired for optimum durability.

2.1.5 Moisture Sensitivity/Damage

Moisture sensitivity/damage can be defined as the loss of strength and durability in HMA mixtures due to the effects of moisture. Moisture damage has been a major concern to pavement engineers for many years. Moisture-related problems are due to or are accelerated by either adhesive failure (stripping of the asphalt film from the aggregate surface) or cohesion failure (breaking within the asphalt mastic). These mechanisms can be associated with the aggregate, the binder, or the interaction between the binder and aggregate. Moisture-related distresses such as stripping and raveling are also accelerated by HMA mixture design or construction issues, including those listed in Table 6 (*67*).

Table 6. Factors That Can Contribute to Moisture-Related Distress (*67*).

MIX DESIGN	• Binder and aggregate chemistry • Binder content • Air voids • Additives
PRODUCTION	• Percent aggregate coating and quality of passing the No. 200 sieve • Temperature at plant • Excess aggregate moisture content • Presence of clay
CONSTRUCTION	• Compaction—high in-place air voids • Permeability—high values • Mix segregation • Changes from mix design to field production (field variability)
CLIMATE	• High-rainfall areas • Freeze–thaw cycles • Desert issues (steam stripping)
OTHER FACTORS	• Surface drainage • Subsurface drainage • Rehab strategies—chip seals over marginal HMA materials • High truck ADTs

Solaimanian et al. (*68*) summarized the existing laboratory tests and those under development for characterizing moisture sensitivity of HMA mixtures. The tests can be classified into two major categories:

- tests on loose HMA mixtures, and
- tests on compacted HMA mixtures.

The static immersion test and the boil test, both conducted on loose HMA mixtures, were among the first tests introduced to the paving industry. These were followed by the immersion–compression test in the late 1940s. This test was conducted on compacted HMA specimens and was the first test to become an ASTM standard in the mid-1950s. Research in the 1960s brought considerable awareness to pavement engineers of the significant effects of climate and traffic on moisture damage. Extensive work by Lottman (*69, 70*) resulted in the laboratory test (IDT strength ratio test) that currently has the widest acceptance in the paving industry. This test was further modified through the work of Tunnicliff and Root (*71*). Wheel tracking of HMA mixtures submerged under water gained popularity for determination of moisture damage in the 1990s. The HWTT is one of these wheel-tracking tests. The Colorado DOT has performed extensive research evaluating HMA mixtures with the HWTT. Aschenbrener et al. (*72, 73, 74, 75*) evaluated factors that influence the results from the HWTT. They found an excellent correlation between stripping observed in laboratory tests and moisture damage of pavements with known field performance. They also noted an excellent correlation between stripping inflection point and known stripping performance. Stuart and Izzo (*76*) worked on finding a correlation between binder stiffness and rutting susceptibility using the HWTT. They found that stiffer binders provided mixtures with lower rutting susceptibility. Izzo and Tahmoressi (*77*) discussed repeatability of the HWTT. Currently, TxDOT has adopted the HWTT as a screening tool for rutting and moisture susceptibility of HMA mixtures (*78, 79*).

2.1.6 Workability

For easy placement in uniform layers with sufficient densification, HMA mixtures must be workable at the desired placement temperature. The term *workability* has been used to describe several properties related to the HMA mixture construction. Workability in the field can be defined as a property that describes the ease with which an HMA mixture can be placed, worked by hand, and compacted (*80*). This definition provides a term that is applicable to movement of HMA mixture through equipment to the roadway, handwork of HMA mixture, and compactibility on the roadway. Satisfactory workability is important in obtaining the desired HMA smoothness and density within a compacted pavement. For harsh HMA mixtures that normally have low workability, it can be very difficult to compact and to construct smooth pavements. These high in-place AV mixtures may experience significant performance problems directly attributable to high AV, such as permeability and associated moisture damage problems, and oxidative aging of the binder that can considerably reduce the pavement life.

With the implementation of Superpave volumetric mixture design, HMA mixtures have become coarser and dryer. The HMA mixtures designed according to the Superpave volumetric mixture design method are both rut resistant and compaction resistant. Additionally, more stiff polymer-modified binders are being used in construction of HMA pavements. All of the above factors make current HMA mixtures less workable. If the compositional properties of an HMA mixture, such as aggregate physical properties and gradation, are kept constant, the workability of the HMA mixture is basically a function of binder properties at a given temperature. The higher the temperature, the better the workability of the HMA mixture in terms of compaction. This good workability is attainable because the viscosity of the binder decreases as the temperature increases. However, increasing the HMA mixture temperature may result in the following problems (*80*):

- damage to asphalt (heat hardening),
- damage to additives,
- increased fuel consumption, and
- increased smoke and volatile organic compounds (VOC) production.

The first attempt to quantify the workability of HMA mixtures found in the literature was made by Marvillet and Bougault (*81*), who designed a laboratory instrument to measure HMA mixture workability, as shown in Figure 7. The workability meter consists of a chamber connected to a rigid frame, into which the test HMA mixture is introduced, and a speed controller which drives a blade in the HMA mixture (Figure 7). Marvillet and Bougault defined workability as the reciprocal of the resistance moment (torque) produced in the mixture against the rotation of the blade. Thus, as torque increased, workability decreased. The results from Marvillet and Bougault's study can be summarized as follows:

- Workability of HMA mixtures increased as the viscosity of the binder grade decreased.
- Increasing asphalt binder content improved the workability of HMA mixtures. However, this improvement was not proportional to the increased asphalt binder content.
- As the filler content in the HMA mixture increased, the workability decreased.
- Mixtures with angular particles were less workable than mixtures having semi-angular or round aggregate particles.

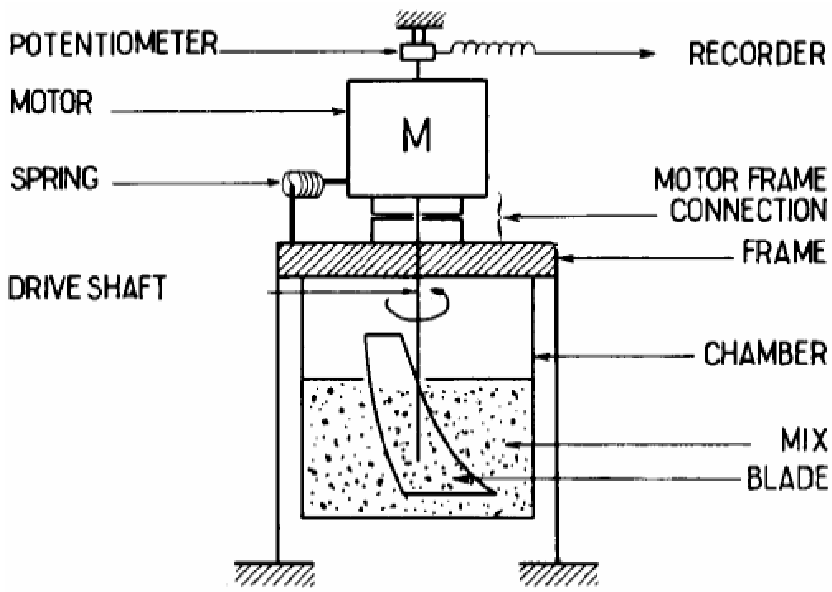


Figure 7. Diagram of Workability Meter (*81*).

The most recent study on workability of HMA mixtures was conducted by Gudimettla et al. (*80*). They developed a similar device (Figure 8) to measure the workability of HMA mixtures and used the same definition of workability of HMA mixtures as that developed by Marvillet and Bougault. Based on these two studies, the following factors are considered to affect the workability of HMA mixtures:

- aggregate type and aggregate properties (such as angularity, crushed faces);
- nominal maximum aggregate size of the gradation;
- asphalt binder Performance Grade (PG) (mixtures using a PG76-22 binder were significantly less workable than mixtures containing an unmodified PG64-22 binder); and
- gradation shape (gradation shape did not have a significant effect on workability; however, there were many two- and three-way interactions that were significant that included gradation shape alone).

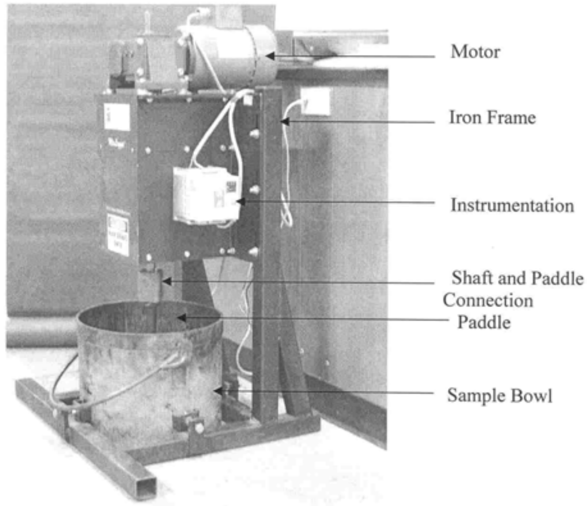


Figure 8. Prototype Workability Device (*80*).

In summary, a well-designed HMA mixture should strike a balance among the performance-related properties of HMA concrete: rut resistance, cracking resistance, durability (age hardening, permeability, and moisture damage), and workability. With implementation of Superpave volumetric mixture design and use of stiffer polymer-modified binders, rutting is no longer a critical issue. Instead, cracking (fatigue cracking, low-temperature cracking, reflective cracking, and top-down cracking), durability (especially permeability), and workability (including compactability) have become the pavement engineers' primary concerns. The research discussed in the remainder of this report focuses on balancing the rutting and cracking resistance of HMA mixes. If the balanced design increases the asphalt content, then both durability and workability of the mixture will improve.

2.2 HMA MIXTURE DESIGN METHODS

The design of HMA mixtures dates to the late 1860s (*23*). Since then, different HMA design methods have been developed. However, the fundamental concepts and principles developed originally have not changed (*82*). As discussed previously, a mixture must be stable to resist traffic loads in the hot seasons. This stability is highly related to the properties of aggregates, such as aggregate size, gradation, shape, and surface texture. Asphalt binder also contributes to stability in the function of a cementing medium. However, too much asphalt binder content reduces stability. The function of asphalt binder is primarily to provide cracking resistance and durability to the mixture. Also, the consistency of the asphalt binder must be neither too brittle in winter nor too soft in summer. Therefore, it is apparent that the design of HMA mixtures generally consists of three basic steps:

1. Select the type and gradation of aggregates.
2. Select the type and grade of asphalt binder, with or without modification.
3. Select the amount of asphalt binder to satisfy the project-specific requirements for HMA mixture properties.

Every step is critical for designing a good HMA mixture. However, the main focus of this project is step 3: "Select the amount of asphalt binder to satisfy the project-specific requirements for HMA mixture properties." Two different concepts have been developed to determine the amount of asphalt binder. Both concepts are discussed in the following sections.

2.2.1 Two Mixture Design Asphalt Content Concepts

Currently, different HMA design methods exist around the world to determine a design asphalt content for a given aggregate and aggregate grading chosen for use in an HMA mixture. However, all these methods are generally based on two basic concepts: *void concept and surface area concept* (*83*). The void concept approach is based on the theory that the amount of asphalt binder required is a function of available space in the compacted aggregate structure. Using this approach, the design asphalt content is that which fills the voids to a degree that still leaves some room for asphalt volume expansion at summer temperatures and for a decrease in space available as the HMA mixture densifies with time under traffic. Most HMA mixture design methods, such as the Hubbard-Field (*31*), Marshall (*32*), and Superpave (*33*), are based on the void concept.

The second concept for the determination of design asphalt content is the surface area theory. This theory is based on the concept that the design asphalt content is that which coats all of the aggregates' surface area with an optimum asphalt FT. Clifford Richardson, considered a great American asphalt technologist, recognized the role of aggregate surface area (*84*). In his book *The Modern Asphalt Pavement* (*84*), published in 1908, he showed that the increased surface area in a fine mixture allows the presence of a larger quantity of bitumen than a coarse mixture. Based on the fact that aggregate size is a primary factor in the relationship between a given weight of aggregate and its surface area, the surface area concept was then employed by the use of empirical formulae based on aggregate grading. The California Highway Department (currently California Department of Transportation [CalTrans]) was the leading state to use the surface area concept (*83*). Today, CalTrans is still using the Hveem method to design its HMA mixtures (*85*). In fact, the Hveem method is representative of the surface area concept. Additionally, current French HMA design procedure also uses the surface area concept to determine the minimum asphalt content (*28*).

It is worth noting that neither the void concept nor the surface area concept alone works well to design asphalt content. One of the lessons learned is that a mechanical test is needed to check potential rutting (*86*). Different mechanical strength tests and performance-related tests have been developed in the past to complement the void or surface area concepts. These tests have been discussed in a previous section.

2.2.2 History of HMA Mixture Design Methods

Most HMA mixture design methods evolved from the fact that HMA mixtures must be stable and durable in service. The stability and durability requirements should be balanced during HMA mixture designs. Table 7 presents the evolution of HMA mixture design methods. The HMA mixture design methods currently used are all empirically based. Up to now, a performance-based HMA mixture design method has not been successfully developed and implemented, although significant efforts were made during the SHRP research to develop a more fundamental and rational HMA mixture design method. After carefully reviewing the HMA mixture design methods widely accepted and used by state DOTs, in the past and even today, one common feature is practicality, in terms of simplicity, speed, and cost. Therefore, the method developed in this research project must be as practical as possible and also be based on laboratory measured properties that are performance related.

Another important issue for HMA mixture design is compaction method in the laboratory. A variety of compaction methods have been employed in order to duplicate the compacted characteristics of the field mixture in the laboratory. These methods (*83*) include:

- direct compression with/without rodding,
- hand tamping,
- impact hammer,
- kneading compactor,
- gyratory compactors,
- vibration, and
- simulated rolling.

Currently, most laboratories in the U.S. use the Superpave Gyratory Compactor (SGC).

Table 7. HMA Mixture Design Methods.

HMA design method	Design asphalt concept	Sample		Mechanical strength test and/or performance-related test			Data interpretation	Criteria	Max. aggregate size	Time	Basic principles
		Sample size	Sample preparation method	Test apparatus	Test temperature	Loading conditions					
Hubbard-Field	Void concept	D: 50 mm H: 25 mm	Hand tamper	Hubbard-Field Stability machine	60 ºC (water bath)	60 mm/min	Plots: 1) Bulk density vs. AC 2) AV vs. AC 3) Stability vs. AC 4) VMA vs. AC	1) Min. stability 2) Range of AV (2-5 %)	19 mm	1920s-1950s	1) Stability controls stability-rutting 2) VMA controls min. asphalt content-durability
Hveem	Surface area concept	D: 100 mm H: 63 mm	Kneading compactor	Hveem stabilometer	60 ºC	0.125 in/min	1) Stability vs. AC 2) Density vs. AC 3) AV vs. AC 4) Cohesiometer vs. AC 5) Swell test results	1) Min. stabilometer value 2) Min. AV: 4 % 3) Swell: max. 0.75 mm	25 mm	1940s-present	1) Surface area controls min. asphalt content-durability 2) Stabilometer control – stability-rutting 3) Swell test controls moisture damage
				Swell test apparatus	25 ºC						
Marshall	Void concept	D: 100 mm H: 63 mm	Marshall hammer	Marshall stabilometer	60 ºC	50 mm/min	1) Density vs. AC 2) Stability vs. AC 3) Flow vs. AC 4) AV vs. AC 5) VMA vs. AC 6) VFA vs. AC	1) Stability 2) Flow 3) AV 4) VMA 5) VFA	25 mm	1940s-present	1) Stabilometer controls stability-rutting 2) VMA controls min. asphalt content-durability
LCPC	Surface area concept	D: 160 mm H: 150 mm	LCPC-gyratory compactor	N/A	N/A	N/A	1) AV vs. Number of gyration to check compactability	1) AV≥10 % 2) 12≤AV≥4 at specified number of gyrations 3) r/R≥0.8 4) RD≤5-10 mm depending on mixtures	25 mm	1960s - present	1) Surface area controls min. asphalt content-durability 2) Gyratory compactor controls compactability
		D: 80 mm	Static compression	Uniaxial loading machine	18 ºC for seven days in dry and in water	1 mm/s	1) R: dry compression strength 2) r: wet compression strength				3) Duriez test controls moisture damage
		500x180x100 mm	LCPC laboratory compactor	LCPC Wheel-tracking rutting test machine	60 ºC for wearing course 50 ºC for base course	P: 0.6 MPa L: 5 kN S: 1 Hz	1) RD@30,000 cycles				4) Wheel tracking test controls rutting
Superpave	Void concept	D: 150 mm H: 115±5 mm	SGC	N/A	N/A	N/A	1) AV vs. AC 2) VMA vs. AC 3) VFA vs. AC 4) AV@N_{ini} 5) AV@N_{max}	1) AV= 4 %, then, check VMA, VFA	37.5 mm	1993 - present	1) No strength test controls stability 2) VMA controls min. asphalt content-durability
TxDOT	Void concept	D: 150 mm H: 115±5 mm	SGC	N/A	N/A	N/A	1) AV vs. AC 2) VMA vs. AC 3) VFA vs. AC 4) AV@N_{ini} 5) AV@N_{max}	1) AV= 4 %, then, check VMA, VFA	37.5 mm	2004 - present	1) HWTT controls stability-rutting 2) HWTT controls moisture damage 3) VMA controls min. asphalt content-durability
		D: 150 mm H: 62 mm	SGC	Hamburg Wheel Tracking Tests	50 ºC	52 passes/min	6) RD vs. number of passes	2) RD ≤12.5 mm			

Note: AC = asphalt content; RD = rut depth; VFA = voids filled with asphalt;

2.3 SUMMARY THOUGHTS

2.3.1 Problems Being Faced in the Field

In the last 15 years, rutting problems have largely been solved or significantly minimized by using mixtures with lower asphalt contents, stiffer polymer-modified binders, and coarse aggregate structures. However, in recent years TxDOT engineers have reported that these measures have resulted in other concerns, namely,

1. increased early cracking,
2. reduced durability, and
3. workability and compactability problems.

In fact, these three problems are highly related. Poorly workable HMA mixtures are difficult to compact in the field, which results in high AV and poor durability (age hardening and permeability) and cracking resistance. In fact, an important component of HMA, asphalt binder content, is closely related to all three of these problems. Adding more asphalt binder into the HMA mixture can significantly improve workability, durability, and cracking resistance *(8)*, but sometimes at the expense of rutting resistance. Therefore, the key to solving these problems is to develop a balanced HMA mixture design procedure.

2.3.2 HMA Mixture Design Balancing Rutting and Cracking Requirements

In research Project 0-4467, the TTI OT was upgraded to a rapid and simple performance test for characterizing cracking resistance of HMA mixtures, and the preliminary pass/fail criterion for cracking resistance was also recommended *(5)*. A mixture with too low (effective) asphalt content will not pass the OT and its associated cracking resistance criterion. Thus, the OT actually determines the minimum asphalt content an HMA mixture needs to avoid premature cracking and durability problems. Meanwhile, the HWTT has been used in HMA mixture design for several years to screen out potentially rutting and moisture damage susceptible mixtures. Generally, too high asphalt binder content causes the mixture to fail the rutting resistance requirement. Therefore, the HWTT actually determines the maximum asphalt binder content an HMA mixture can contain without causing rutting problems in the field. The design asphalt binder content should be selected between the minimum asphalt content for the cracking resistance requirement and the maximum asphalt content for the rutting resistance requirement. Thus, a balanced HMA mixture design can be obtained.

CHAPTER 3
RESEARCH APPROACH

3.1 INTRODUCTION

As discussed in Chapter 1, the major objectives of this research project in this Year 1 report are as follows:

1. To propose a methodology to integrate the OT into current TxDOT HMA mixture design process.
2. To develop a balanced HMA mixture design procedure, and demonstrate its application with typical Texas mixtures.

To achieve these objectives, the research was conducted in three phases:

- Phase I: Integrate the OT into current TxDOT HMA mixture design process.
- Phase II: Propose a balanced HMA mixture design procedure.
- Phase III: Develop guidelines for balancing rutting and cracking requirements.

These three phases are discussed in the subsequent sections.

3.2 PHASE I: INTEGRATE THE OVERLAY TESTER INTO CURRENT TXDOT HMA MIXTURE DESIGN PROCESS

Current TxDOT HMA mixture design procedures are documented in TxDOT 200-F, *Bituminous Test Procedures Manual*: Chapter 6, Tex-204-F, Design of Bituminous Mixtures *(79)*. The design process is summarized as follows:

- Stage 1: Select materials including asphalt binder and aggregates.
- Stage 2: Prepare laboratory-mixed samples: either the SGC or the Texas Gyratory Compactor (TGC), depending on the type of HMA mixtures, is used to mold two samples for each asphalt binder content. Generally, four asphalt contents are selected.
- Stage 3: Determine optimum asphalt content (OAC): The OAC is determined at a target density (typically 96 percent of maximum theoretical density). Then, check VMA (and VFA). If the VMA (and VFA) is not within the allowable range, go back to Stage 1.
- Stage 4: Evaluate mixture properties: Mold two test specimens at the OAC to 93 ± 1 percent density for the HWTT. Run the HWTT according to "Tex-242-F, Hamburg Wheel Tracking Test *(79)*." If the rut depth is not within specification, go back to Stage 1.

Since the OT is proposed to be used to evaluate the crack resistance of HMA mixtures, only Stage 4 needs to be modified in order to integrate the OT into the current process. Thus, the integrated HMA mixture design process is presented as follows.

- Stage 1: The same as the existing Stage 1,
- Stage 2: The same as the existing Stage 2,
- Stage 3: The same as the existing Stage 3, and
- Stage 4: Evaluate mixture properties: Mold four test specimens to 93 ± 1 percent density at OAC: two for the HWTT and two for the OT. Run the HWTT according to *"Tex-242-F, Hamburg Wheel Tracking Test (79)."* Run the OT according to the test protocol recommended in reference 87. Compare the HWTT and the OT results with the rutting and cracking criteria. If either rutting, cracking requirement, or both cannot be met, go back to Stage 1.

More than 10 HMA mixtures will be used to evaluate this integrated HMA mixture design process. It was found that most HMA mixtures designed following current TxDOT HMA mixture design procedure are rut resistant, but generally not crack resistant. A balanced HMA mixture design procedure is thus needed. The Phase I study is documented in Chapter 4 of this report.

3.3 PHASE II: PROPOSE A BALANCED HMA MIXTURE DESIGN PROCEDURE

In general, the OAC determined by current TxDOT HMA mixture design procedure is relatively low, resulting in HMA mixtures with poor crack resistance. It is well known that there are several ways to increase the OAC. The method employed under this research project is based on the objective of minimizing the changes to current design procedure. Variable asphalt contents around the OAC from Stage 3 should be evaluated under the HWTT and the OT. Then, a balanced HMA mixture passing both rutting and cracking resistance criteria can be selected. The enhanced HMA design process is presented as follows:

- Stage 1: The same as the existing Stage 1;
- Stage 2: The same as the existing Stage 2;
- Stage 3: The same as the existing Stage 3; and
- Stage 4: Evaluate mixture properties: Mold four test specimens to 93 ± 1 percent density at each of three asphalt binder contents: OAC, OAC + 0.5 percent, and OAC + 1.0 percent. Note that the interval of these three asphalt binder contents may vary based on binders' PG and types of aggregates. Four test specimens are needed at the OAC: two for the HWTT and two for the OT. Run the HWTT according to *"Tex-242-F, Hamburg Wheel Tracking Test (79)."* Run the OT according to the test protocol recommended in Reference 87.

- Stage 5: Determine a balanced asphalt content: Balance rutting and cracking resistance requirements and determine a balanced asphalt content (Figure 9). If either the rutting or cracking resistance requirement, or both, cannot be met, go back to Stage 1.

Seven case studies are presented in Chapter 5 to demonstrate the balanced HMA mixture design procedure.

Balancing Rutting and Cracking Requirements

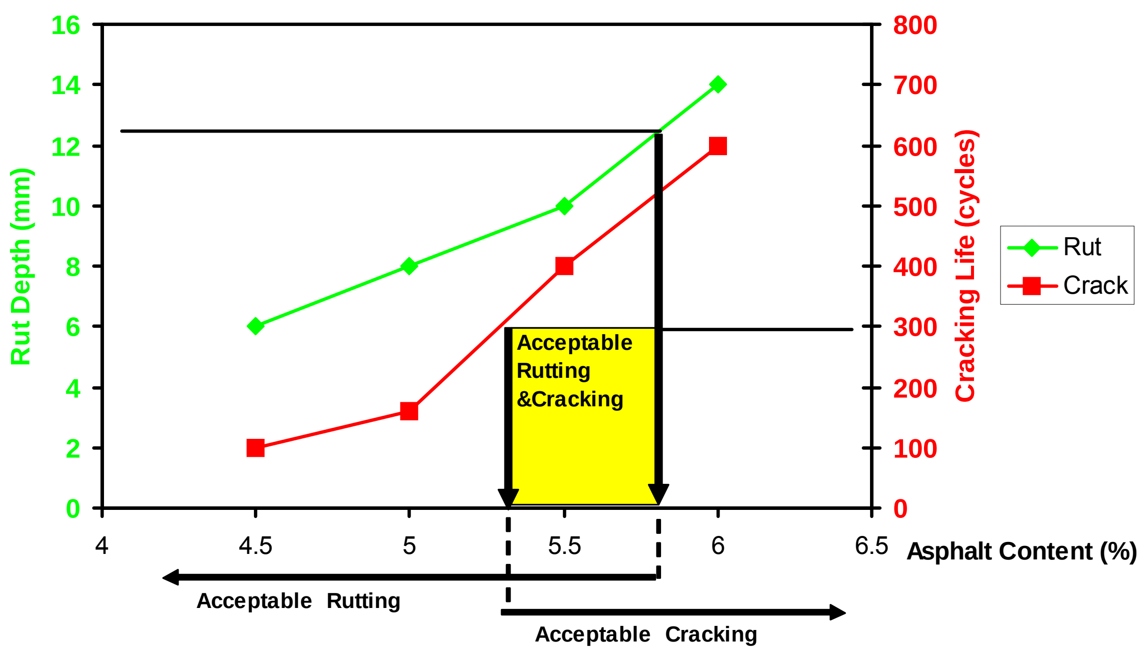


Figure 9. The Balanced Mixture Design Concept.

It is worth noting that there is some overlap between Stages 2 and 3 and Stage 4 in the balanced HMA mixture design procedure. In Stages 2 and 3, mold the specimens using SGC or TGC. Then, determine the OAC based on the volumetric criteria (density and VMA). In Stage 4, vary asphalt binder content around the OAC again, and mold specimens for performance evaluation. The OAC determined from Stages 2 and 3 is used only as a starting trial asphalt content for performance evaluation in Stage 4. Finally, determine the balanced asphalt content based on the rutting and cracking criteria rather than volumetric properties calculated in Stages 2 and 3. Stages 2 and 3 become unnecessary if the trial asphalt contents for Stage 4 are known in advance. It also becomes possible to develop a simplified HMA mixture design procedure balancing rutting and cracking resistance requirements, as shown in Figure 10. Chapter 6 discusses the recommendation of trial asphalt contents and a simplified HMA mixture design procedure, and associated guidelines are presented in Chapter 7.

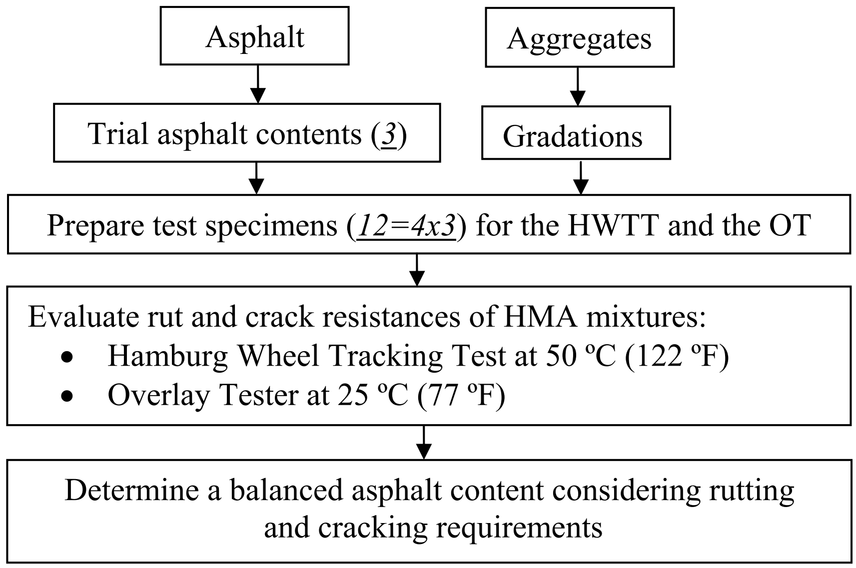


Figure 10. Flow Chart of Simplified HMA Mixture Design Procedure.

3.4 PHASE III: DEVELOP GUIDELINES FOR BALANCING RUTTING AND CRACKING REQUIREMENTS

The results in Phase II will be the foundation of proposed HMA (overlay) mixture design procedure. The guidelines for selecting asphalt binder PG and aggregates will also be proposed. More detailed information is presented in Chapter 7.

CHAPTER 4

ENHANCEMENT OF CURRENT TXDOT HMA MIXTURE DESIGN

4.1 INTRODUCTION

As described previously, the current TxDOT HMA mixture design is a volumetric-based design method. The OAC is first determined based on a target density (typically 96 percent). Then the VMA requirement is checked. Finally, the HWTT is run to check rut resistance of the mixture at the OAC. There is no direct evaluation of reflective crack resistance of the same mixture at the OAC. Currently, the OT is proposed to enhance current HMA mixture design by measuring the crack resistance of the designed mixture. First, this chapter recommends the methodology of integrating the OT into the current TxDOT HMA mixture design process. Then, 11 HMA mixtures are used to demonstrate this enhanced HMA mixture design process.

4.2 ENHANCED HMA MIXTURE DESIGN PROCESS

As discussed in Chapter 3, the best place to integrate the OT into the current HMA mixture design process is at *Stage 4: evaluate mixture properties*. The simplest way for this integration is to mold two additional specimens at the OAC for the OT, as shown in Figure 11. Then, run the OT to check their crack resistance.

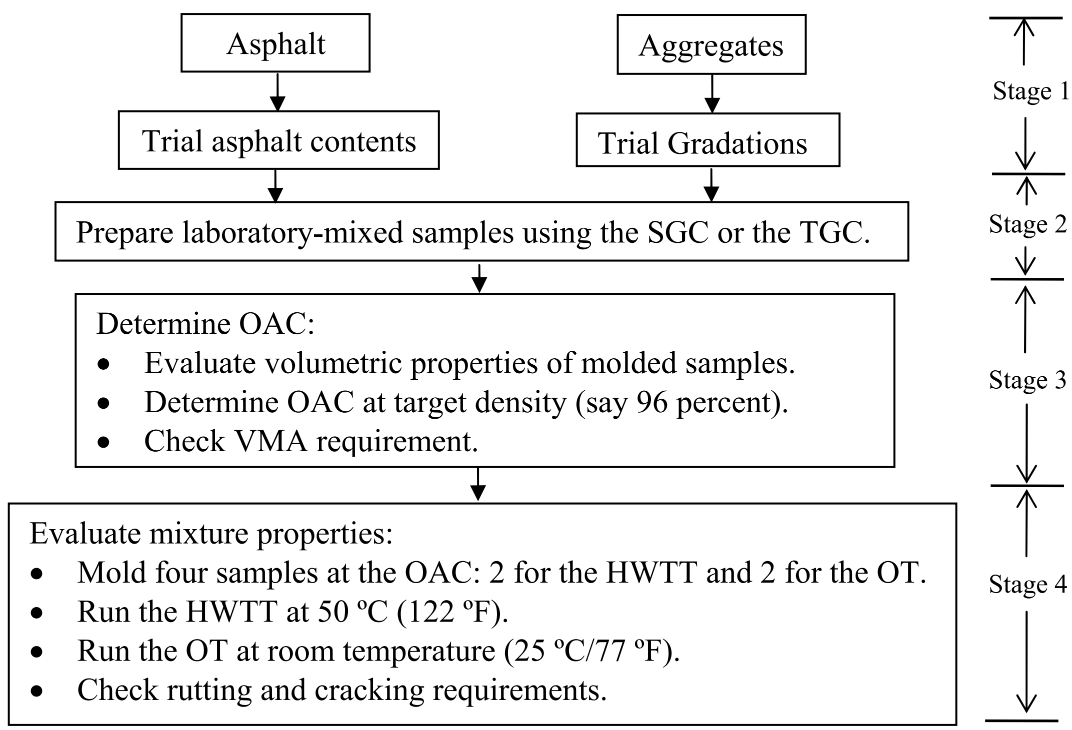


Figure 11. Simple Illustration of Enhanced HMA Mixture Design Process.

Eleven mixtures commonly used in Texas were designed following the above process with two purposes: 1) to demonstrate this integrated HMA mixture design process and 2) to evaluate rutting and cracking resistance of current TxDOT HMA mixtures. Detailed information is presented as follows.

4.3 LABORATORY STUDY ON THE ENHANCED HMA MIXTURE DESIGN PROCESS

The objectives of this laboratory study are to demonstrate the enhanced HMA mixture design process and to evaluate rutting and cracking resistance of TxDOT mixtures. An experimental design was initially conducted, and the design principle was to include as many variables as can be tested within the scheduled work plan in the original project proposal, since many variables (such as asphalt binder PG and type of aggregate) affect HMA mixture properties. For the purpose of demonstration, the design process is presented stage by stage as follows:

4.3.1 Stage 1: Materials Selection and Experimental Design

4.3.1.1 Asphalt binder PG

Three grades of asphalt binder are generally used in Texas: PG64-22, PG70-22, and PG76-22. All three grades were included in this proposed test plan. The Dynamic Shear Rheometer (DSR) was used to characterize dynamic shear modulus of each binder after Rolling Thin Film Oven (RTFO) aging. The test results are listed in Table 8. As can be seen in Table 8, the three asphalt binder PGs were verified.

Additionally, the DSR tests were conducted at 50 ºC, since the test temperature of the HWTT is 50 ºC. The results in Table 8 show that the G*/sin(δ) value of PG76-22 binder at 50 ºC is 1.5 times that of PG70-22 binder and 3 times that of PG64-22 binder. Thus, it is reasonable to expect that the mixtures with PG76-22 binder will have significantly better rut resistance than those mixtures with PG70-22 or PG64-22 binder. Similar conclusions are true for PG70-22 binder and PG64-22 binder.

Table 8. Dynamic Shear Modulus of Each Binder after RTFO Aging.

Temperature (ºC)	PG64-22			PG70-22			PG76-22		
	G* (Pa)	δ (º)	G*/sin(δ) (Pa)	G* (Pa)	δ (º)	G*/sin(δ) (Pa)	G* (Pa)	δ (º)	G*/sin(δ) (Pa)
50	16497.0	75.5	17041.0	34225.0	70.1	36390.0	46400.0	63.7	51772.0
58	5121.7	79.4	5210.1	N/A	N/A	N/A	N/A	N/A	N/A
64	2220.8	81.8	2243.9	5780.7	73.5	6028.3	9606.9	64.8	10618.0
70	1015.1	83.3	1022.0	2871.4	75.0	2972.5	5155.1	65.7	5655.1
76	N/A	N/A	N/A	716.6	75.2	741.2	2813.7	67.6	3042.9
82	N/A	N/A	N/A	N/A	N/A	N/A	1582.8	69.7	1687.5

4.3.1.2 Trial asphalt contents

The trial asphalt contents are usually selected based upon past experience. For most HMA overlay mixtures, the OAC ranges from 4.0 to 6.0 percent. Four trial asphalt contents were selected in this project: 4.0, 4.5, 5.0, and 5.5 percent (by total weight of mixture) for Dense-Graded Type D mixtures molded with PG64-22 binder, and 4.5, 5.0, 5.5, and 6.0 percent for all other mixtures. The reason for this selection is that Dense-Graded Type D mixtures molded with PG64-22 binder generally have relatively poor rutting resistance under the HWTT. Lower trial asphalt contents are thus preferred.

4.3.1.3 Aggregates

In Phase I, six types of aggregates commonly used in Texas were included to study the role of aggregates on rutting and cracking resistance. These aggregates were: 1) TXI limestone, 2) TCS limestone, 3) gravel, 4) sandstone, 5) quartzite, and 6) granite.

The bulk specific gravity and water absorption of each aggregate were measured according to "Tex-201-F, Bulk Specific Gravity and Water Absorption of Aggregate *(79)*." The purpose of measuring the bulk specific gravity is to calculate asphalt absorption by aggregates from analysis of volumetric properties of molded samples. Table 9 presents the bulk specific gravity and water absorption values of the aggregates. Note that laboratory studies on asphalt absorption are relatively few, and no clear definitions of high, intermediate, and low absorptions could be found in the literature. In this research project, absorption is arbitrarily categorized based on the water absorption capacity:

- High absorption: water absorption is larger than 2.0 percent;
- Intermediate absorption: water absorption is between 1.0 and 2.0 percent; and
- Low absorption: water absorption is less than 1.0 percent.

Table 9. Bulk Specific Gravity and Water Absorption of Aggregates.

Type of mixtures	Aggregate	Sources	Bulk specific gravity	Water absorption (%)
Dense-Graded Type D	TXI-limestone	Wichita Falls-US82	2.752	0.7
	TXI-limestone		2.559	0.7
	TCS-limestone	TCS	2.615	2.3
	Sandstone	Houston-US529	2.710	0.6
Superpave-C	SandStone_L	IH20	2.481	2.3
	SandStone_NL		2.482	2.2
	Gravel-1		2.584	0.9
	Quartzite-MD_L		2.628	1.5
SMA-C	Granite	Houston	2.686	0.6
SMA-D	Granite	Dallas-IH635	2.765	0.6
	Granite	Beaumont-US96	2.633	0.7

4.3.1.4 Mixture types and aggregate gradations

The focus of this study is on asphalt overlays. The general overlay mixtures used in Texas are C and D mixtures (12.5 mm [1/2 inch] or 9.5 mm [3/8 inch] NMAS, either Dense-Graded Type C or D, Superpave C or D), or stone-matrix asphalt (SMA) C or D.

Dense-Graded Type D, Superpave C, SMA-C, and SMA-D mixtures were investigated in this project.

In addition, gradations were varied for the same aggregate to investigate the influence of gradation. The same gradation was used for the different aggregates to compare the influence of aggregates on rutting and cracking resistance. Gradations of Dense-Graded Type D mixtures and Superpave C mixtures are shown in Figures 12 and 13, respectively. Figures 14 and 15 show the gradations of SMA-C and SMA-D mixtures, respectively. The detailed aggregate gradations are listed in Appendix A.

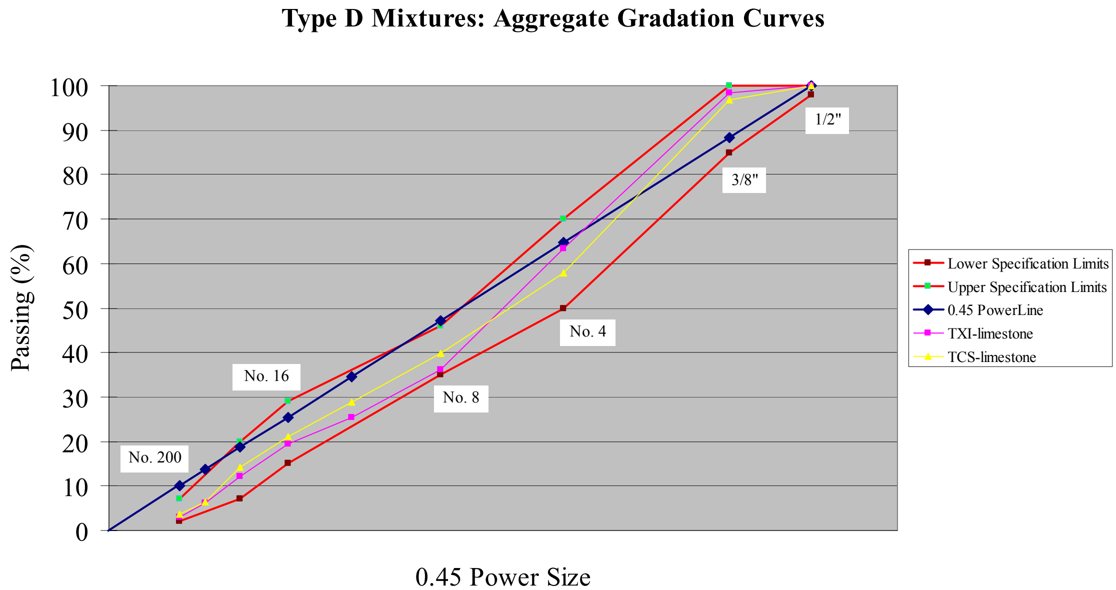


Figure 12. Aggregate Gradations of Dense-Graded Type D Mixtures.

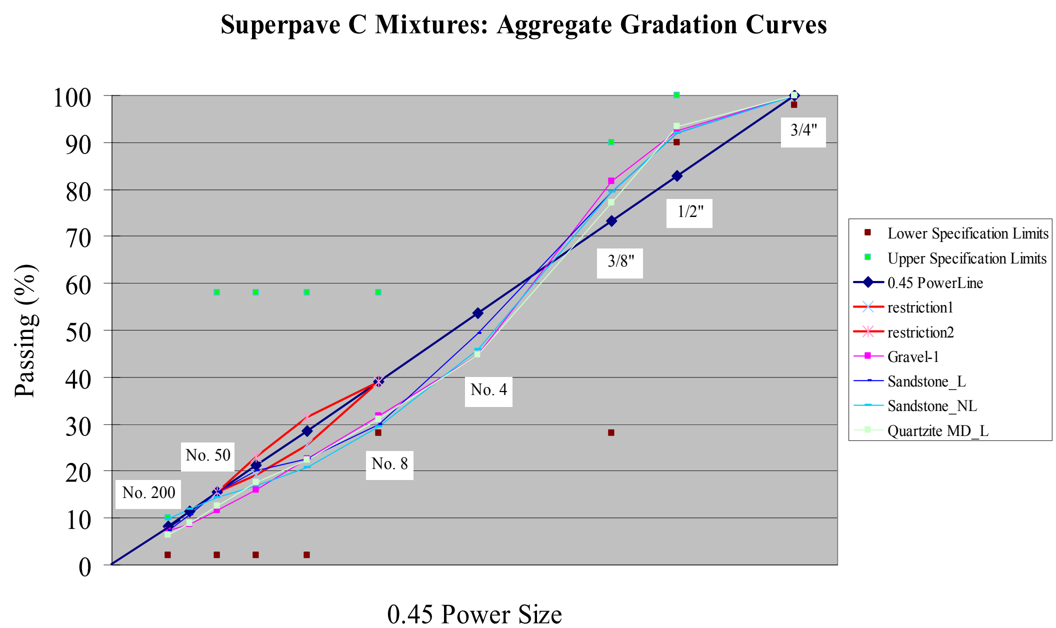


Figure 13. Aggregate Gradations of Superpave C Mixtures.

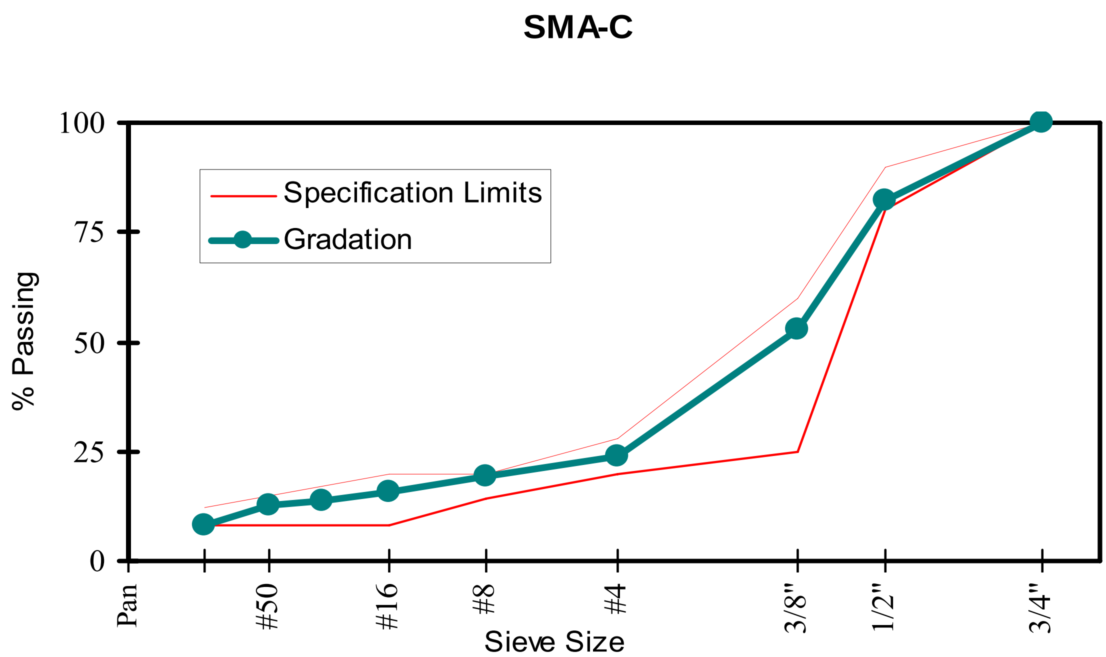


Figure 14. Aggregate Gradation of SMA-C Mixture.

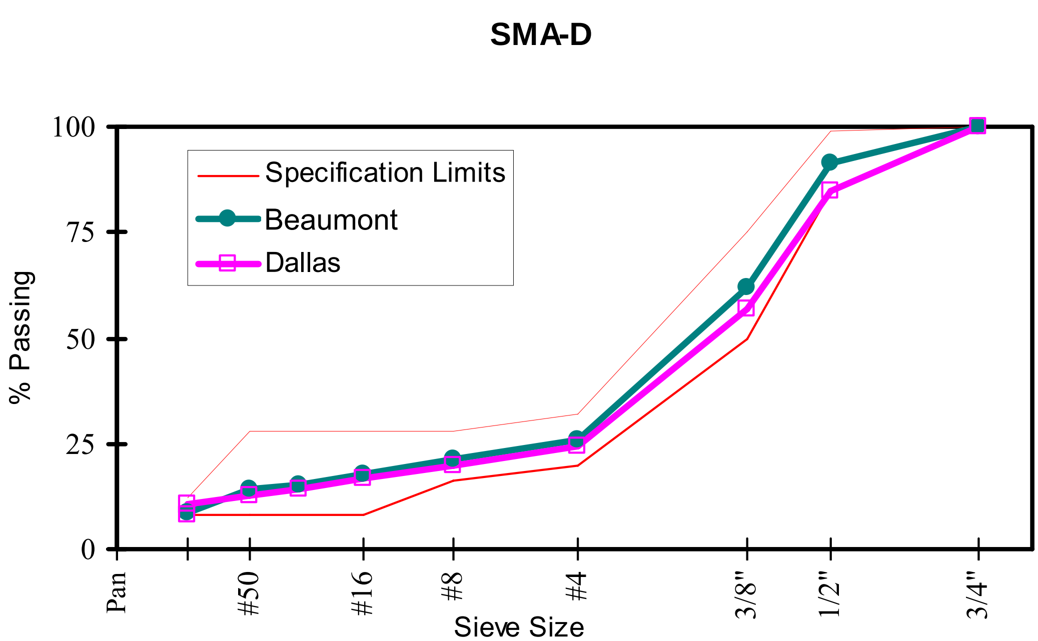


Figure 15. Aggregate Gradations of SMA-D Mixtures.

4.3.1.5 Laboratory test matrix

The laboratory test matrix is shown in Table 10. This matrix contains almost three times the number of HMA mixtures initially stipulated for evaluation in the original project proposal.

Table 10. Laboratory Test Matrix.

Aggregate	Dense-graded Type D mixture		Superpave C mixture			SMA-C mixture	SMA-D mixture
Type	PG64-22	PG76-22	PG64-22	PG70-22	PG76-22	PG76-22	PG76-22
TXI-limestone	√	√					
TCS-limestone	√						
Sandstone-Houston		√					
Gravel-1					√		
Sandstone_L			√				
Sandstone_NL				√			
Quartzite MD L			√				
Granite						√	√ (2)

Note: √=mixture was designed at that cell.

4.3.2 Stage 2: Prepare Laboratory-Mixed Samples

Following the steps described in the "Tex-204-F, Design of Bituminous Mixtures (*79*)," the TGC for Dense-Graded Type D mixtures and the SGC for Superpave C mixtures were used to compact the samples for volumetric analysis. For Superpave C mixtures, N_{design} was 100 gyrations. Two samples were compacted at each asphalt content. A total of eight samples were compacted for each HMA mixture. The theoretical maximum specific gravity was determined at each asphalt content according to *"Tex-227-F, Theoretical Maximum Specific Gravity of Bituminous Mixtures (79)."* The bulk specific gravity of each sample was determined at each asphalt content according to "Tex-207-F, Determining Density of Compacted Bituminous Mixtures (*79*)." Then, the automated mix design program was used to calculate density and the VMA (and VFA) of the molded specimens. The detailed volumetric design information is presented in Appendix B.

4.3.3 Stage 3: Determine OAC of HMA Mixtures

The OAC was determined at 96 percent density for each mixture. The results are presented in Table 11. Table 11 also lists the VMA (and VFA) value of each mixture. It can be seen that the VMA value of each mixture is larger than the minimum VMA requirement, and the VFA value of each Superpave mixture is within the allowable range. The next stage is to evaluate the performance of these mixtures.

Table 11. OAC Determined at 96 Percent Density for Each Mixture.

Mixtures			OAC (%) at 96 % density	VMA (%)		VFA (%)	
Mixture type	Aggregate+asphalt binder	Source		Calculated	Required min. value	Calculated	Required min. value
Dense-Graded Type D	TXI-PG76-22	US82	4.7	15.2	15.0	N/A	N/A
	TXI-PG64-22	US82	4.8	15.2	15.0	N/A	N/A
	TCS-PG64-22	TCS	5.5	16.3	15.0	N/A	N/A
	Sandstone-PG76-22	Houston	5.4	16.0	15.0	N/A	N/A
Superpave C N_{design}=100	Gravel-1-PG76-22	IH20	5.5	16.4	15.0	74.5	73-76
	Sandstone_L-PG64-22		5.0	15.1	15.0	73.0	73-76
	Sandstone_NL-PG70-22		5.1	15.3	15.0	74.0	73-76
	Quartzite MD L-PG64-22		5.4	16.3	15.0	75.4	73-76
SMA-C N_{design}=75	Granite-PG76-22	Houston	6.0	18.0	17.5	N/A	N/A
SMA-D N_{design}=75	Granite-PG76-22	Dallas-IH635	6.0	18.0	17.5	N/A	N/A
	Granite-PG76-22	Beaumont-US96	6.3	18.4	17.5	N/A	N/A

4.3.4 Stage 4: Evaluate Mixture Properties

4.3.4.1 Sample preparation

The SGC was used to compact samples for the HWTT and the OT. Initially, 150 mm (6 in) diameter by 62 mm (2.5 in) high samples were compacted to 7±1 percent AV contents under a variable number of gyrations. Four specimens (two for the HWTT and two for the OT) were molded at the OAC (Table 11) for each type of mixture.

For each mixture type, two replicates were prepared for the HWTT after minimal trimming. Figure 16 shows a pair of the trimmed test specimens in the HWTT mold.

Figure 16. Trimmed HWTT Specimens.

For the OT, the molded samples need more trimming; test samples are required to be 150 mm (6 in) long by 75 mm (3 in) wide by 38 mm (1.5 in) high. Figure 17 shows the process of sample cutting. A double-blade saw was used to prepare the OT samples. Two replicates were prepared for the OT. It should be noted that AV contents of OT specimens normally ranged from 6.0 to 7.6 percent after trimming the gyratory molded samples.

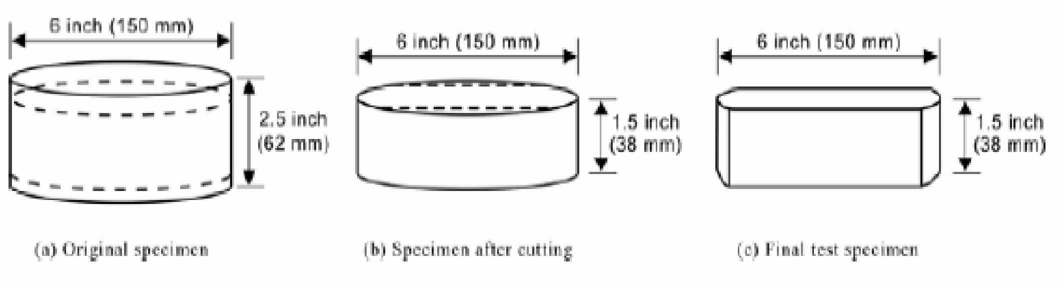


Figure 17. OT Sample Preparation.

4.3.4.2 Hamburg Wheel Tracking Test

The HWTT is a routine TxDOT standard test ("Tex-242-F, Hamburg Wheel Tracking Test [*79*]") for evaluating the potential rutting and moisture damage of asphalt mixtures. The Hamburg Wheel Tracking Device is shown in Figure 18. TxDOT has found that asphalt binder PG has a significant influence on rutting and moisture damage. The pass/fail criteria, as presented in Table 12, is based on asphalt binder PG.

The original HWTT used an HMA slab with dimensions of 320 mm × 260 mm × 40 mm (12.6 in × 10.2 in × 1.6 in). However, the test procedure was modified to accommodate gyratory molded samples: 150 mm (6 in) diameter by 62 ± 1 mm (2.5 in) height. The test is conducted in a water bath at constant temperature: 50 ºC (122 °F). The sample is tested under a rolling 47 mm (1.85 in) wide steel wheel using a 705 N (158 lb) force. As shown in Table 12, for a PG76-22 binder the sample is subjected to 20,000 load passes or to failure that is specified as a rut of 12.5 mm (0.5 in). Rut depths are measured at several locations including the center of the wheel travel path, where usually it reaches the maximum value. One forward and backward motion is counted as two passes.

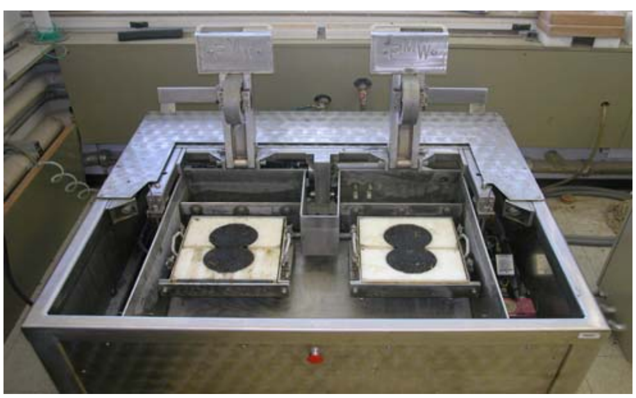


Figure 18. Hamburg Wheel Tracking Device.

Table 12. Hamburg Wheel Tracking Test Criteria (79).

Asphalt binder	No. of passes	Max. rut depth
PG64-22	10,000	12.5 mm (0.5 in)
PG70-22	15,000	
PG76-22	20,000	

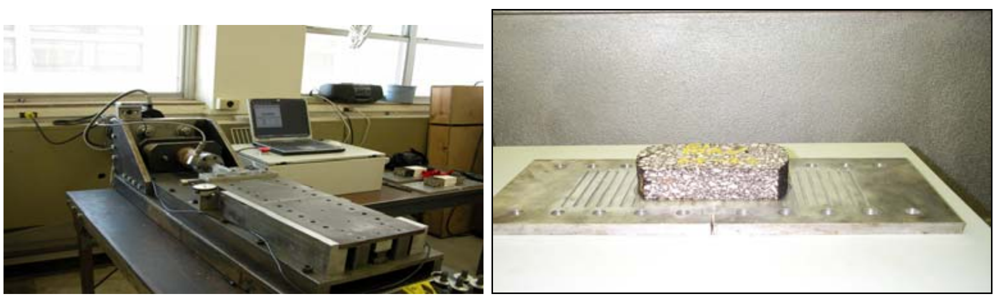
4.3.4.3 TTI Overlay Tester

The TTI OT was upgraded to a fully computer-controlled system by Zhou and Scullion (5) under Project 0-4467. Figure 19 shows the upgraded OT equipment. The key parts of the apparatus consist of two steel plates, one fixed and the other which moves horizontally to simulate the opening and closing of joints or cracks in old pavements beneath an overlay.

- OT Testing Conditions

The upgraded OT is operated in a controlled-displacement mode under the following conditions:

- o temperature: 25 ºC (77 ºF);
- o opening displacement: 0.62 mm (0.025 in);
- o loading rate: 10 sec per cycle; and
- o load form: a repeated load is applied in a cyclic triangular waveform with a constant maximum displacement (shown in Figure 20).



(a) Equipment **(b) Plate and Specimen**

Figure 19. Upgraded TTI Overlay Tester Equipment, Plate, and Specimen.

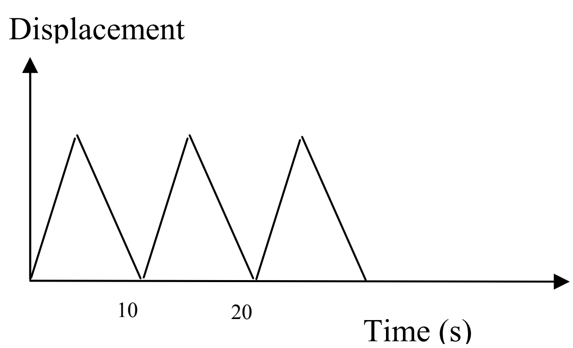


Figure 20. Schematic Diagram of Loading Form.

- Definition of Cracking Life of an HMA Mixture

Cracking life of an HMA mixture is defined as the number of cycles needed to propagate a crack through a specimen under a defined test condition. As validated in Project 0-4467 (*5*), this value is a good indicator of cracking resistance of HMA mixtures.

- Determination of Cracking Life of an HMA Mixture

In the past, the number of cycles to failure was subjectively determined by the operator's visual observation on crack propagation. The life was defined as the number of cycles until a crack was clearly present on the top of the specimen. There are two disadvantages regarding visual observation of the crack: the first is that the operator(s) has to watch the whole testing period; the second is the subjectivity of the operator(s). Thus, it is necessary to develop a methodology to objectively determine the cracking life of an HMA mixture.

Two alternatives have been proposed for cracking life determination: 1) loading shape method and 2) load reduction method. Both methods are discussed as follows:

1) Loading shape method

Zhou and Scullion (*5*) proposed to automate the cracking life of an HMA mixture determination by analyzing the load and displacement versus the time plot for Project 0-4467. A typical set of data is presented in Figure 21, showing load and displacement for each opening and closing cycle. From observations of the results from many overlay tests, it is proposed that this plot has three distinct phases, as described below.

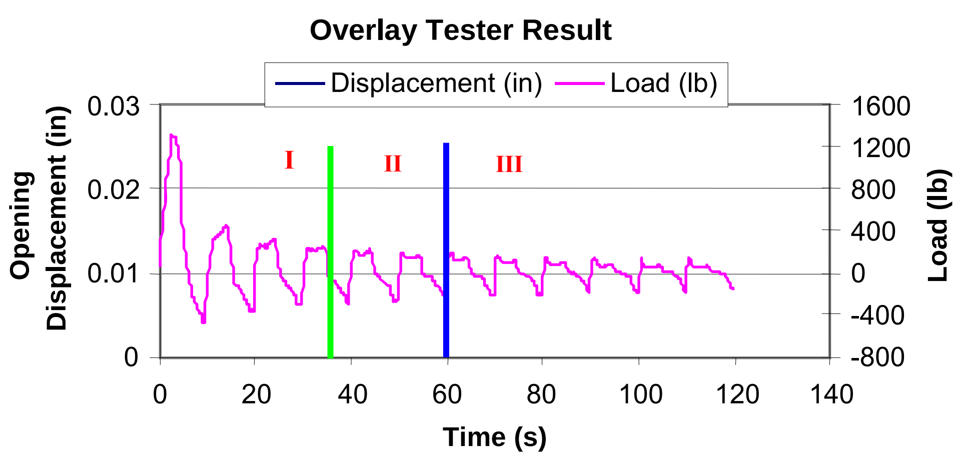


Figure 21. Typical Overlay Tester Result (Each Opening and Closing Cycle is 10 s).

- Phase I: Crack initiation and steady propagation

In this phase the load and displacement have similar shapes. As the displacement increases, the load also increases. For the first cycle, the load reaches its maximum value before the displacement arrives at the maximum displacement. This indicates that the crack initiates at the bottom. After the first cycle, the load decreases rapidly as the crack starts to propagate through the specimen. However, both load and displacement reach maximum value at the same time. In this stage, the cracking is steadily and slowly propagating to the top surface.

- Phase II: Late crack propagation

  Phase II is the late stage of crack propagation, which is monitored as a saddle-shaped load. The saddle-shaped load indicates that the crack has partially gone through the whole cross section of the specimen. In fact, the first peak load is associated with the minor adhesion as the specimen gap is closed and the two halves of the specimen bond together. Then, the load rapidly decreases just after breaking the weak adhesion bonds. With the increasing opening displacement, more loading is needed to break the remaining parts of the specimen. Corresponding to the maximum displacement, there is another peak load. With continuing cyclic loading, the crack will completely break the specimen and the second peak load disappears. This indicates the onset of Phase III.

- Phase III: Specimen failure

  As described above, the crack has propagated completely through the specimen in this phase. The maximum load induced by the minor adhesion occurs well before the maximum displacement.

Based on the above discussion, the cracking life of an HMA mixture can thus be defined by the number of cycles corresponding to the onset of Phase III. To demonstrate, Figure 21 shows the OT result of an HMA specimen. Using the loading shape method described above, the reflective cracking life of the specimen was determined to be six (6) cycles.

The loading shape method is theoretically sound and has been successfully employed to define the cracking life of HMA mixtures since the beginning of Project 0-4467. Furthermore, a similar approach has been proposed to define the fatigue life of an HMA mixture under bending beam fatigue tests *(88)*. However, the loading shape method has a limitation: complexity. It is well known that the traditional bending beam fatigue life of an HMA mixture is determined based on 50 percent load reduction from the initial load. Compared to the load reduction method, the loading shape method is relatively complex. In order to make the OT more acceptable to pavement engineers and industries, a load reduction method, similar to the bending beam fatigue test, was developed for determining the crack life of HMA mixtures. This approach is discussed below.

2) Load reduction method

First, a starting load value should be selected. Theoretically, any load value can be selected, such as the maximum load values either at the first, second, or fifth cycle. However, it will be significant if the one selected has a physical meaning. The maximum load at the first cycle is preferred, as it represents a "strength" characterization of HMA mixture. Beyond this value, an HMA mixture starts to crack. Therefore, the maximum load at the first cycle was selected as the starting load in the OT.

In all subsequent cycles of the test the load is measured, and this value progressively decreases as the crack propagates. The next step is to establish a load reduction level to define failure. Results from more than 200 OT trials were reviewed. From preliminary comparison of the results with the shape method described above, it appeared that a value of around 90 percent would be required. However, it was difficult to find a perfect percentage of load reduction applicable to all HMA mixtures. The reasonable percentage of load reduction, for dry and stiff HMA mixtures (less than 100 cycles to fail), was around 91 percent; however, this value became 95 percent for rich and soft HMA mixtures. The results are shown in Figure 22. The number of cycles shown in the x-axis was determined based on the loading shape method, which was taken as the baseline to select the load reduction level. The y-axis shows the number of cycles determined based on the load reduction method. Three load reduction levels, 91, 93, and 95 percent, are shown in Figure 22. It can be seen that 95 percent load reduction over predicts the number of cycles when its value is less than 600 cycles. Inversely, 91 percent load reduction often underestimates cracking life of mixtures. For those mixtures having crack life less than 300 cycles, 93 percent load reduction is a good choice. More than that, the number of cycles will be underestimated using 93 percent load reduction level, but this makes the mixtures more conservative.

Based on the data generated in this research project, 93 percent load reduction was recommended. This means that an HMA sample fails when losing 93 percent of its "strength," measured from the first load cycle.

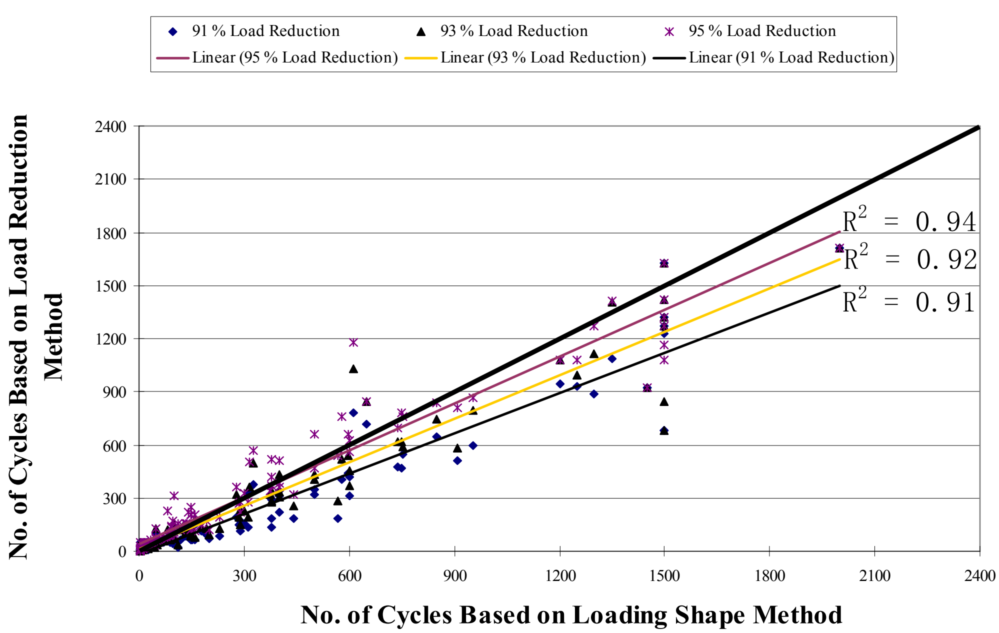


Figure 22. Comparison of the No. of Cycles Load Reduction Method vs. Loading Shape Method.

- Preliminary pass/fail criterion on cracking requirement

The preliminary failure criterion on cracking resistance is 300 cycles, which was established based upon the OT testing results of field cores from different locations around Texas (5). HMA mixtures that cannot achieve this minimum level of performance may experience early reflective cracking distress. In year one of the study this criterion will be used until a better pass/fail criterion is developed. In summary, the reflective cracking criteria are:

  - Pass: OT result ≥ 300 cycles @ 93 percent load reduction;
  - Fail: OT result < 300 cycles @ 93 percent load reduction.

4.3.4.4 The HWTT and OT results

Table 13 presents the HWTT and OT results for each mixture. It should be specifically noted that the HWTT rut depths reported in Table 13 for mixtures with PG64-22, PG70-22, and PG76-22 binder correspond to 10,000, 15,000, and 20,000 wheel passes, respectively, and that the cracking life (no. of cycles) shown in Table 13 was determined based on 93 percent load reduction starting from the first cycle.

Table 13. The HWTT and the OT Results for Each Mixture at OAC.

Mixture type	Aggregate	Asphalt binder	Design AC (%)	VMA (%)	HWTT (mm)	OT* (cycles)	Asphalt absorption (%)
Dense-Graded Type D	TXI-limestone	PG64-22	4.8	15.2	3.0	189	0.07
	TXI-limestone	PG76-22	4.7	15.2	5.0	200	0.14
	TCS-limestone	PG64-22	5.5	16.3	13.4	25	0.93
	Sandstone	PG76-22	5.4	16.0	4.6	580	0.16
Superpave-C N_{design}=100	Sandstone	PG64-22	5.0	15.1	5.9	112	1.07
	Sandstone	PG70-22	5.1	15.3	2.4	35	1.37
	Gravel	PG76-22	5.5	16.4	3.0	105	0.30
	Quartzite	PG76-22	5.4	16.3	3.0	230	0.63
SMA-C N_{design}=75	Granite	PG76-22	6.0	18.0	4.9	450	0.30
SMA-D N_{design}=75	Granite	PG76-22	6.0	18.0	4.2	410	0.27
	Granite	PG76-22	6.3	18.4	7.2	>1500	0.31

Note: the OT result for each mixture is an averaged cycles of two samples.

The following interesting observations can be seen from Table 13.

- For Dense-Graded Type D and Superpave C mixtures: All mixtures, except Dense-Graded Type D mixtures with TCS-limestone, passed the HWTT test. In contrast, all except the Dense-Graded Type D mixture with low absorptive sandstone failed the OT criterion. This observation indicates that mixtures designed based on the current TxDOT mixture design procedure are generally rut resistant but not crack resistant.
- For SMA mixtures: All three SMA mixtures passed both rutting and cracking criteria. These mixtures have: 1) higher quality aggregates, 2) minimum asphalt content requirement: 6 percent, and 3) PG76-22 asphalt binder. This observation also indirectly indicates that the OT criterion (300 cycles requirement) is reasonable, and OT criterion is a good indicator of having enough asphalt binder in the mixtures passing the requirement.
- In general, more asphalt content is needed to balance rutting and cracking requirements. More discussion on this issue is presented in the next section.

The main conclusion the authors wish to strongly emphasize is the significance of asphalt absorption by the aggregate on the performance of these Texas mixtures in the OT. It is proposed that absorption is a selective process where the light oils from the asphalt are drawn into the aggregates over time, leaving a dry, brittle binder. The quality of the rock appears to be critical in developing mixtures that pass both requirements. From Table 13, four aggregates had asphalt absorptions more than 0.5 percent, and the highest asphalt absorption is 1.37 percent for the Superpave C mixture with sandstone from IH20. In the current TxDOT mixture design procedure, the asphalt absorption by aggregates is partially addressed through increasing the minimum VMA requirement by 1 percent. This approach is used because it is difficult to measure the bulk specific gravity of aggregates that is required to calculate the asphalt absorption by aggregates. It is worth noting that a 1 percent increase in VMA provides only an increase of 0.4 percent in asphalt content. If the asphalt absorption by aggregates is higher than 0.4 percent, the design asphalt content will still be low potentially.

4.4 DISCUSSION ON WAYS TO INCREASE DESIGN ASPHALT CONTENT

As stated previously, the current TxDOT HMA mixture design is a volumetric-based procedure. The most important volumetric parameters are AV and VMA. Generally speaking, the difference between AV and VMA controls the amount of asphalt content (in volume) required in the mixture. Thus, it is apparent that either reducing the AV requirement, increasing the VMA requirement, or both, can achieve higher design asphalt content. However, there is another attractive way to increase design asphalt content: reducing the compaction effort (or reduce the number of gyrations), because a lower number of gyrations will result in higher VMA. In summary, there are at least four ways to increase the design asphalt content:

1. Reduce the compaction effort (or reduce the number of gyrations).
2. Increase VMA requirement.
3. Reduce design AV requirement.
4. Increase VMA and reduce AV requirements.

The advantages and disadvantages of each method are discussed in Table 14.

Table 14. Summary Comparison among Ways to Increase the Asphalt Content.

Ways to increase asphalt content	Advantage	Disadvantage
Reducing N_{design}	• No need to change volumetric requirements	• Potentially no asphalt content increase at all if contractors change the aggregate gradation • Weak skeleton and stiffness • Increase the risk of potential rutting for all mixes, especially those with soft binder (PG64-XX)
Increasing min. VMA	• Surely increasing asphalt content • No need to change the AV requirement • No need to change N_{design} • Little (or no) effect on rutting resistance of stiff mixtures with PG76-XX or PG70-XX	• Potential rutting risk for mixtures with soft binder (such as PG64-22) • Need more crushed and good quality aggregates • Significantly difficult to meet the increased minimum VMA requirement • Difficult to compact
Increasing design density (or reducing design AV)	• Surely increasing asphalt content • No need to change aggregate requirements • No need to change VMA requirement • No need to change N_{design} • More practical implementation in districts • Little (or no) effect on rutting resistance of stiff mixtures with PG76-XX or PG70-XX	• Potential rutting risk for mixes with soft binder (such as PG64-22)
Increasing min. VMA and design density (or reducing design AV)	• Surely increasing asphalt content • No need to change N_{design}	• Need to change aggregate requirements • Need to change VMA and design density (or AV) requirements • Potential rutting risk for mixes with soft binder (such as PG64-22)

It should be noted that reducing N_{design} has a significant effect on asphalt mixture properties. ***If the aggregate blend is held constant during design of a mixture***, reducing the N_{design} will definitely increase the calculated VMA value and hence design asphalt content (if the design AV is kept constant). Figure 23 shows the influence of N_{design} on changes in VMA. For an increase of 25 gyrations, VMA decreases nearly 1 percent. Conversely, for a decrease of 25 gyrations, VMA increases about 1 percent. Note that for a typical HMA mixture, a 1 percent increase in VMA is equal to an increase of 0.4 percent in design asphalt content. However, it is well known that asphalt binder is the most expensive component of HMA mixture. Thus, ***if there is room to reduce the VMA and still meet the specification, the contractor will have incentive to redesign the mixture with a different aggregate skeleton (blend) to reduce the VMA and hence the design asphalt content.*** As a result, reducing only the N_{design} may not guarantee an increase in the design asphalt content, and the VMA and AV may need to change as well. In summary, it is not a simple issue to increase the design asphalt content through changing the N_{design} and/or volumetric requirements.

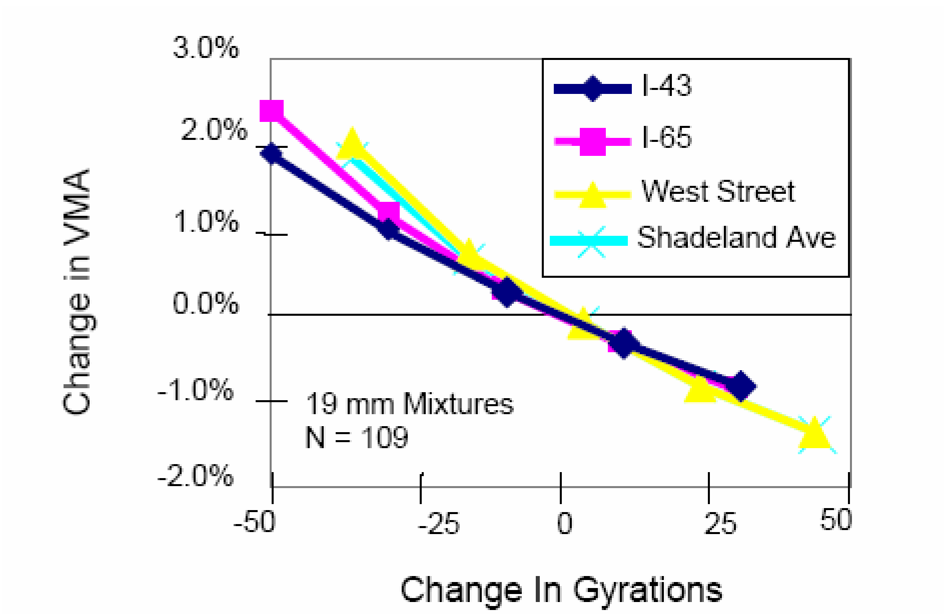


Figure 23. Influence of Change in Gyrations on Change in VMA *(89)*.

The purpose of increasing the design asphalt content is to improve cracking resistance of HMA mixtures. Thus, an alternative way is to evaluate crack resistance of HMA mixtures using the OT and thereby directly determine the design asphalt content based upon the rutting and cracking requirements. This alternative may be simpler, requiring only minor changes to current TxDOT HMA mixture design procedures, and it also has the advantage that it is based on performance-related criteria. More discussion on this proposed design procedure will be presented in the next chapter.

CHAPTER 5

BALANCED HMA MIXTURE DESIGN PROCEDURE

5.1 INTRODUCTION

As reported in Chapter 4, the mixtures designed following the current HMA mixture design procedure are rut resistant, but generally not crack resistant. There are at least four ways (reducing N_{design}, increasing minimum VMA, increasing design density, or both) to potentially increase the design asphalt content and hence cracking resistance of HMA mixtures. Each way has advantages and disadvantages and requires changes to the current HMA design procedure. With this situation, an alternative way to design a balanced HMA mixture is proposed. This proposed HMA mixture design procedure introduces minor changes to the current HMA design practice and is a performance-based design method. The balanced design concept will be described in the next section of this report. Then the 11 HMA mixtures discussed in Chapter 4 will be re-evaluated using the balanced design approach. A case by case discussion on selection of balanced design asphalt content is presented. Finally, a brief discussion on this method is provided at the end of this chapter.

5.2 BALANCED HMA MIXTURE DESIGN PROCEDURE

In Chapter 4 the OT was successfully integrated into the current HMA mixture design process, which is shown in Figure 11. Using this design process in Stage 4 the following four scenarios can occur;

- Scenario 1: the mixture fails rutting and cracking resistance requirements.
- Scenario 2: the mixture passes only the cracking resistance requirement.
- Scenario 3: the mixture passes only the rutting resistance requirement.
- Scenario 4: the mixture at the OAC passes both rutting and cracking resistance requirements.

If Scenario 1 occurs a mixture redesign is required, which could involve a different combination of aggregates, different materials, or a different PG. In the case of Scenario 2, the mixture generally needs to be redesigned, because the current HMA mixture design has a tendency to produce lean mixtures; there is very little room for further reducing the asphalt content. As demonstrated in Table 13 of Chapter 4, most mixture design cases result in Scenario 3. The designed mixture has very good rut resistance, but there is still some potential to add more asphalt binder to improve its crack resistance. For this scenario the researchers propose to increase the asphalt content from the OAC determined from Stage 3, mold additional specimens, and run the HWTT and the OT to evaluate performance. For Scenario 4, the mixture design is complete. If further adjustment of the OAC is preferred, more specimens molded with variable asphalt contents are required.

As demonstrated in Chapter 4, Scenario 3, compared to the others, occurs most frequently. Thus, it is proposed to mold specimens at variable asphalt contents above the OAC rather than only at the OAC. Figure 24 shows the proposed HMA design procedure for balancing rutting and cracking resistance requirements.

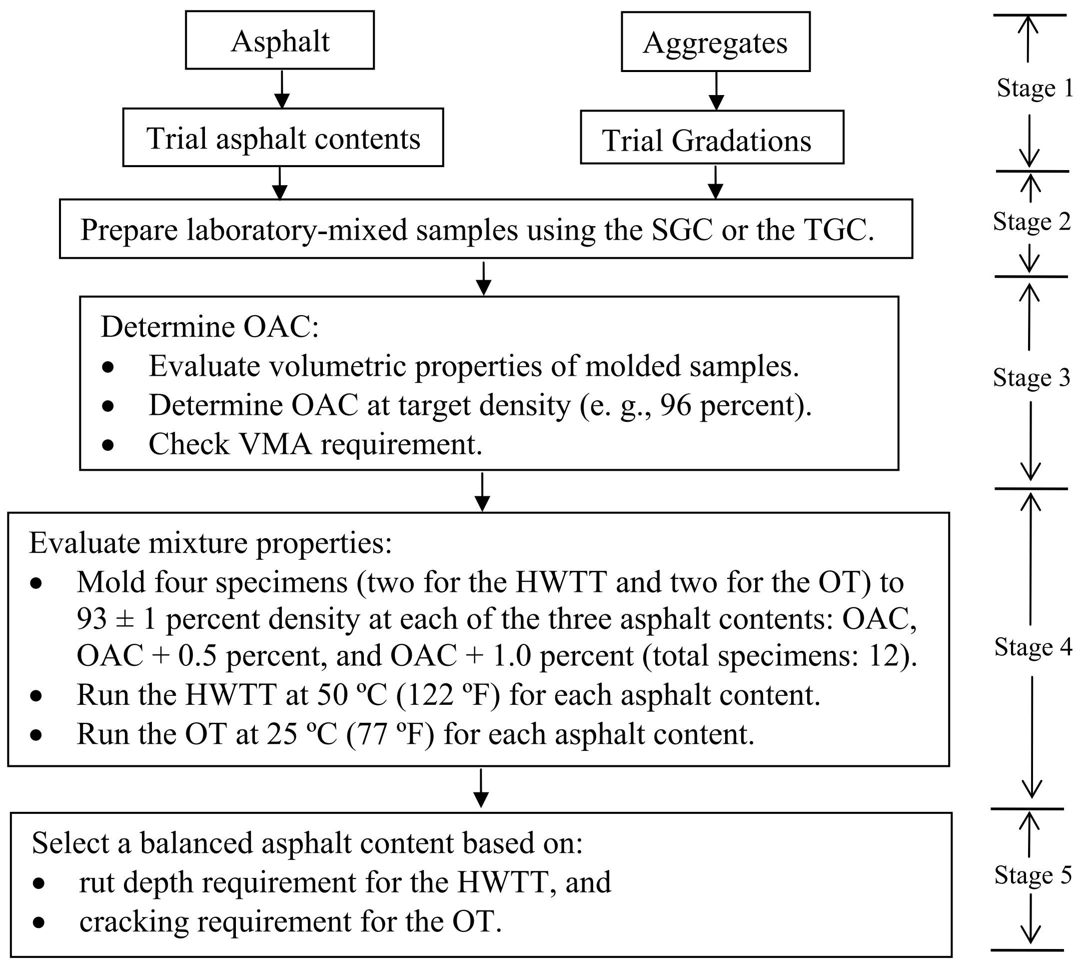


Figure 24. Balanced HMA Mixture Design Procedure.

For a smooth transition from the current design procedure, Stages 1, 2, and 3 in the balanced design procedure are exactly the same as those of current design procedure. The only difference in Stage 4 (*evaluate mixture properties*) is to mold two more Hamburg samples at each asphalt content. The final step will then be to select (if possible) a balanced asphalt content with consideration of rutting, cracking, and construction tolerance. This balanced HMA mixture design procedure is demonstrated in the next section.

5.3 LABORATORY STUDY ON THE BALANCED HMA MIXTURE DESIGN PROCEDURE

The objectives of this laboratory study are to demonstrate the balanced HMA design process and to check its accuracy. In Chapter 4, eleven mixtures were designed following the current TxDOT HMA mixture design procedure. As noted in Chapter 4, six could not meet the cracking requirement (Scenario 3); another failed both the rutting and cracking resistance requirements (Scenario 1). Thus, these seven mixtures were chosen for evaluating the balanced design approach shown in Figure 24. Since Stages 1, 2, and 3 were presented in Chapter 4, only Stages 4 and 5 are presented in this chapter.

5.3.1 Stage 4: Evaluate Mixture Properties

5.3.1.1 Variable asphalt contents

In this stage, both rutting and cracking resistance are evaluated at variable asphalt contents above the OAC determined from Stage 3. The three asphalt contents being evaluated are OAC, OAC + 0.5 percent, and OAC + 1.0 percent. Table 15 presents the variable asphalt contents used for the seven mixtures.

Table 15. Asphalt Contents Used for Performance Evaluation.

Mixture		OAC (%)	OAC + 0.5 (%)	OAC + 1.0 (%)
Mixture type	Aggregate + asphalt binder			
Dense-Graded Type D	TXI-PG76-22	4.7	5.2	5.7
	TXI-PG64-22	4.8	5.3	5.8
	TCS-PG64-22	5.5	6.0	6.5
Superpave C	Gravel-1-PG76-22	5.5	6.0	6.5
	Sandstone_L-PG64-22	5.0	5.5	6.0
	Sandstone_NL-PG70-22	5.1	5.6	6.1
	Quartzite MD L-PG64-22	5.4	5.9	6.4

5.3.1.2 Sample preparation

The SGC was used to compact samples for the HWTT and the OT. Initially, 150 mm (6 in) diameter by 62 mm (2.5 in) high samples were compacted to 7 ± 1 percent air void contents under a variable number of gyrations. Four specimens (two for the HWTT and two for the OT) were molded at each asphalt content, and a total of 12 specimens were fabricated for each type of mixture.

5.3.1.3 Hamburg Wheel Tracking Test

The HWTT described in Chapter 4 was conducted to evaluate rutting resistance of HMA mixtures with variable asphalt contents.

5.3.1.4 Overlay Tester

The OT described in Chapter 4 was conducted to evaluate cracking resistance of the HMA mixtures with variable asphalt contents.

5.3.1.5 The HWTT and the OT results

Figures 25 to 31 show the HWTT and the OT results for each mixture at three asphalt contents: OAC, OAC + 0.5 percent, and OAC + 1.0 percent. It should be specifically noted that the HWTT rut depths illustrated in Figures 25 to 31 for mixtures with PG64-22, PG70-22, and PG76-22 binder correspond to 10,000, 15,000, and 20,000 wheel passes, respectively that the cracking life (no. of cycles) shown in Figures 25 to 31 was determined based on 93 percent load reduction starting from the first cycle. The arrows and box on each of these graphs indicate the asphalt content range over which a balanced design can be achieved.

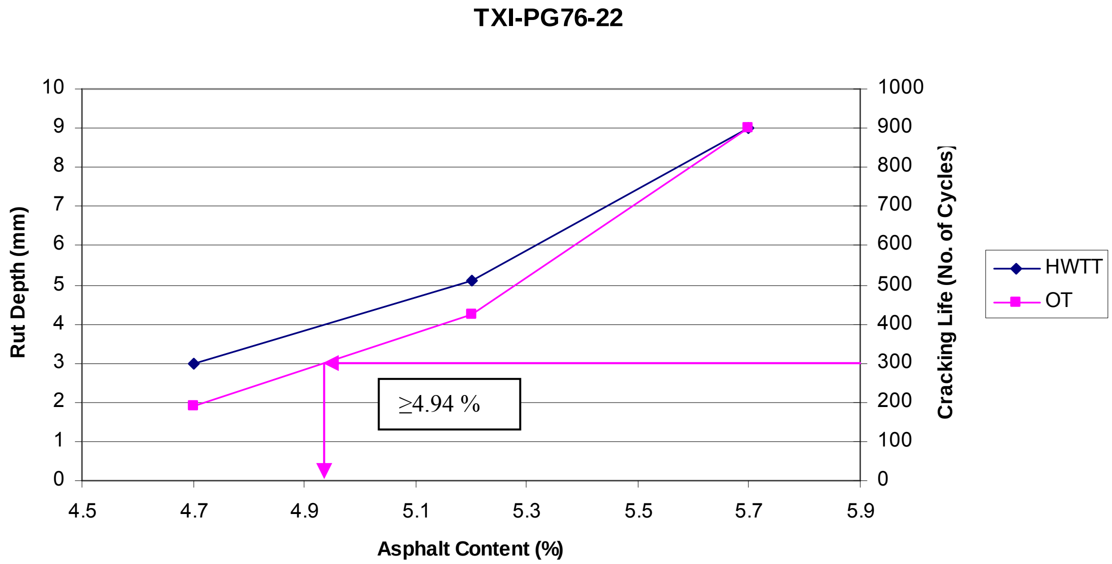


Figure 25. Performance Evaluation: TXI- PG76-22.

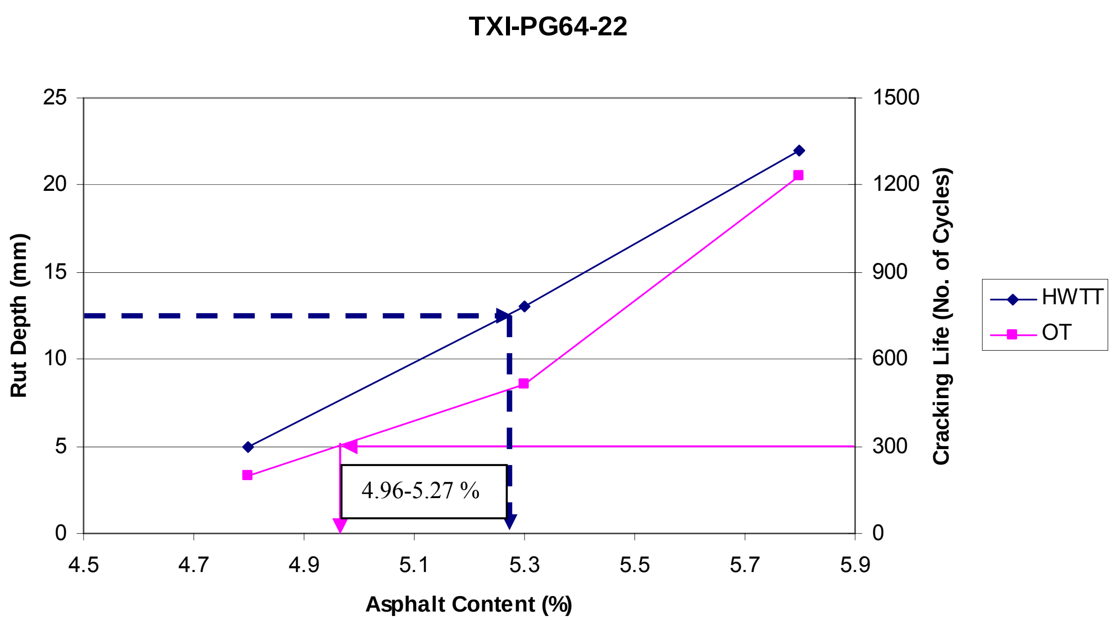


Figure 26. Performance Evaluation: TXI- PG64-22.

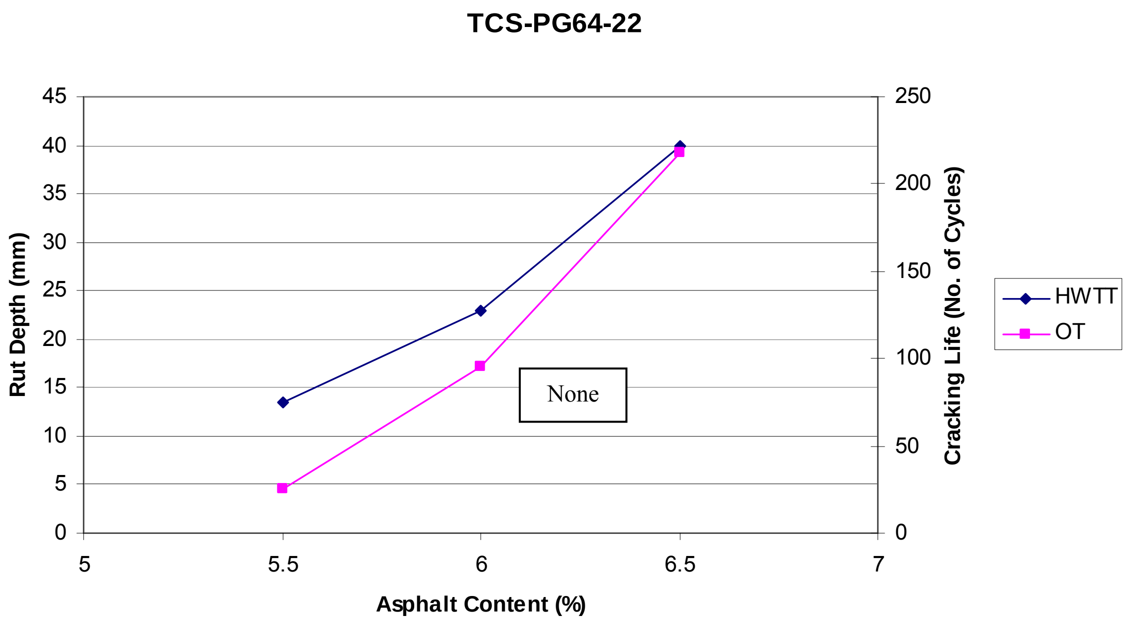


Figure 27. Performance Evaluation: TCS- PG64-22.

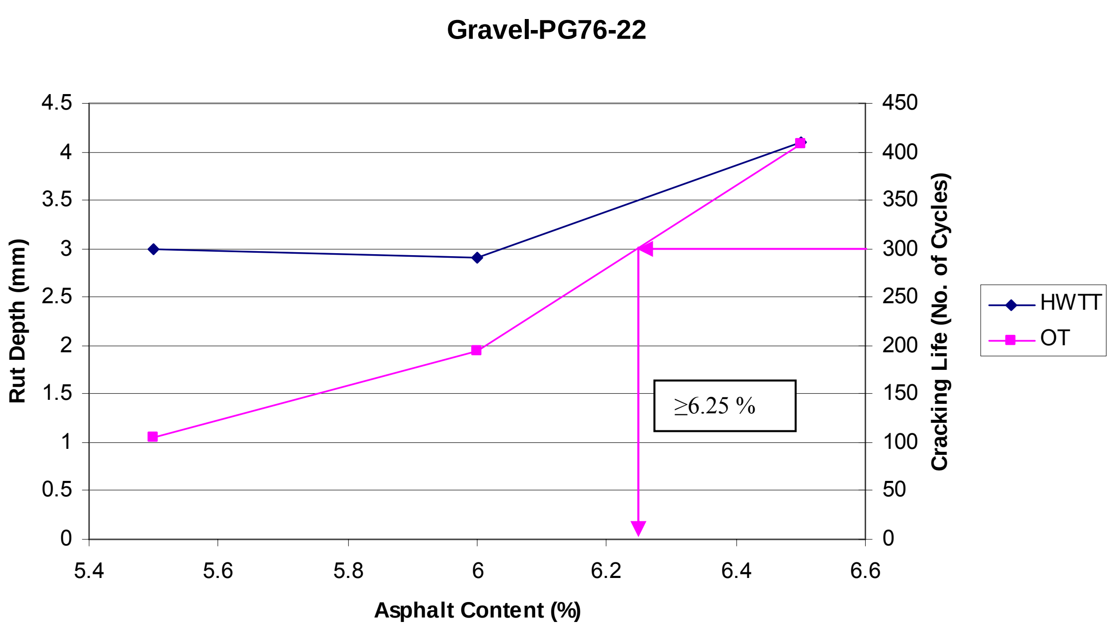


Figure 28. Performance Evaluation: Gravel-1- PG76-22.

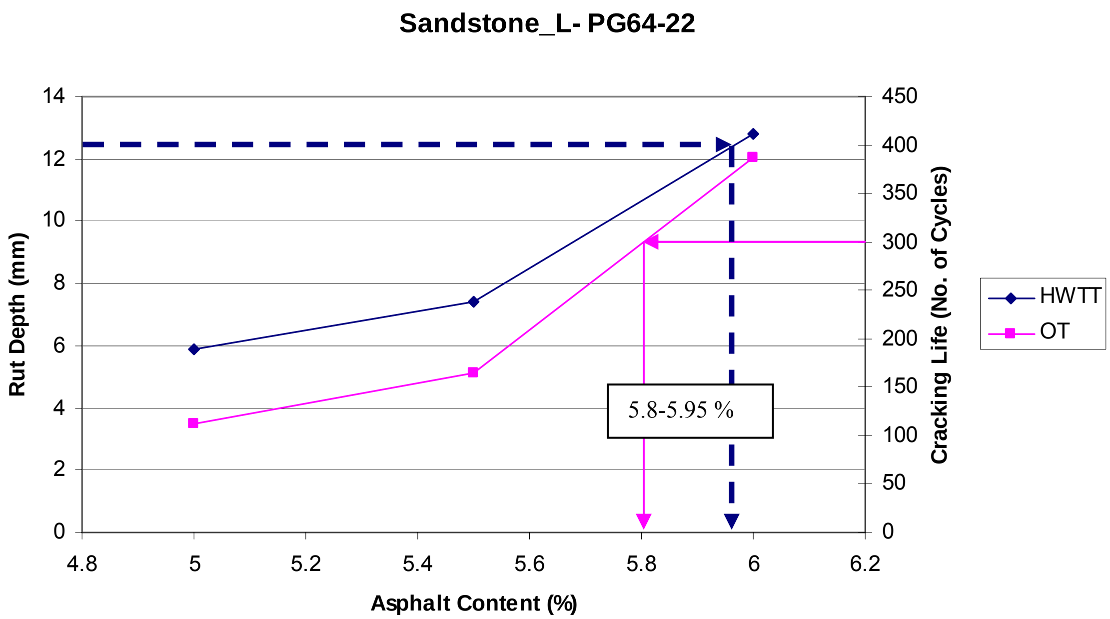


Figure 29. Performance Evaluation: Sandstone_L- PG64-22.

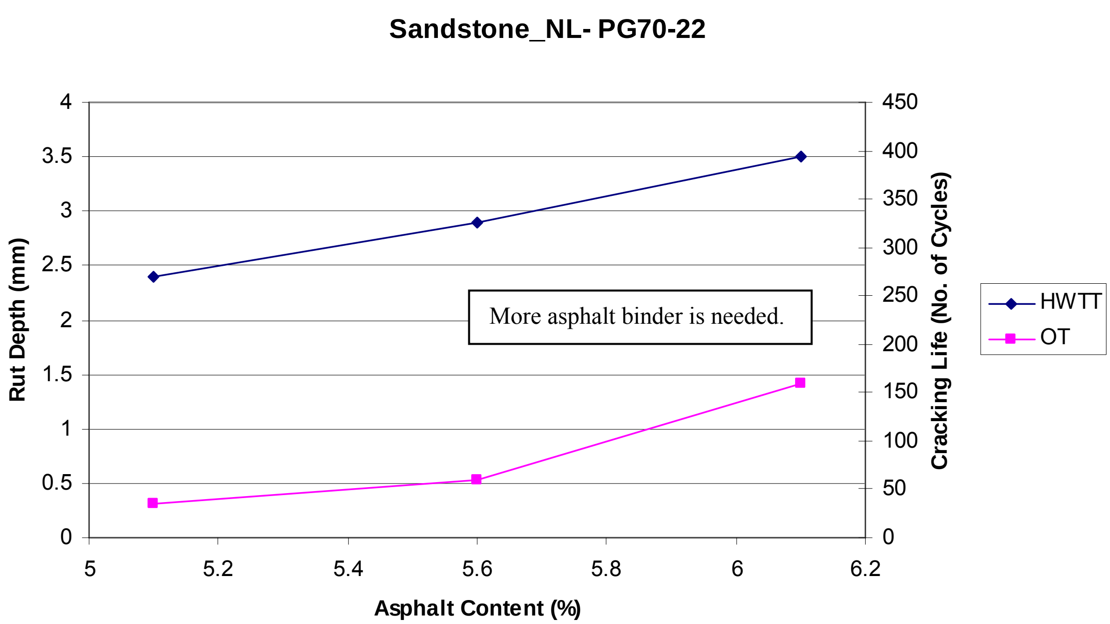


Figure 30. Performance Evaluation: Sandstone_NL- PG70-22.

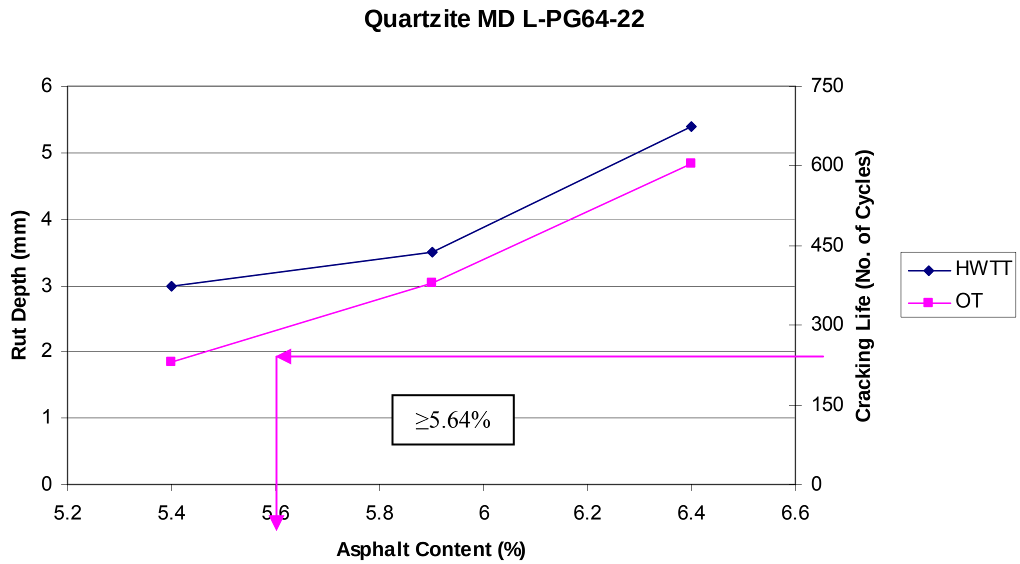


Figure 31. Performance Evaluation: Quartzite MD L- PG64-22.

5.3.2 Selection of a Balanced Asphalt Content

It is sometimes difficult to select an asphalt content because many factors must be considered. In addition to balancing rutting and cracking resistance requirements, construction and other factors also need to be taken into account. For example, there is a ± 0.3 percent operational tolerance of asphalt content in current TxDOT *Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges (78)*. If possible, this tolerance range should be considered when selecting an asphalt content balancing rut and crack resistance. In summary, selection of an asphalt content needs to consider three factors:

- rutting resistance requirement,
- cracking resistance requirement, and
- construction tolerance, if possible.

For the purpose of demonstration, the seven mixtures are analyzed case by case in the following paragraphs.

- Case 1: TXI-PG76-22 Mixture

As shown in Figure 25, the TXI-PG76-22 mixtures passed the rutting resistance requirement at all three asphalt contents. A minimum asphalt content of 4.94 percent is determined to satisfy the cracking resistance requirement. There is a wide range between minimum asphalt content (4.94 percent) and the maximum asphalt content tested in this case (5.7 percent). An asphalt content between 4.94 and 5.70 percent is considered as a balanced design content. Finally, taking into account the construction tolerance (± 0.3 percent), the balanced asphalt content recommended for this case is 5.3 percent. During construction, varying ± 0.3 percent asphalt content will result in construction asphalt content ranging from 5.0 to 5.6 percent, which still fulfills the requirements for rutting and cracking resistance of HMA mixtures. This is an ideal case.

- Case 2: TXI-PG64-22 Mixture

Rutting and cracking requirements are first considered. As indicated in Figure 26, asphalt content should be within a range of 4.96 to 5.27 percent, and its average value is around 5.1 percent. In this case, the asphalt content can vary only ± 0.15 percent without causing rutting and cracking susceptible mixtures and therefore is less than the construction tolerance (± 0.3 percent). In this situation, the designer should select an asphalt content by carefully balancing rutting and cracking resistance requirements. If the rutting resistance requirement outweighs the cracking resistance requirement, the designer may choose 5 percent as the design asphalt content. Conversely, the designer may choose 5.2 percent asphalt content for mixtures having better crack resistance property.

- Case 3: TCS-PG64-22 Mixture

Case 3 is the worst case. Neither the rutting nor cracking resistance requirement can be met within the three trial asphalt contents, as illustrated in Figure 27. In this situation, the designer must change the materials (aggregates and asphalt binder) and redesign the mixture.

- Case 4: Gravel-1-PG76-22 Mixture

As plotted in Figure 28, the gravel-1-PG76-22 mixture has very good rut resistance. The rut depth measured under the HWTT is only 4.1 mm (0.16 in) after 20,000 passes, even if the asphalt content reaches 6.5 percent. To satisfy the cracking resistance requirement, the minimum asphalt content needs to be 6.25 percent. Note that above the 6.5 percent asphalt content, neither the HWTT nor the OT was conducted. In this situation, the designer can either select an asphalt content between 6.25 and 6.5 percent or add one more asphalt content (e. g., 7.0 percent) to further evaluate its rutting and cracking resistance properties before making a final decision.

- Case 5: Sandstone_L-PG64-22 Mixture

Case 5 is similar to Case 2. In the asphalt content range from 5.8 to 5.95 percent, the mixture meets the rutting and cracking resistance requirements, as shown in Figure 29. The designer can choose an asphalt content between 5.8 and 5.95 percent after balancing rutting and cracking resistance requirements.

- Case 6: Sandstone_NL-PG70-22 Mixture

At the three trial asphalt contents (5.1, 5.6, and 6.1 percent), the mixtures show excellent rut resistance but very poor crack resistance, as shown in Figure 30. Since the rut depth after 15,000 passes is still low for the mixture with 6.1 percent asphalt content, it is reasonable to recommend that the designer should add two more sets of asphalt contents (e. g., 6.6 and 7.1 percent) for further evaluation before selecting a balanced asphalt content.

- Case 7: Quartzite MD L- PG64-22 Mixture

As can be seen in Figure 31, Case 7 is another ideal case. Similar to Case 1, the designer can choose the design asphalt content in the range of 5.64 to 6.40 percent, within which the mixture has neither rutting nor cracking problems. In this case, the balanced asphalt content recommended is 6.1 percent, since rutting is still not a problem even above 6.4 percent.

In summary, the above seven case studies cover most of the situations HMA designers often face during the mixture design process. Note that the TCS-limestone Dense-Graded Type D mixture was included to demonstrate the situation of failing the rutting and the cracking resistance criteria. In the future if mixtures initially fail both the HWTT and OT then they should be redesigned. Except for the TCS-limestone Dense-Graded Type D mixture, a balanced asphalt content could be determined following the proposed HMA mixture design procedure.

5.4 DISCUSSION

As noted previously, Stages 1, 2, and 3 of the current HMA mixture design were included in the balanced HMA mixture design procedure in order to minimize changes. However, it is worth noting that there is some overlap between Stages 2, 3, and 4 in the balanced HMA design procedure. In Stages 2 and 3, specimens are molded at four trial asphalt contents using the SGC or TGC. Then, the OAC is determined based on the volumetric criteria (such as density and VMA). In Stage 4, the asphalt content is varied around the OAC, and the specimens are again molded at three asphalt contents (e. g., OAC, OAC + 0.5 percent, and OAC + 1.0 percent). Finally, the balanced asphalt content is based on meeting the rutting and cracking resistance requirements in Stage 5. It can be seen that the output of Stages 2 and 3 is only the OAC, which then becomes the starting asphalt content for the Stage 4 evaluation. As demonstrated above, the final (or balanced) asphalt content is selected based on performance (rutting and cracking) rather than volumetric requirements, Stages 2 and 3 would become unnecessary if the range of trial asphalt contents for Stage 4 is known in advance. Furthermore, the technique of taking the OAC as a starting trial asphalt content in Stage 4 should be further investigated, because Case 6 could not pass the cracking requirement even if 1 percent additional asphalt binder was added. The reasonable trial asphalt contents are discussed in Chapter 6. With the known range of trial asphalt contents within which HMA mixtures can pass rutting and cracking resistance requirements, a simplified version of balanced HMA mixture design procedure is recommended in Chapter 7.

CHAPTER 6

RECOMMENDATION OF TRIAL ASPHALT CONTENTS FOR BALANCED HMA MIXTURE DESIGN

6.1 INTRODUCTION

Selection of reasonable trial asphalt contents is the key to successfully determining a balanced asphalt content at which the designed mixture can meet the rutting and cracking resistance criteria. The trial asphalt contents are recommended based on the volumetric design in the balanced HMA mixture design method proposed in Chapter 5. In most cases, these trial asphalt contents work well. However, they may fail to cover the balanced asphalt content, such as Case 6 in Chapter 5. Thus, it is important to recommend reasonable trial asphalt contents for different HMA mixtures.

In fact, the rutting resistance requirement defines the upper limit of the trial asphalt content, above which an HMA mixture will not be able to meet the rutting resistance requirement. Meanwhile, the cracking resistance requirement determines the lower limit of the trial asphalt contents, below which an HMA mixture will not have adequate cracking resistance and durability. It is well known that the upper and lower limits (or the rutting and cracking resistance of an HMA mixture) are influenced by many factors, such as asphalt binder PG, aggregate gradation, types of aggregates, etc. Therefore, extensive laboratory tests are required to accurately estimate the upper and lower limits of the trial asphalt contents. In this chapter a Phase II experimental design is undertaken to identify reasonable upper and lower limits of asphalt content. The HWTT and associated rutting criteria are used to evaluate the upper limit concept; meanwhile, the OT is employed to evaluate the lower limit. With the known upper and lower limits of trial asphalt contents, trial asphalt contents can be recommended for different HMA mixtures. Finally, a simplified version of balanced HMA mixture design procedure is proposed at the end of this chapter.

6.2 EXPERIMENTAL TESTING PROGRAM

The objectives of this experimental design are to identify the major factors influencing rutting and cracking performance, then to estimate the upper and lower limits of the trial asphalt contents. The principle for this Phase II experimental design is to include as many variables as can be tested within the scheduled work plan in the original project proposal.

6.2.1 Mixture Variables Considered

The major variables considered in Phase II include the following:

- Asphalt binder PG: Two asphalt binders generally used in Texas were included, PG64-22 and PG76-22. Note that these two asphalt binders are the same as those used in Phase I in Chapters 4 and 5. Their dynamic shear moduli are presented in Table 8 (Chapter 4).

- Aggregates: The same aggregates as those used in Phase I were included. Additionally, a limestone with medium absorption from US281, Fort Worth, Texas, was also included in Phase II. The seven aggregate types used in this Phase II study were:

  1) TXI-limestone,
  2) FW-limestone,
  3) TCS-limestone,
  4) quartzite,
  5) sandstone,
  6) gravel, and
  7) granite.

  Again, bulk specific gravity and water absorption of each aggregate were measured according to "Tex-201-F, Bulk Specific Gravity and Water Absorption of Aggregate *(79)*." The purpose of measuring the bulk specific gravity is to determine asphalt absorption by aggregate type from analysis of volumetric properties of molded samples. Table 16 presents the bulk specific gravity and water absorption values of each aggregate. In this research project, asphalt absorption is arbitrarily categorized based on the water absorption:

  - High absorption: water absorption is larger than 2.0 percent;
  - Intermediate absorption: water absorption is between 1.0 and 2.0 percent; and
  - Low absorption: water absorption is less than 1.0 percent.

Table 16. Bulk Specific Gravity and Water Absorption of Aggregates.

Type of mixtures	Aggregate	Bulk specific gravity	Water absorption (%)
Dense-Graded Type D	TXI-limestone	2.752	0.7
	TCS-limestone	2.559	2.3
	FW-limestone	2.676	1.0
Superpave-C	Gravel-1	2.584	0.9
	Gravel-2	2.578	0.9
	Sandstone_L	2.481	2.3
	Quartzite-MD_L	2.628	1.5
	Granite	2.680	0.7

- Mixture types and aggregate gradations: Since the SMA mixtures generally pass the rutting and cracking resistance criteria, they were excluded from Phase II. Two types of mixtures (Dense-Graded Type D and Superpave C) were investigated in Phase II. For each type of mixture, aggregate gradation was varied to investigate the influence of gradation. Additionally, the same gradation was used for the different aggregates to compare the influence of aggregate on rutting and cracking resistance.

Gradations of Dense-Graded Type D mixtures and Superpave C mixtures are shown in Figures 32 and 33, respectively. Detailed aggregate gradations are listed in Appendix A.

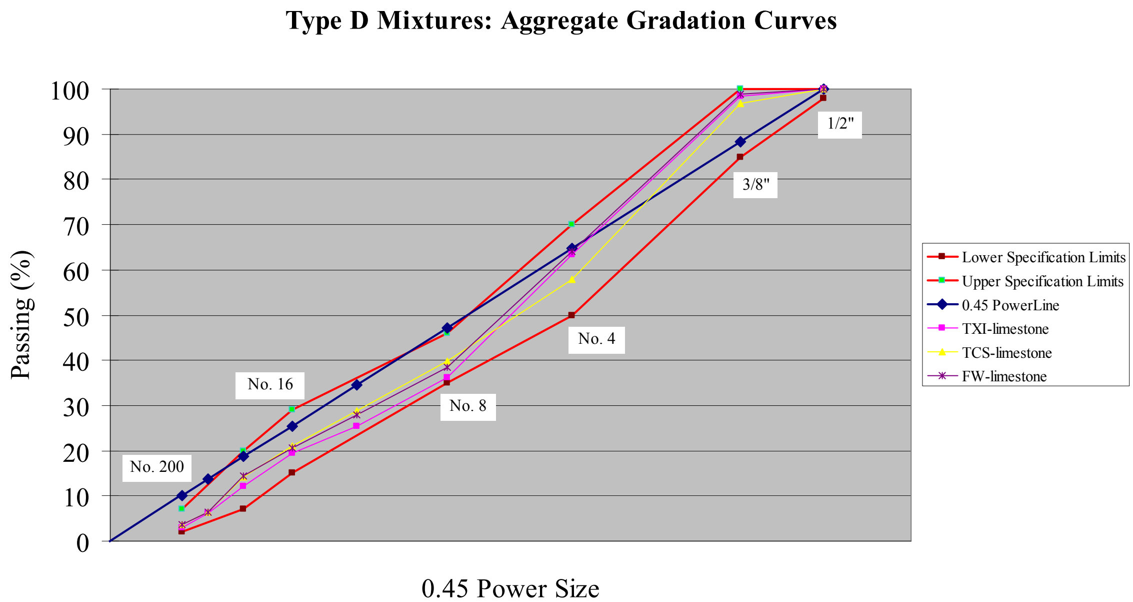


Figure 32. Aggregate Gradation Curves of Dense-Graded Type D Mixtures.

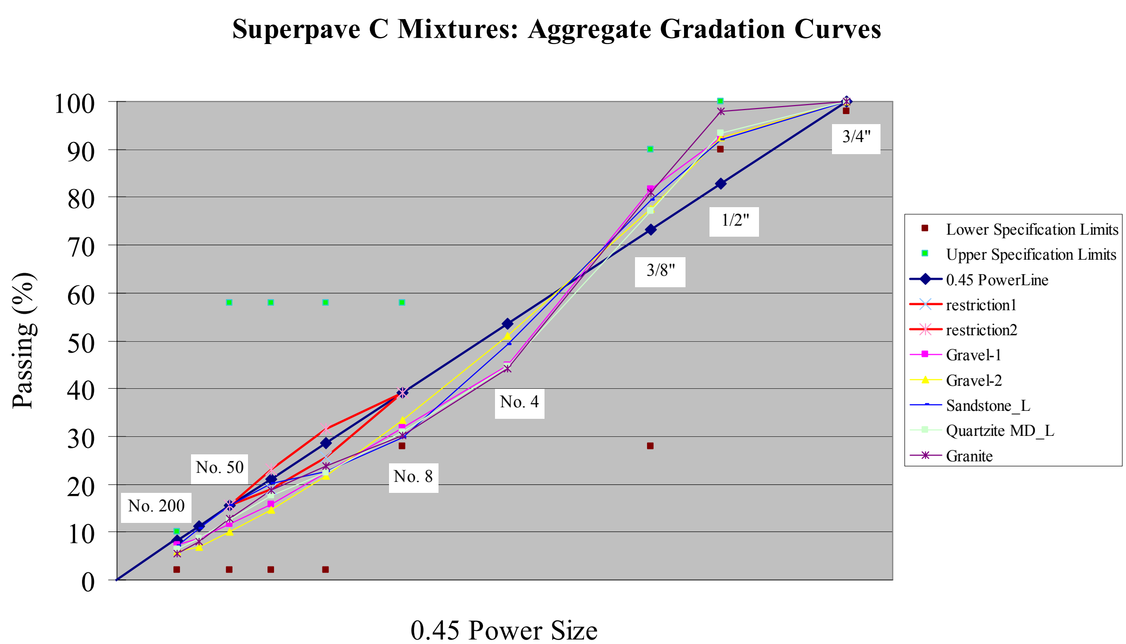


Figure 33. Aggregate Gradation Curves of Superpave C Mixtures.

- Asphalt content: Asphalt content varied between 4.0 and 7.0 percent in 0.5 percent increments. However, this does not mean that seven asphalt contents were investigated for each type of mixture. Instead, four asphalt contents were tested for most mixtures in Phase II. Two parameters were considered when selecting the range of asphalt contents:
    - asphalt binder PG, and
    - potential asphalt absorption by aggregates.

  For most of the mixtures molded with PG64-22 binder, the four asphalt contents tested were 4.0, 4.5, 5.0, and 5.5 percent; for mixtures with PG76-22, the asphalt contents ranged from 4.5 percent to 6.0 percent. Detailed information about asphalt contents used for each mixture is presented in Appendix C.

- Air voids/sample compaction: The SGC was used to mold the specimens for the HWTT and the OT. Although the intent was not to investigate the influence of AV contents on rutting and cracking resistance, it was found that the AV contents of the specimens varied. Therefore, the AV content was considered as an independent variable during statistical analysis. Detailed information about AV contents of each mixture is documented in Appendix C.

6.2.2 Laboratory Test Matrix

The laboratory test matrix is shown in Table 17.

Table 17. Laboratory Test Matrix.

Aggregate		Dense-Graded type D mixture		Superpave C mixture	
Type	Absorption	PG64-22	PG76-22	PG64-22	PG76-22
TXI-limestone	LA	√	√		
TCS-limestone	HA	√			
FW-limestone	IA	√			
Quartzite_MD_L	IA			√	
Sandstone_L	HA			√	
Gravel-1	LA				√
Gravel-2	LA				√
Granite	LA			√	√

Note: 1) HA=high absorptive, IA=intermediate absorptive, LA=low absorptive.
2) √=tests were conducted at that cell.

6.3 LABORATORY TESTING AND RESULTS

The same OT and HWTT as those described in Chapter 4 were conducted. The OT results and the HWTT results are presented in Appendices C and D, respectively. Note that for the OT testing, the number of cycles to failure of each sample was determined based on 93 percent reduction of maximum load recorded at the first cycle; for the HWTT, the rut depth (RD_{HWTT}) reported in Appendix D for mixtures with PG64-22 and PG76-22 binder corresponds to 10,000 and 20,000 wheel passes, respectively.

6.4 OVERLAY TESTER RESULT ANALYSES

The objective of the OT result analysis was to recommend the minimum asphalt content for a specific mixture at which it can pass the cracking resistance criterion. It is a well- known fact that many factors affect cracking resistance of HMA mixtures. These factors can be single parameters including PG, asphalt contents, bulk specific gravity, water absorption, Surface Area (SA), and number of gyratory compaction. Alternatively, factors that affect crack resistance can also be PG plus combined parameters (such as air void contents, VMA, effective binder contents by volume [VBE], asphalt absorption, SA, and FT). The PG plus combined parameters are preferred because these factors are closely related to current volumetric mixture design parameters. In this study statistical analyses were performed to identify significant factor(s). Then, regression equations to predict reflective cracking life (the number of cycles [N_{OT}] to break the sample) were developed using the "Solver" optimization technique in Excel. Finally, minimum asphalt contents were recommended for each mixture tested.

6.4.1 Statistical Analyses of the OT Results

The statistical analysis for OT results included the following sequence:

- Pearson Correlation analysis to check the correlations between dependent variable (N_{OT}) and each of the independent variables, and the correlations for each pair of independent variables (AV contents, VMA, VBE, asphalt absorption, FT, and SA);
- Analysis of Covariance (ANCOVA) to determine the factors having significant effect on crack resistance of HMA mixtures. ANCOVA is an extension of "analysis of variance" (ANOVA). Note that ANOVA can only be used to assess the effects of categorical independent factors. However, the OT results contain a categorical factor (PG) and other continuous factors, such as AV contents, VMA, VBE, SA, and FT. Thus, ANCOVA was used instead of ANOVA.

All of the statistical analyses were conducted using a statistics software program called *JMP* which is a SAS product (*90*).

6.4.1.1 Pearson Correlation analysis

Table 18 lists the results of the Pearson Correlation analysis. Note that a Pearson correlation coefficient (*r*) describes a linear relationship (in terms of strength and direction) between two variables. Statistically, Pearson Correlation coefficients can range from -1.00 to +1.00. The value of -1.00 represents a perfect *negative* correlation, while a value of +1.00 represents a perfect *positive* correlation. A value of 0.00 represents a lack of correlation. In terms of interpreting the OT results, a negative (-) sign implies that the variable (material property) is inversely related to N_{OT}, e. g., an increase in asphalt absorption will cause N_{OT} to decrease. For a positive (+) sign, it means that the variable (material property) and N_{OT} are linearly and proportionally related to each other, e.g., an increase in FT will cause a proportional increase in N_{OT}. A Pearson Correlation coefficient of zero means that the variable (material property) has no influence on N_{OT} or N_{OT} is independent of that variable (material property), which is not the case for the variables evaluated in Table 18.

Table 18. Pearson Correlation Coefficients.

Variables	N_{OT}	Air void content	VMA	VBE	Asphalt absorption	SA	FT
N_{OT}	1.00						
AV contents	0.11	1.00					
VMA	0.62	0.69	1.00				
VBE	0.72	-0.12	0.63	1.00			
Asphalt absorption	-0.29	-0.41	-0.67	-0.49	1.00		
SA	-0.20	-0.65	-0.62	-0.17	0.72	1.00	
FT	0.68	0.37	0.87	0.80	-0.73	-0.69	1.00

As can be seen from Table 18, there is no single factor that shows a very strong linear relationship ($r > 0.9$) with N_{OT}. This observation simply indicates that more than one parameter is needed to accurately predict N_{OT}. However, both VBE and FT, as illustrated in Table 18, do show relatively high correlations with N_{OT}. Both variables are potentially significant factors for crack resistance. Additionally, Table 18 also presents correlations between independent variables. The variables with higher correlations are shown below:

- VBE and FT ($r = 0.80$);
- FT and VMA ($r = 0.87$); and
- asphalt absorption and FT ($r = -0.73$).

These higher correlations indicate that it is reasonable to choose only one rather than two variables during ANCOVA in the next section. For example, only VBE, rather than both VBE and FT, is chosen for ANCOVA. The same rule was applied to other highly related variables. Based on the above observations, two groups of variables were recommended to run ANCOVA.

- Group 1: PG, VBE, AV contents, VMA, asphalt absorption, and SA.
- Group 2: PG, FT, AV contents, and SA.

6.4.1.2 Analysis of covariance

The purpose of ANCOVA is to determine which factors significantly affect cracking resistance. Several linear models of OT results with the two groups of variables selected above were first explored. It needs to be noted that not all three volumetric variables: AV contents, VMA, and VBE could be included in the model at the same time because of the known linear relationship (VMA = VBE - AV) among these three variables. VMA was excluded because 1) the Pearson Correlation coefficients (see Table

18) indicate that VBE is highly related to the OT results; and 2) AV contents is a volumetric parameter most commonly used during HMA mixture design. The results from applying those linear models suggested that one of the underlying assumptions (a constant variance assumption) for ANCOVA was violated. To solve this problem, a natural log transformation was applied to the dependent variable: N_{OT}. Log linear models for OT results were then explored with $\ln(N_{OT})$ as a dependent variable and various subsets of independent variables.

Table 19 shows the list of models used for determining significant factors for cracking resistance. The significant levels of different variables determined from ANCOVA are shown in Table 20. Note that in this analysis, if $\alpha \leq 0.05$, the variable is considered significant. The following observations can be seen from Table 20.

1. PG, VBE, and FT are significant at $\alpha = 0.05$. These factors must be considered regardless of HMA mixture design or model development.
2. The influence of AV contents on cracking resistance of HMA mixtures, compared to other factors including PG, VBE, and FT is not statistically significant at $\alpha = 0.05$. Thus, the AV contents will not be included in the prediction model of OT results being developed.
3. Asphalt absorption is not a significant factor at $\alpha = 0.05$ here, although it was discovered in Chapter 4 to have a huge impact on crack resistance of HMA mixtures. The reason for this observation is that the factor, VBE, has taken the asphalt absorption into account.
4. SA is not significant at $\alpha = 0.05$ in Group 1 variables, but it is significant in Group 2 variables. The reason for the reverse observation is that FT is defined as the ratio of effective volume of asphalt binder to surface area of the combined aggregates. Thus, SA is significant if FT is chosen. Otherwise, the SA is not significant.

Table 19. Models for ANCOVA.

Group	Factors included	Specific model forms
1	PG, air void content, asphalt absorption, VBE, SA	ln(OT results) = a1 + a2 * I(PG64-22 or PG76-22) + a3 * VBE + a4 * AV contents + a5 * asphalt absorption + a6 * SA Where a1-a6 are coefficients; I is an indicator function. I(PG64-22) = 1 only when the PG is PG64-22; otherwise I = 0. The same thing is true for I(PG76-22).
2	PG, air void content, SA, FT	ln(OT results) = a1+ a2 * I(PG64-22 or PG76-22) + a3 * FT + a4 * AV contents + a5 * SA Where a1-a5 are coefficients; I is an indicator function. I(PG64-22) = 1 only when the PG is PG64-22; otherwise I = 0. The same thing is true for I(PG76-22).

Table 20. ANCOVA Results.

Model type	Variables	Range of variables	F ratio	Level of significance
1	**PG**	**PG64-22, PG76-22**	**2.94**	**0.0575**
	Air void contents (%)	0.10-8.75	2.55	0.1137
	Asphalt absorption (%)	0.07-1.37	3.34	0.0704
	SA (m^2)	4.43-7.18	0.36	0.5498
	VBE (%)	6.93-14.16	**168.35**	**<0.0001**
2	**PG**	**PG64-22, PG76-22**	**5.31**	**0.0064**
	Air void contents (%)	0.10-8.75	1.38	0.2435
	SA (m^2)	**4.43-7.18**	**38.64**	**<0.0001**
	FT (microns)	**5.24-13.67**	**209.52**	**<0.0001**

Note: 1) Significant factors are shown in bold.
2) PG in Model 2 is at the borderline of p-value = 0.05.

6.4.2 Development of Prediction Model for N_{OT}

The purpose of developing a model to predict the N_{OT} is to determine the minimum asphalt content of an HMA mixture at which the HMA mixture can pass the cracking resistance criterion. It was found from the above analysis that no single factor (or variable) shows a very strong linear relationship with the N_{OT} (Pearson Correlation), that PG, VBE, FT, and SA have significant influence on the N_{OT} (or crack resistance of HMA mixtures). Thus, the prediction model being developed must contain at least two of the above significant factors. Similar to selection of variables during ANCOVA, two options are available to choose prediction model parameters among the significant factors: 1) PG and VBE or 2) PG, FT, and SA. The second option was chosen based on the following reasons:

1. The minimum asphalt content being determined is not the final asphalt content but a baseline for selecting trial asphalt contents before making any samples.
2. Table 21 lists the parameters required for estimating the asphalt contents for choosing the VBE or the FT and the SA. Obviously, it is easy and simple to estimate the asphalt contents based on the FT and SA.

Table 21. Parameters Required to Estimate Asphalt Contents.

Known parameter	Parameters required
VBE	Air void contents, specific gravity of asphalt binder, bulk specific gravity of aggregates, and asphalt absorption by aggregates
FT and SA	Specific gravity of asphalt binder and asphalt absorption by aggregates

Furthermore, since PG is a categorical variable, a specific model has to be developed for each PG. Table 22 lists the number of OT tests for HMA mixtures with each asphalt binder.

Table 22. Number of OT Tests for HMA Mixtures.

HMA mixtures	PG64-22	PG76-22
Number of OT Tests	58	34

Different forms of prediction models were explored to fit the measured OT results using the "Solver" optimization technique in Excel by minimizing the sum of squares due to error (SSE) between the measured OT results and the predicted OT results. Finally, the following exponential model showed the best fit:

$$N_{OT} = a_1 \exp\left(a_2 * FT\right) * SA^{a_3} \qquad (6)$$

where:

N_{OT} = number of cycles to break the OT sample,
FT = film thickness (microns),
SA = surface area (m^2), estimated from aggregate gradation and surface area factors recommended by Hveem (*32*), and
a_{1-3} = regression coefficients.

The goodness of fit results are illustrated in Figures 34 and 35 for HMA mixtures with PG64-22 binder and PG76-22 binder, respectively. Both types of mixtures showed high R^2 values (>0.85). The corresponding prediction models are presented in the following equations.

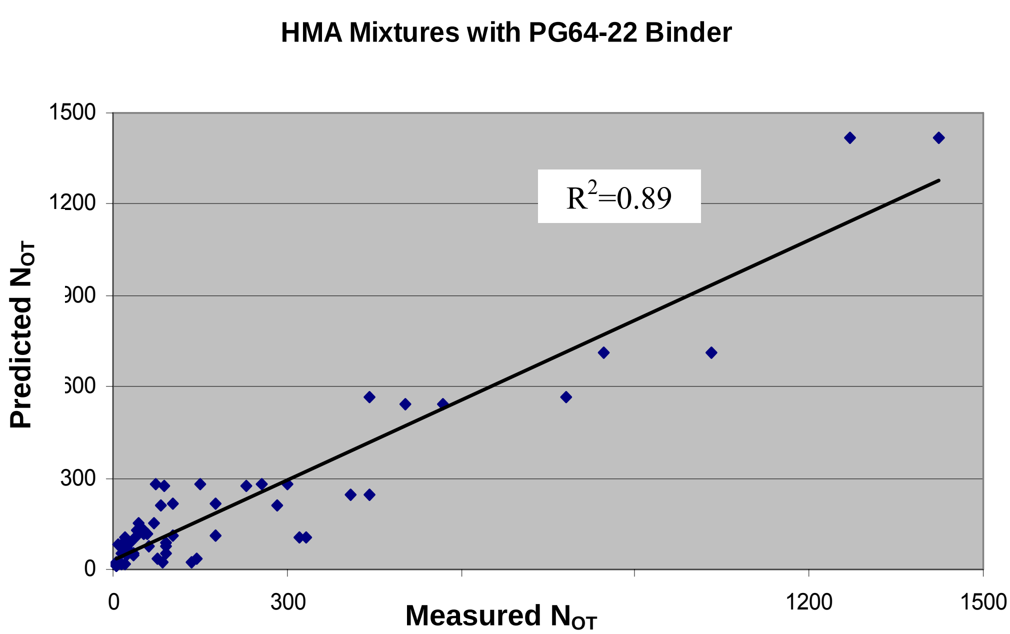


Figure 34. Measured vs. Predicted N_{OT} for HMA Mixtures with PG64-22 Binder.

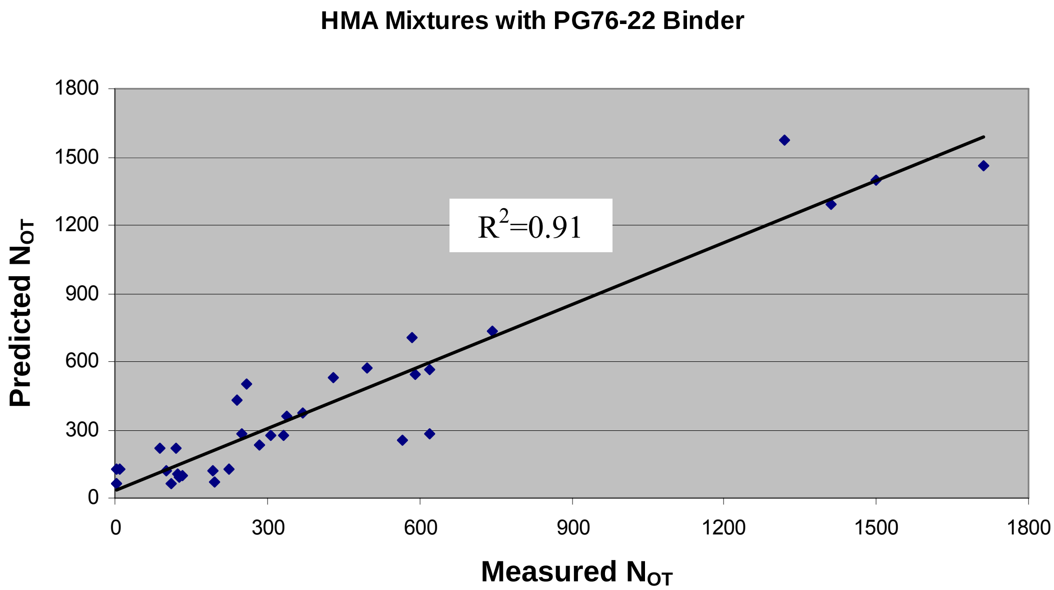


Figure 35. Measured vs. Predicted N_{OT} for HMA Mixtures with PG76-22 Binder.

HMA Mixtures with PG64-22 binder: $N_{OT} = 9.7936 \times 10^{-6} \exp(0.7743 * FT) * SA^{5.6400}$ (7)

HMA Mixtures with PG76-22 binder: $N_{OT} = 2.6659 \times 10^{-5} \exp(0.7520 * FT) * SA^{5.0646}$ (8)

Table 23 is prepared to check the reasonableness of the above equations. As expected, increasing the FT (correspondingly increasing asphalt contents) significantly improves the crack resistance of HMA mixtures, regardless of the PG. Compared to HMA mixtures molded with PG64-22 and PG76-22 binders, HMA mixtures with PG64-22 binder have better crack resistance than those with PG76-22 binder.

Table 23. Predicted N_{OT} for HMA Mixtures.

FT (Microns)		8	9	10	11	12	13
SA = 6.5 m^2	PG64-22	185	400	868	1884	4086	8863
	PG76-22	143	304	644	1366	2898	6148

6.4.3 Minimum Asphalt Contents Recommendation

Minimum asphalt contents of an HMA mixture required to pass the crack resistance criterion (300 cycles) can be estimated based upon the above N_{OT} prediction equations. It is apparent that the minimum FT can be determined from the above regression equations (7 and 8), if the SA is known (note that $N_{OT} = 300$). Then, the minimum asphalt content can be estimated based on the relationship between FT and asphalt contents. It is clear that the key is to derive a detailed relationship between FT and asphalt contents. The derivation of this relationship is presented as follows:

Known: FT, SA, asphalt absorption (P_{ba}), and specific gravity of asphalt binder (G_b).
Unknown: asphalt content (P_b).

The basic equation for calculating the FT is given in reference *62*. An alternative expression of the FT (microns) is presented in Equation 9.

$$FT = \frac{\frac{P_{be}}{G_b}}{SA * P_s} \times 1000 \tag{9}$$

where:

FT = average film thickness (microns);
P_b = asphalt content, percent by total mass of mixture;
P_{be} = effective asphalt content, percent by total mass of mixture;
P_{be}/G_b = effective volume of asphalt binder;
SA = surface area of the aggregates (m^2/kg), estimated based on aggregate gradation and surface area factors proposed by Hveem (*32*); and
P_s = aggregate content (= 100 - P_b), percent by total mass of mixture.

From Equation 9, we have Equation 10.

$$P_{be} = \frac{FT * SA * G_b}{1000} * P_s \tag{10}$$

Based on the definition of effective asphalt content of an HMA mixture, we have Equation 11 (*32*).

$$P_{be} = P_b - \frac{P_{ba}}{100} * P_s \tag{11}$$

Since Equations 10 and 11 are equal, we have Equation 12.

$$P_{be} = \frac{FT * SA * G_b}{1000} * P_s = P_b - \frac{P_{ba}}{100} * P_s \tag{12}$$

Since $P_s = 100 - P_b$, Equation 12 can be simplified step by step as follows:

$$\frac{FT * SA * G_b}{1000} * (100 - P_b) = P_b - \frac{P_{ba}}{100} * (100 - P_b) \tag{13}$$

$$\frac{FT * SA * G_b}{1000} * 100 - \frac{FT * SA * G_b}{1000} * P_b = P_b - \frac{P_{ba}}{100} * 100 + \frac{P_{ba}}{100} * P_b \tag{14}$$

$$\frac{FT * SA * G_b}{10} + P_{ba} = P_b + \frac{P_{ba}}{100} * P_b + \frac{FT * SA * G_b}{1000} * P_b \tag{15}$$

$$\frac{FT * SA * G_b + 10 * P_{ba}}{10} = \left(1 + \frac{P_{ba}}{100} + \frac{FT * SA * G_b}{1000}\right) * P_b \tag{16}$$

$$\frac{FT * SA * G_b + 10 * P_{ba}}{10} = \left(\frac{1000 + FT * SA * G_b + 10 * P_{ba}}{1000}\right) * P_b \tag{17}$$

$$P_b = \frac{FT * SA * G_b + 10 * P_{ba}}{10} * \left(\frac{1000}{1000 + FT * SA * G_b + 10 * P_{ba}}\right) \tag{18}$$

Finally, the following equation to calculate asphalt content of an HMA mixture is deduced.

$$P_b = \frac{100 * FT * SA * G_b + 1000 * P_{ba}}{1000 + FT * SA * G_b + 10 * P_{ba}} \tag{19}$$

6.4.3.1 Minimum asphalt contents for HMA mixtures with PG64-22 binder

Using the above equations, an example is presented to demonstrate how to use these equations to estimate the minimum asphalt content for a paving mixture with PG64-22 binder.

Assumptions: N_{OT} = 300, SA = 5.0 m^2/kg, P_{ba} = 0.5 % and G_b = 1.025.

- ***Step 1: determine the minimum FT***

Since this mixture was molded with PG64-22 binder, Equation 7 should be used for calculation of the minimum FT. Take the natural log of Equation 7, and Equation 7 becomes Equation 20.

$$\ln(\mathrm{N_{OT}}) = \ln(9.7936) - 6.0 * \ln(10) + 0.7743 * FT + 5.6400 * \ln(SA) \tag{20}$$

Let N_{OT} = 300 and SA = 5.0, then, Equation 20 becomes:

$$\ln(300) = \ln(9.7936) - 6.0 * \ln(10) + 0.7743 * FT + 5.6400 * \ln(5) \tag{21}$$

$$8.1603 = 2.2817 - 6.0 * 2.3026 + 0.7743 * FT + 5.6400 * 1.6094 \tag{22}$$

Finally, the minimum FT calculated from Equation 22 is 10.54 microns.

- ***Step 2: determine the minimum asphalt contents***

With known FT, SA, G_b, and P_{ba}, the minimum asphalt contents can be determined from Equation 19. The detailed calculation is presented as follows:

$$P_b = \frac{100 * 10.54 * 5.0 * 1.025 + 1000 * 0.5}{1000 + 10.54 * 5.0 * 1.025 + 10 * 0.5} = 5.57\,(\%) \tag{23}$$

The same steps as presented above were used to calculate the minimum asphalt contents for other SA and asphalt absorption values. The results are listed in Table 24 and the corresponding plot is shown in Figure 36. Note that a G_b value of 1.025 was assumed for producing Table 24.

Table 24. Estimated Minimum Asphalt Contents for HMA Mixtures with PG64-22 Binder.

Surface area (m^2/kg)	Asphalt absorption (%)						
	0.00	0.25	0.50	0.75	1.00	1.25	1.50
4.5	4.96	5.18	5.41	5.63	5.85	6.07	6.29
5.0	5.12	5.35	5.57	5.79	6.02	6.24	6.46
5.5	5.26	5.48	5.70	5.93	6.15	6.37	6.59
6.0	5.36	5.58	5.81	6.03	6.25	6.47	6.69
6.5	5.44	5.66	5.88	6.10	6.32	6.54	6.76
7.0	5.48	5.71	5.93	6.15	6.37	6.59	6.81
7.5	5.51	5.73	5.95	6.17	6.39	6.61	6.83

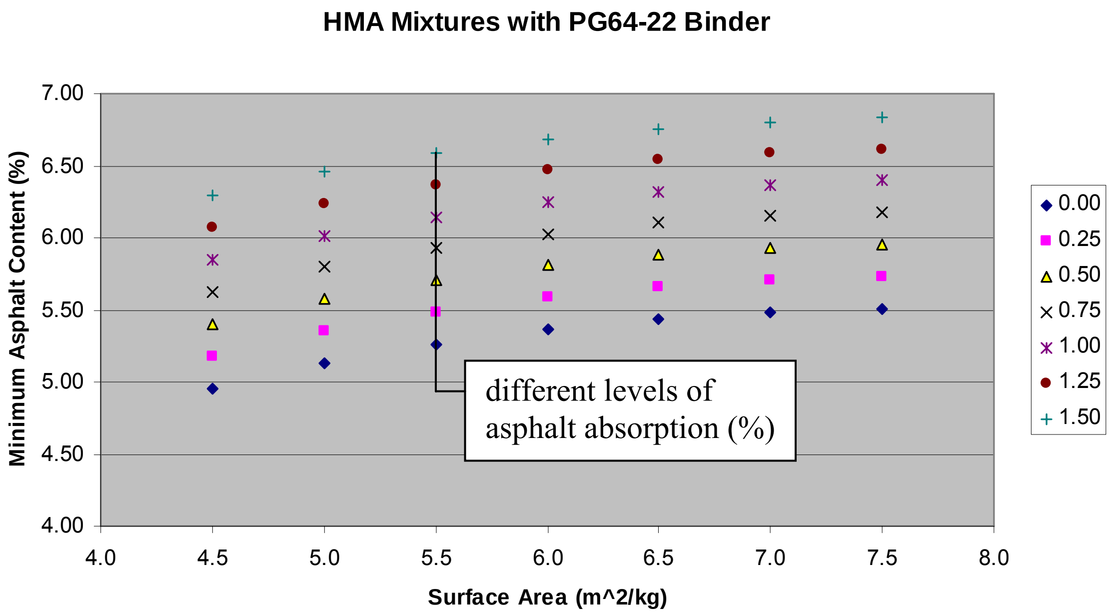


Figure 36. Estimated Minimum Asphalt Contents for HMA Mixtures with PG64-22 Binder.

6.4.3.2 Minimum asphalt contents for HMA mixtures with PG76-22 binder

Using the same procedures as those used for HMA mixtures with PG64-22 binder, the minimum asphalt contents were estimated for HMA mixtures with PG76-22 binder. The only difference is that Equation 8 rather than Equation 7 was used to determine the minimum FT. The minimum asphalt contents are shown in Table 25 and Figure 37.

Table 25. Estimated Minimum Asphalt Contents for HMA Mixtures with PG76-22 Binder.

Surface area (m^2/kg)	Asphalt absorption (%)						
	0.00	0.25	0.50	0.75	1.00	1.25	1.50
4.5	5.07	5.29	5.52	5.74	5.96	6.18	6.40
5.0	5.27	5.49	5.72	5.94	6.16	6.38	6.60
5.5	5.44	5.66	5.89	6.11	6.33	6.55	6.76
6.0	5.58	5.81	6.03	6.25	6.47	6.68	6.90
6.5	5.70	5.92	6.14	6.36	6.58	6.80	7.01
7.0	5.79	6.01	6.23	6.45	6.67	6.89	7.10
7.5	5.86	6.08	6.30	6.52	6.74	6.96	7.17

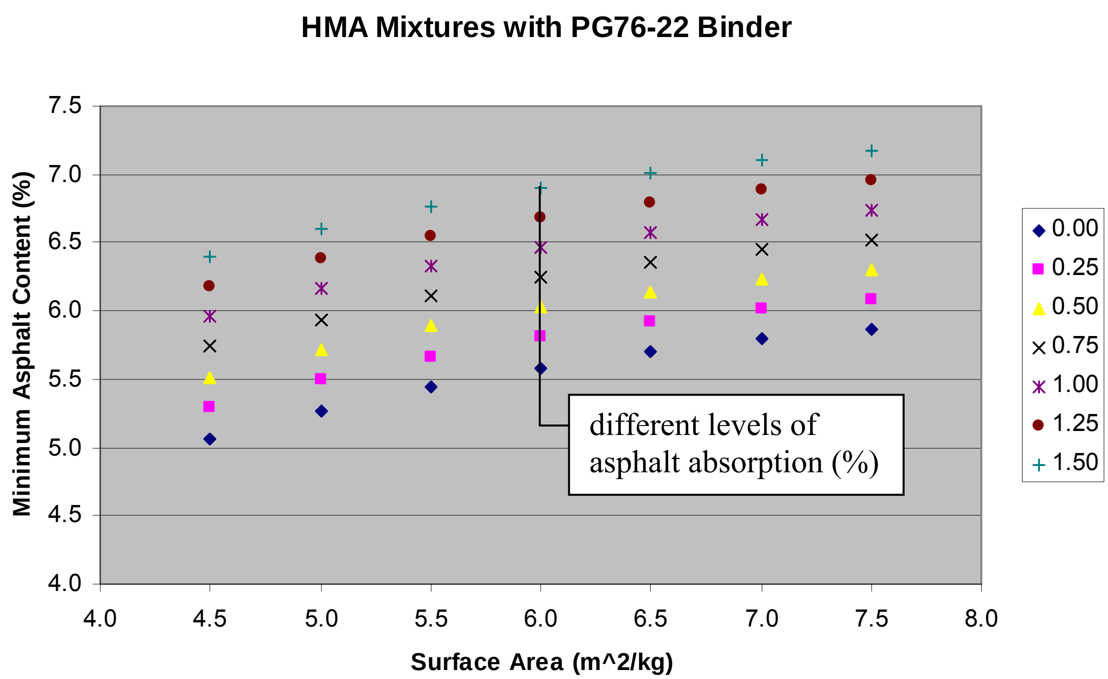


Figure 37. Estimated Minimum Asphalt Contents for HMA Mixtures with PG76-22 Binder.

6.4.4 Preliminary Verification of the Recommended Minimum Asphalt Content

The most recent experimental asphalt overlay sections available for study related to this project are 100 mm (4 in) asphalt overlays over a continuous reinforced concrete pavement on IH20, Atlanta, Texas. Three types of HMA mixtures and three types of aggregates − total of nine HMA mixtures − were placed on IH20. One PG76-22 asphalt binder was used in all nine test sections. Two months after opening to traffic, reflective cracking was observed in seven test sections; after one year of service, reflective cracking appeared on all nine test sections (*91*). The asphalt overlay mixture design information is briefly tabulated in Table 26. As can be seen in Table 26, the asphalt contents of the IH20 mixtures are much less than the recommended minimum asphalt contents for meeting the cracking resistance requirement. Thus, the poor cracking resistance of the IH20 mixtures should not be surprising. Certainly, more field test sections are needed to further validate the results in Tables 24 and 25.

Table 26. Asphalt Mixtures Information on IH20 Experimental Sections.

Section no.	Mixture type	Aggregate type	In-place		Estimated asphalt absorption (%)	Minimum asphalt contents recommended (%)
			SA (m^2/kg)	AC (%)		
1	Superpave-C	Gravel	5.25	4.55	0.35	5.67
2		Sandstone	7.24	4.90	1.37	7.00
3		Quartzite	6.88	5.10	0.63	6.30
4	CMHB-C	Gravel	6.48	4.70	0.35	6.00
5		Sandstone	7.04	4.57	1.37	7.00
6		Quartzite	5.45	4.77	0.63	5.99
7	Type-C	Gravel	6.84	4.08	0.35	6.07
8		Sandstone	8.17	4.76	1.37	7.12
9		Quartzite	7.04	4.70	0.63	6.35

Note: CMHB = coarse matrix high binder.

6.5 HAMBURG WHEEL TRACKING TEST RESULT ANALYSIS

The objective of the HWTT result analysis was to recommend a maximum asphalt content for a specific HMA mixture below which it can meet the rutting resistance criterion (12.5 mm [0.5 in]). Similar sequences as those used for the OT results analyses were followed to analyze the HWTT results. The detailed analyses are presented as follows.

6.5.1 Statistical Analyses of the HWTT Results

6.5.1.1 Pearson Correlation analysis

As noted previously, the number of passes of the HWTT is specified based on asphalt binder PG. For instance, the number of passes is 10,000 for HMA mixtures with PG64-22 binder. Thus, it is reasonable to analyze the HWTT results based upon asphalt binder PG of HMA mixtures. The same statistical software (*JMP*) was used to run the following analyses (*90*).

Table 27 lists the results of the Pearson Correlation analysis for HMA mixtures with different PGs. As noted previously, Pearson Correlation coefficients can range from -1.00 to +1.00. A value of -1.00 represents a perfect *negative* correlation, while a value of +1.00 represents a perfect *positive* correlation. A value of 0.00 represents a lack of correlation. In terms of interpreting the HWTT results, a negative (-) sign implies that the variable (material property) is inversely related to RD_{HWTT}, e. g., an increase in asphalt absorption causes RD_{HWTT} to decrease. For a positive (+) sign, it means that the variable (material property) and N_{OT} are linearly and proportionally related to each other, e.g., an increase in FT causes a proportional increase in RD_{HWTT}. A Pearson Correlation coefficient of zero means that the variable (material property) has no influence on RD_{HWTT} or RD_{HWTT} is independent of that variable (material property), which is not the case for the variables evaluated in Table 27.

Table 27. Pearson Correlation Coefficients for HMA Mixtures.

Variables	PG	RD_{HWTT}	Air void content	VMA	VBE	Asphalt absorption	SA	FT
RD_{HWTT}	PG76-22	1.00						
Air void content		0.53	1.00					
VMA		***0.80***	0.67	1.00				
VBE		0.61	0.00	0.74	1.00			
Asphalt absorption		-0.66	-0.63	-0.47	-0.07	1.00		
SA		-0.69	-0.51	-0.46	-0.16	0.81	1.00	
FT		***0.88***	0.41	0.87	0.81	-0.56	-0.68	1.0
RD_{HWTT}	PG64-22	1.00						
Air void content		0.28	1.00					
VMA		***0.69***	0.51	1.00				
VBE		0.57	-0.21	0.74	1.00			
Asphalt absorption		-0.37	-0.12	-0.44	-0.41	1.00		
SA		-0.41	-0.30	-0.55	-0.39	0.75	1.00	
FT		***0.69***	0.12	0.87	0.89	-0.57	-0.72	1.0

As can be seen in Table 27, no single factor shows a very strong linear relationship ($r > 0.9$) with RD_{HWTT}. This observation simply indicates that more than one parameter is needed to accurately predict RD_{HWTT}. However, both VMA and FT, as illustrated in Table 27, show relatively high correlations with RD_{HWTT}. Both variables are potentially significant factors for rutting resistance. In addition, Table 27 also presents correlations between independent variables. The variables with higher correlations are shown below:

- VBE and FT ($r = 0.89$ for PG64-22 and $r = 0.81$ for PG76-22);
- FT and VMA ($r = 0.87$ for both PG64-22 and PG76-22); and
- Asphalt absorption and SA ($r = 0.75$ for PG64-22 and $r = 0.81$ for PG76-22).

These higher correlations indicate that it is reasonable to choose only one rather than two variables during ANCOVA in the next section. For example, only VMA rather than both VMA and FT is chosen for ANCOVA. The same rule was applied to other highly related variables. Based on the above observations, two groups of variables were recommended to run the ANCOVA for each type of PG binder.

- Group 1: VMA, air void content, asphalt absorption, and SA.
- Group 2: FT, air void content, and SA.

6.5.1.2 Analysis of covariance

Similar variables to those for OT results were explored for the HWTT results. Table 28 shows the forms of models used for determining significant factors on the rut resistance of the HMA mixtures. The significant levels of different variables determined from ANCOVA are shown in Table 29. Note that in the analysis, the variable is considered significant if $\alpha \leq 0.05$. The following observations can be derived from Table 29.

1. Regardless of PG, VMA and FT are significant at $\alpha = 0.05$, which is consistent with the Pearson Correlation analysis. These two factors must be considered for HMA mixture design.
2. Regardless of PG, asphalt absorption is not a significant factor at $\alpha = 0.05$ here. The reason for this observation is that the variable, VMA, has taken the asphalt absorption into account.
3. The influence of air void content is relatively complex. Note that VMA (= VBE + AV) actually includes the AV content. Thus, it is reasonable that for HMA mixtures with PG64-22 binder, the AV content is not significant if PG, AV content, asphalt absorption, VMA, and SA are grouped. However, the air void content is significant at $\alpha = 0.05$ if it is the only volumetric variable included in the analysis, such as Model 2 with variables FT, SA, and AV content. For HMA mixtures with PG76-22 binder, the influence of AV content is also significant at $\alpha = 0.05$. Thus, AV content generally is a significant factor for rut resistance.
4. Whether or not the influence of SA is significant at $\alpha = 0.05$ depends on the asphalt binder PG. The SA is significant for HMA mixtures with PG76-22 binder, but is not for HMA mixtures with PG64-22 binder. One possible explanation for this observation is stripping during the HWTT. Rut depth measured during the HWTT includes both rutting and stripping. There was no stripping for the HMA mixtures with PG76-22 binder, but some of the HMA mixtures with PG64-22 binder had substantial stripping during the HWTT. This is the possible reason for this difference.

Table 28. Models for ANCOVA for Each PG.

Group	Factors included	Specific model forms
1	PG, AV content, asphalt absorption, VMA, SA	$\ln(RD_{HWTT}) = a1 + a2 * VMA + a3 * AV\ content + a3 * asphalt\ absorption + a5 * SA$ Where a1-a5 are coefficients.
2	PG, AV content, SA, FT	$\ln(RD_{HWTT}) = a1 + a2 * FT + a3 * AV\ content + a4 * SA$ Where a1-a4 are coefficients.

Table 29. ANCOVA Results for HWTT.

Model type	PG	Variables	Ranges of variables	F ratio	Level of significance
1	PG64-22	Air void contents (%)	5.02-9.29	1.05	0.311
		Asphalt absorption (%)	0.07-1.07	0.23	0.633
		SA (m^2)	4.43-6.04	0.27	0.608
		VMA (%)	**14.97-21.28**	**31.15**	**<0.000**
	PG76-22	**Air void contents (%)**	**5.29-9.95**	**6.04**	**0.018**
		Asphalt absorption (%)	0.14-0.42	2.23	0.142
		SA (m^2)	**4.43-5.87**	**21.18**	**<0.000**
		VMA (%)	**14.78-22.76**	**89.77**	**<0.000**
2	PG64-22	**Air void contents (%)**	**5.02-9.29**	**6.37**	**0.015**
		SA (m^2)	4.43-6.04	2.27	0.139
		FT (microns)	**5.84-13.39**	**44.97**	**<0.000**
	PG76-22	**Air void contents (%)**	**5.29-9.95**	**3.01**	**0.004**
		Surface area	**4.43-5.87**	**-2.48**	**0.017**
		FT	**7.28-13.62**	**8.64**	**<0.000**

Note: Significant factors are shown in bold.

6.5.2 Development of Prediction Model for RD_{HWTT}

The purpose of developing a model to predict RD_{HWTT} is to determine a maximum asphalt content level of an HMA mixture, below which it can pass the rutting resistance criteria. The above analysis indicates that no single factor (or variable) has a very strong linear relationship with RD_{HWTT} (Pearson Correlation), although AV, VBE, FT, and SA have influence on RD_{HWTT} (or rut resistance of HMA mixtures). Thus, the prediction model being developed must contain at least two of the above significant factors. Again, the variables including FT, SA, and AV contents were preferred. The reason for this preference has been explained previously. Table 30 lists the number of HWTT samples for HMA mixtures for each type of PG binder.

Table 30. Number of HWTT Samples.

HMA mixtures	PG64-22	PG76-22
Number of HWTT samples	49	54

Different forms of prediction models were explored to fit RD_{HWTT} using the "Solver" optimization technique in Excel by minimizing the SSE between the measured RD_{HWTT} and the predicted RD_{HWTT}. Finally, the following exponential model showed the best fit and was used for predicting RD_{HWTT}.

$$\mathrm{RD}_{\mathrm{HWTT}} = a_1 \exp(a_2 * FT) * SA^{a_3} * (Air\ void\ content)^{a_4} \tag{24}$$

where:

RD_{HWTT} = rut depth under the HWTT,
FT = film thickness (microns),
SA = surface area (m^2), estimated from aggregate gradation and surface area factors recommended by Hveem (*32*), and
$a_{1\text{-}4}$ = regression coefficients.

The goodness of fit results are illustrated in Figures 38 and 39 for HMA mixtures with PG64-22 binder and PG76-22 binder, respectively. Both type of mixtures showed relatively high R^2 values (≥ 0.80). The corresponding prediction models are presented in the following equations.

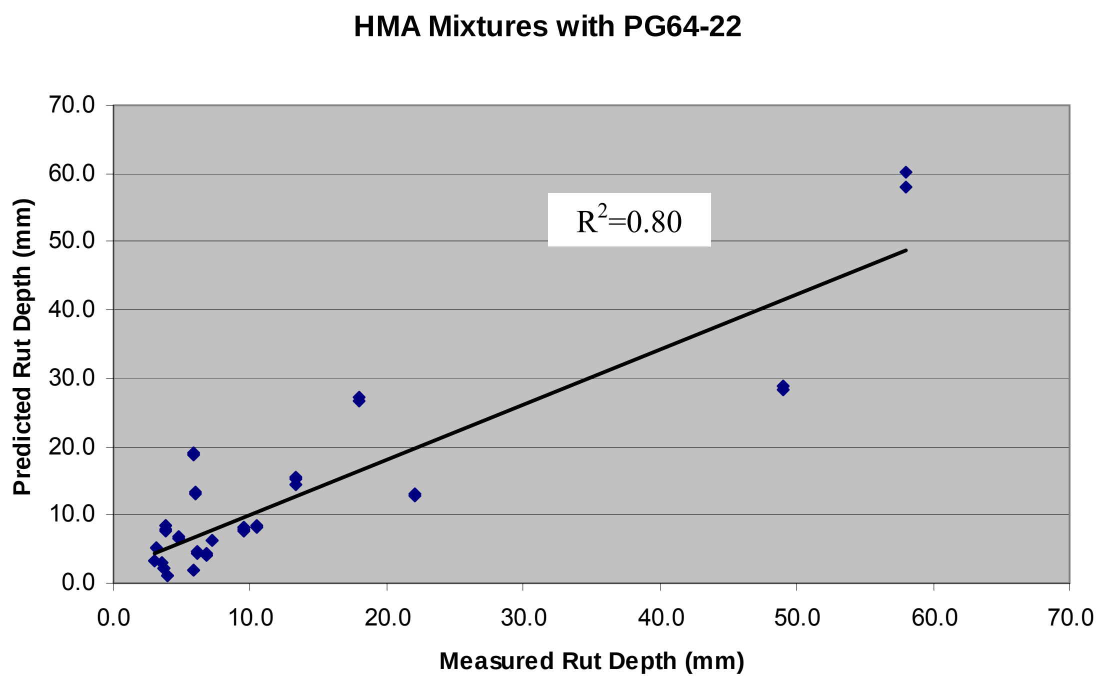


Figure 38. Measured vs. Predicted Rut Depth for HMA Mixtures with PG64-22 Binder.

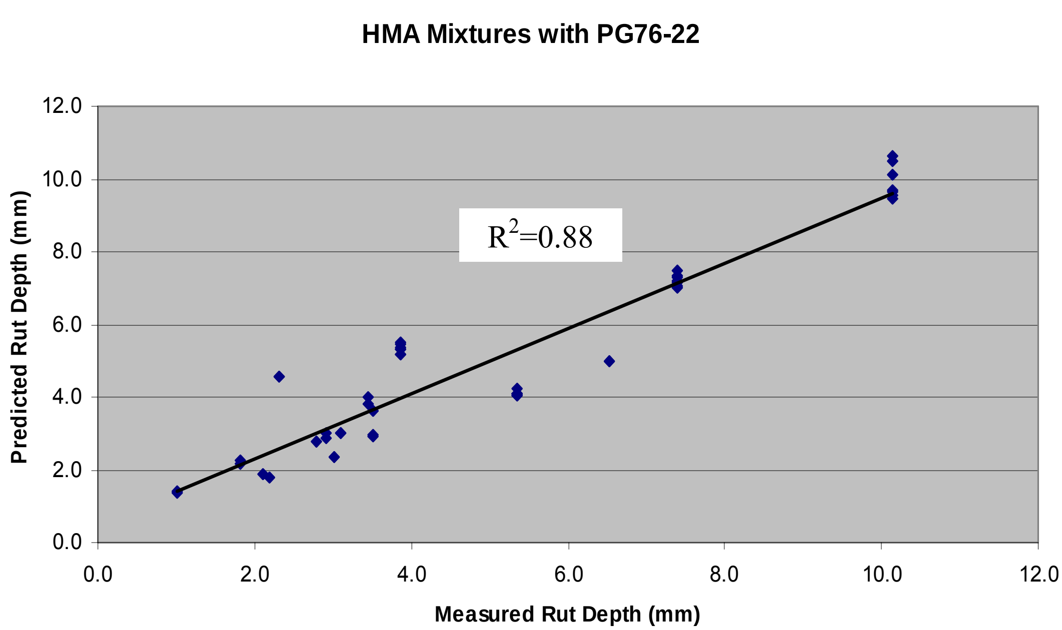


Figure 39. Measured vs. Predicted Rut Depth for HMA Mixtures with PG76-22 Binder.

HMA mixtures with PG64-22 binder:

$$\mathrm{RD_{HWTT}} = 0.00091\exp(0.57533 * FT) * SA^{1.42607} * (Air\ void\ content)^{0.58859} \tag{25}$$

HMA mixtures with PG76-22 binder:

$$\mathrm{RD_{HWTT}} = 1.12318\exp(0.24189 * FT) * SA^{-1.17934} * (Air\ void\ content)^{0.30989} \tag{26}$$

Table 31 is prepared to check the reasonableness of the above equations. As expected, increasing the FT (correspondingly increasing asphalt contents) reduces the rut resistance of HMA mixtures, regardless of PG. However, compared to HMA mixtures molded with PG64-22 binder, HMA mixtures with PG76-22 binder have considerably better rut resistance than those with PG64-22 binder. This finding from the HWTT is consistent with the field observation of the test track at the National Center for Asphalt Technology (NCAT): "*adding an additional 0.5 percent binder above optimum to the mixes produced with PG64-22 increased permanent deformation by approximately 50 percent. However, there was no increase when an extra 0.5 percent binder was added to mixes produced with PG76-22 (92)*."

Table 31. Predicted $\mathrm{RD_{HWTT}}$ (mm).

FT (microns)		8	9	10	11	12	13
SA= 6.5 m^2	PG64-22	2.8	5.0	8.9	15.8	28.2	50.0
	PG76-22	1.6	2.0	2.5	3.2	4.1	5.2

6.5.3 Maximum Asphalt Content Recommendation

Similar to minimum asphalt content, maximum asphalt content of an HMA mixture without a rutting problem can be estimated based upon the above $\mathrm{RD_{HWTT}}$ prediction equations. A similar approach used for estimating the minimum asphalt content was used to determine the maximum asphalt content.

6.5.3.1 Maximum asphalt contents for HMA mixtures with PG64-22 binder

An example is presented as follows to demonstrate how to use Equations 19 and 25 to estimate the maximum asphalt content for a paving mixture with PG64-22 binder.

Assumptions: $\mathrm{RD_{HWTT}}$ = 12.5 mm, SA = 5.0 m^2/kg, P_{ba} = 0.5 %,

AV content = 7.0 %, and G_b = 1.025.

- ***Step 1: determine the maximum FT***

  Since this mixture was molded with PG64-22 binder, Equation 25 is used to calculate maximum FT. Taking a natural log of Equation 25, Equation 25 becomes Equation 27.

$$\ln(\mathrm{RD_{HWTT}}) = \ln(0.00091) + 0.57533 * FT + 1.42607 * \ln(SA) + 0.58859 * \ln(Air\ void\ content) \tag{27}$$

  Let $\mathrm{RD_{HWTT}}$ = 12.5 mm, AV content = 7.0 %, and SA = 5.0 m^2/kg. Then, Equation 27 becomes:

$$\ln(12.5) = \ln(0.00091) + 0.57533 * FT + 1.42607 * \ln(5.0) + 0.58859 * \ln(7.0) \qquad (28)$$

Finally, the maximum FT calculated from Equation 29 is 10.59 µm.

- ***Step 2: determine the maximum asphalt content***

With known FT, SA, G_b, and P_{ba}, the maximum asphalt content can be determined from Equation 19. The detailed calculation is presented as follows:

$$P_b = \frac{100 * 10.59 * 5.0 * 1.025 + 1000 * 0.5}{1000 + 10.59 * 5.0 * 1.025 + 10 * 0.5} = 5.60\,(\%) \qquad (30)$$

The same steps as presented above were used to calculate the maximum asphalt contents for other SA and asphalt absorption values. The results are listed in Table 32, and the corresponding plot is shown in Figure 40. Note that a G_b value of 1.025 is assumed for producing Table 32.

Table 32. Estimated Maximum Asphalt Contents for HMA Mixtures with PG64-22 Binder.

Surface area (m^2/kg)	Asphalt absorption (%)						
	0.00	0.25	0.50	0.75	1.00	1.25	1.50
4.5	4.77	4.99	5.22	5.44	5.66	5.89	6.11
5.0	5.15	5.37	5.60	5.82	6.04	6.26	6.48
5.5	5.51	5.74	5.96	6.18	6.40	6.62	6.83
6.0	5.87	6.09	6.31	6.53	6.75	6.96	7.18
6.5	6.21	6.43	6.65	6.87	7.08	7.30	7.51
7.0	6.54	6.76	6.98	7.19	7.41	7.62	7.83
7.5	6.86	7.08	7.29	7.51	7.72	7.93	8.15

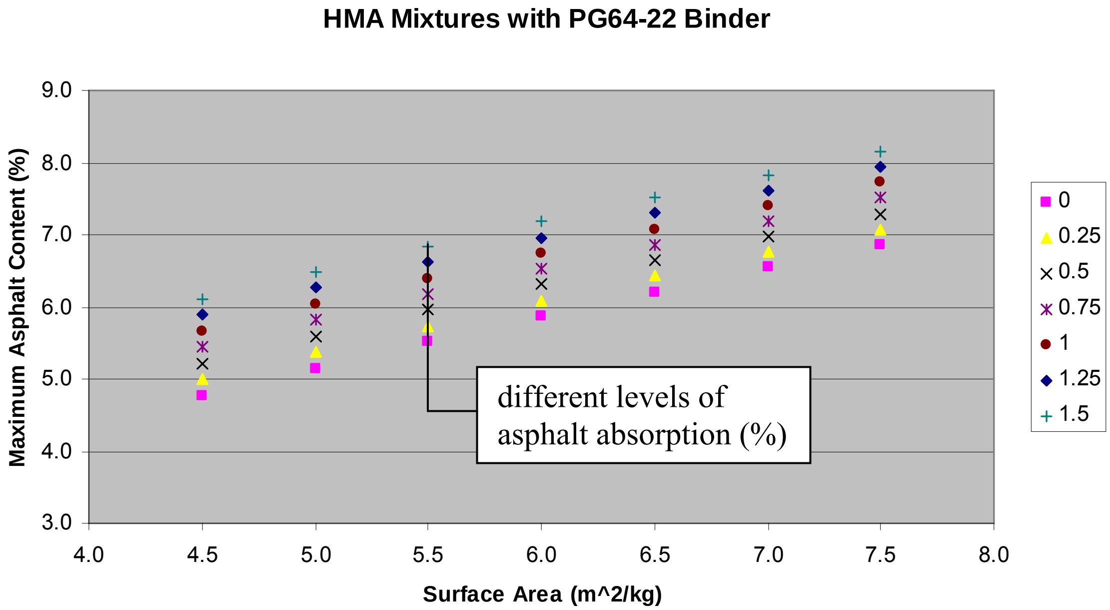


Figure 40. Estimated Maximum Asphalt Contents for HMA Mixtures with PG64-22 Binder.

6.5.3.2 Maximum asphalt contents for HMA mixtures with PG76-22 binder

Similarly, the maximum asphalt contents were estimated for HMA mixtures with PG76-22 binder. The only difference is that Equation 26 rather than Equation 25 was used to determine the maximum FT. The maximum asphalt contents are shown in Table 33 and Figure 41.

Table 33. Estimated Maximum Asphalt Contents for HMA Mixtures with PG76-22 Binder.

Surface area (m^2/kg)	Asphalt absorption (%)						
	0.00	0.25	0.50	0.75	1.00	1.25	1.50
4.5	6.39	6.61	6.83	7.04	7.26	7.47	7.69
5.0	7.28	7.49	7.71	7.92	8.13	8.34	8.55
5.5	8.17	8.38	8.59	8.80	9.00	9.21	9.42
6.0	9.06	9.27	9.47	9.68	9.88	10.08	10.29
6.5	9.96	10.16	10.36	10.56	10.76	10.96	11.16
7.0	10.85	11.04	11.24	11.44	11.63	11.83	12.02
7.5	11.73	11.93	12.12	12.31	12.51	12.70	12.89

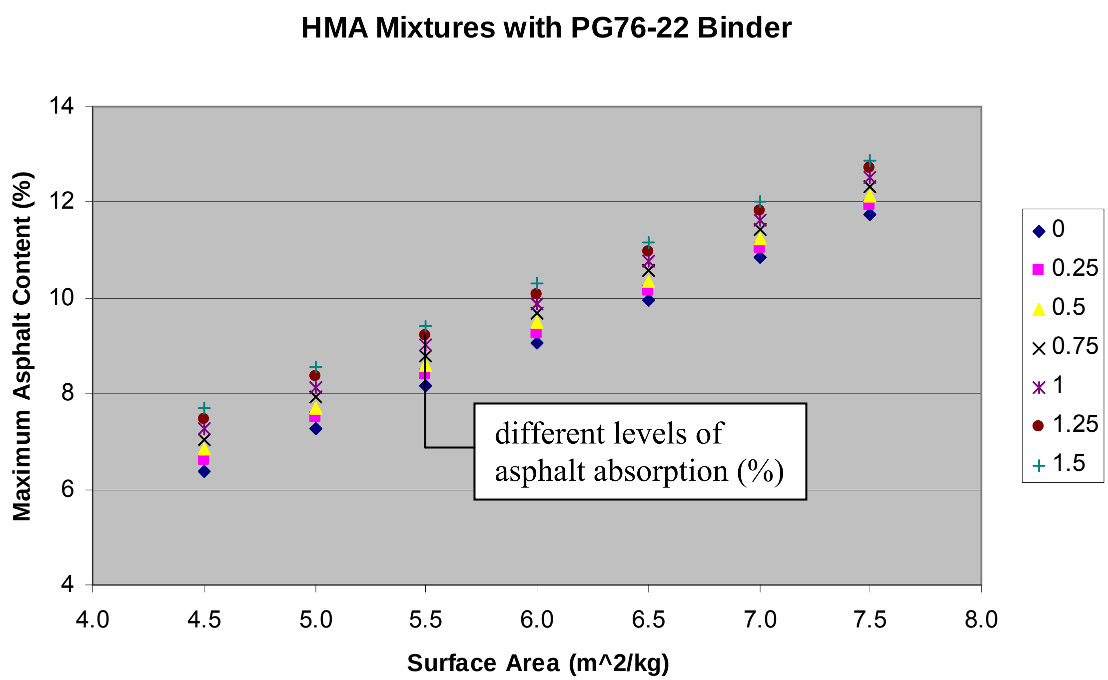


Figure 41. Estimated Maximum Asphalt Contents for HMA Mixtures with PG76-22 Binder.

6.5.4 Verification of the Recommended Maximum Film Thickness for Rutting

To verify the recommended maximum FT for rutting, available field rutting performance data from the WesTrack studies *(93)* and the NCAT test track *(92)* were pooled. Table 34 presents the results. The maximum FTs estimated from Equations 26 and 27 are also listed in Table 34. It can be seen that the mixtures with PG64-22 binder had very poor rutting resistance if their FT values were larger than the maximum FT estimated from Equation 26. The FTs of mixtures with PG64-22 binder placed on the NCAT test track are all less than the estimated maximum FTs. As expected, the rutting performance of these mixtures was generally good on the NCAT test track. Thus, the upper limit of FT for rutting is preliminarily validated for PG64-22 binders.

The maximum FT of mixtures with PG76-22 binder on the NCAT test track was 9.11 microns, which is much less than the estimated maximum FT. It is reasonable to expect that these mixtures will perform well in terms of rut resistance. As seen in Table 34, the rut depth corresponding to 9.11 microns FT of mix is only 2 mm (0.08 in) after 9 million ESALs (Equivalent Standard Axle Load). These observations indirectly validate the recommended maximum FT (or maximum asphalt content). Certainly, more field observations, especially from Texas, are needed to further check and verify the recommended maximum asphalt contents (Tables 32 and 33).

Table 34. Field Rutting Performance Data.

Site	Section	Mixture type	Binder PG	ESALs (million)	RD (mm)	Air voids (%)	SA (m^2/kg)	FT (microns)	Max. FT (microns)
WesTrack -Replacement	35	Superpave-B (BRZ)	64-22	0.58	20.0	7.60	4.59	12.0	10.72
	36				36.0	12.50	4.75	11.5	10.12
	37				26.0	8.00	4.67	11.0	10.62
	38				13.0	8.70	5.01	9.8	10.36
	39				11.0	6.20	4.65	11.2	10.89
	54				17.0	8.20	4.73	10.8	10.56
	55				21.0	4.60	4.84	11.3	11.10
	56				27.0	13.70	4.93	10.0	9.94
NCAT-Test Track	N1	Superpave-D (ARZ)	76-22	9	2.0	4.9	6.64	9.11	17.16
	N2				1.8	5.3	7.09	8.88	17.37
	N3		64-22		6.1	5.9	6.56	9.14	10.09
	N4				4.1	6.6	6.34	8.50	10.06
	N5	Superpave-C (BRZ)	64-22		5.3	6.2	6.81	6.74	9.95
	N6				3.3	5.6	6.55	6.61	10.15
	N7		76-22		1.3	6.1	6.38	8.07	16.68
	N8				0.8	5.3	6.31	7.27	16.81
	N9				0.5	5.5	7.04	6.15	17.29
	N10				1.0	5.3	6.31	7.61	16.81
	N11	Superpave-C (TRZ)	76-22		0.8	6.9	6.92	5.96	16.92
	N12	SMA-D	76-22		1.5	5.4	8.25	6.69	18.09
	N13				2.8	8.0	8.13	7.10	17.51
	S1	Superpave-C (BRZ)	76-22		1.5	5.2	5.82	8.67	16.44
	S2				0.8	6.2	7.03	7.66	17.13
	S3	Superpave-D (BRZ)	76-22		0.8	7.3	7.32	7.58	17.12
	S4	Superpave-C (ARZ)	76-22		0.8	5.7	6.71	7.75	17.01
	S5	Superpave-C (TRZ)	76-22		0.8	5.1	5.31	6.42	16.01
	S6	Superpave-C (ARZ)	64-22		2.0	7.1	7.65	6.22	9.52
	S7	Superpave-C (BRZ)	64-22		3.6	6.8	6.21	8.08	10.08
	S8	Superpave-D (BRZ)	76-22		1.3	8.2	6.87	6.19	16.66
	S9	Superpave-C (BRZ)	64-22		1.0	6.6	5.77	8.55	10.29
	S10	Superpave-C (ARZ)	64-22		2.5	6.3	7.20	7.77	9.79
	S11	Superpave-D (BRZ)	76-22		1.5	6.8	7.27	5.74	17.18
	S12	Superpave-C (TRZ)	70-28		2.0	6.1	6.53	7.17	N/A
	S13	Superpave-C (ARZ)	70-28		1.3	6.6	7.16	6.47	N/A

Note: BRZ=below the restricted zone; TRZ= through the restricted zone; ARZ= above the restricted zone.

6.6 RECOMMENDATION OF TRIAL ASPHALT CONTENTS

The purpose of recommending trial asphalt contents is to simplify Stages 2 and 3 in the balanced HMA mixture design procedure proposed in Chapter 5. The output from Stages 2 and 3 is the trial asphalt content for performance evaluation in Stage 4. If the trial asphalt contents are known, then Stages 2 and 3 may be skipped. For economic reasons, design asphalt content is always the lower limit that meets the cracking resistance requirement. Thus, it is reasonable to recommend the estimated minimum asphalt content (EMAC) for passing the cracking criteria as the baseline for the trial asphalt contents. Then, vary ± 0.4 percent from the baseline to obtain the range (note that the ± 0.4 percent was arbitrarily selected). Therefore, the recommended trial asphalt contents are as follows:

- EMAC − 0.4,
- EMAC, and
- EMAC + 0.4.

Based on the previous work on the minimum asphalt contents (Tables 21 and 22), the trial asphalt contents for different PG binders are presented in Table 35. With known trial asphalt contents for HMA mixtures with PG76-22 and PG64-22 binders, the associated trial asphalt contents for HMA mixtures with PG70-22 binder are interpolated and listed in Table 35 as well. Note that 0.5 percent asphalt absorption by aggregates is assumed for all the mixtures in Table 35. For other asphalt absorption by aggregates, the corresponding trial asphalt contents can be determined based upon the minimum asphalt contents that were tabulated in Tables 24 and 25.

Table 35. Recommended Trial Asphalt Contents for HMA Mixtures.

Surface area (m^2/kg)	PG64-22			PG70-22			PG76-22		
	Asphalt absorption = 0.5 %								
4.5	5.0	5.4	5.8	5.1	5.5	5.9	5.2	5.6	6.0
5.0	5.2	5.6	6.0	5.3	5.7	6.1	5.4	5.8	6.2
5.5	5.3	5.7	6.1	5.4	5.8	6.2	5.5	5.9	6.3
6.0	5.4	5.8	6.2	5.5	5.9	6.3	5.6	6.0	6.4
6.5	5.5	5.9	6.3	5.7	6.1	6.5	5.8	6.2	6.6
7.0	5.5	5.9	6.3	5.7	6.1	6.5	5.8	6.2	6.6
7.5	5.6	6.0	6.4	5.8	6.2	6.6	5.9	6.3	6.7

To check the accuracy of the trial asphalt contents recommended in Table 35, the minimum and maximum asphalt contents estimated based upon the HWTT and the OT results are pooled and tabulated in Table 36. Comparing Table 35 with Table 36, the recommended trial asphalt contents in Table 35 are generally reasonable. It can also be seen that for the PG76-22 binder, there is no problem to design an HMA mixture that meets the rutting and cracking criteria. For the PG64-22 binder, only when the mixture has a very low surface area (such as SA = 4.5 m^2/kg) is there a potential rutting problem. Since the PG70-22 binder is between the PG76-22 and the PG64-22 binders, it is reasonable to state that there is no problem in designing an HMA mixture with PG70-22 binder that meets both rutting and cracking requirements.

Table 36. Minimum and Maximum Asphalt Contents for Passing Rutting and Cracking Criteria.

Surface area (m^2/kg)	PG64-22		PG76-22	
	Asphalt absorption = 0.5 %			
	Min. asphalt content	Max. asphalt content	Min. asphalt content	Max. asphalt content
4.5	5.4	5.2	5.6	6.9
5.0	5.6	5.6	5.8	7.7
5.5	5.7	6.0	5.9	8.6
6.0	5.8	6.3	6.0	9.5
6.5	5.9	6.7	6.2	10.4
7.0	5.9	7.0	6.2	11.2
7.5	6.0	7.3	6.3	12.1

6.7 DISCUSSION

The previous statistical analyses clearly indicate the significant influence of the FT on rutting and cracking resistance of HMA mixtures. Currently, the FT is a calculated rather than a measured value. As discussed in Chapter 2, the only technique of measuring FT found in the literature is the digital imaging method. However, its accuracy has not been verified. As noted in Chapter 2, only three Wisconsin Superpave mixtures were explored using this technique. To verify its accuracy of measuring FT, three Dense-Graded Type D mixtures were tested using the digital imaging technique with the help of Dr. Masad. Figure 42 shows the results from the digital imaging analyses. Note that these HMA mixtures have exactly the same gradation, same asphalt binder (PG64-22), and same OAC (5.1 percent) determined from volumetric design, but variable asphalt absorption. Additionally, crack resistances of these three HMA mixtures were evaluated with the TTI OT (5). The OT results are listed in Table 37.

In Figure 42, Mixture C shows the largest FT, which is reasonable and consistent with the OT result listed in Table 37. However, it does not provide a reasonable FT measurement for Mixture A, which has the poorest crack resistance. Although Mixture B has better cracking resistance, its FT measured from the digital imaging technique is lower than that of Mixture A. These preliminary trials clearly indicate that the current digital imaging technique is not accurate enough to measure the FT.

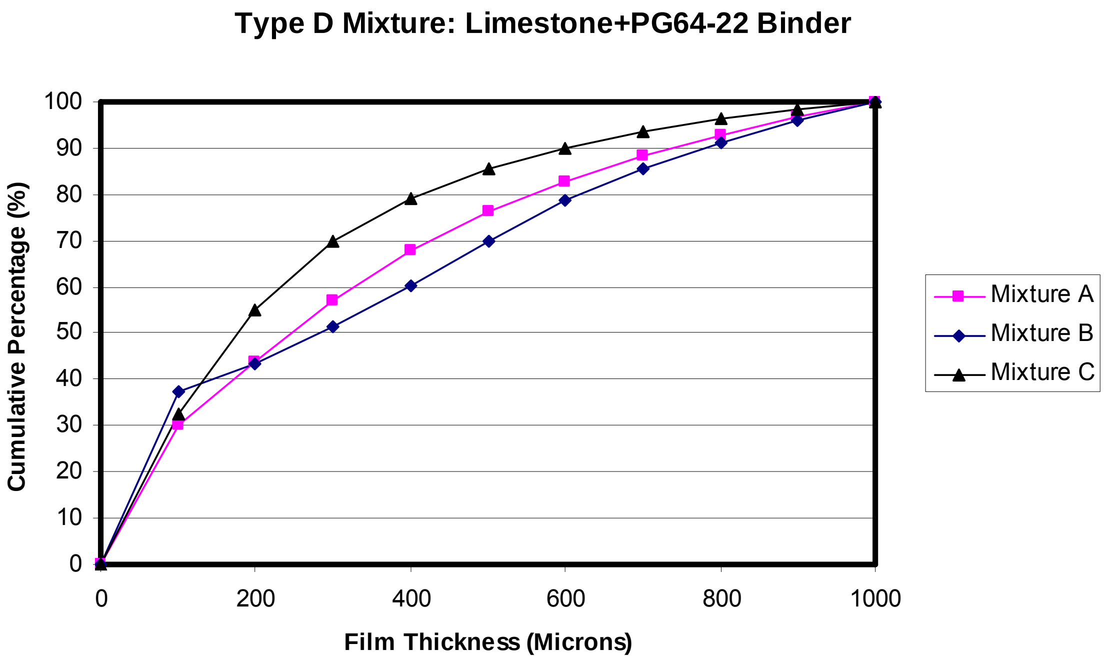


Figure 42. Measured FT Distributions for Dense-Graded Type D Mixtures A, B, and C.

Table 37. OT Results: Dense-Graded Type D Mixtures A, B, and C.

Mixture	Aggregate absorption	OT results (cycles)
A	High	4
B	Medium	90
C	Low	>750

CHAPTER 7
SIMPLIFIED VERSION OF BALANCED HMA MIXTURE DESIGN PROCEDURE

7.1 INTRODUCTION

Based on the work described in Chapters 5 and 6, a simplified version of the balanced HMA mixture design procedure is proposed in this chapter. Guidelines for each component of the simplified mixture design procedure are provided. Finally, two case studies are presented to verify and demonstrate this simplified procedure.

7.2 SIMPLIFIED VERSION OF BALANCED HMA MIXTURE DESIGN PROCEDURE

With the recommended trial asphalt contents, it becomes possible to simplify the balanced HMA mixture design procedure proposed in Chapter 5. In the simplified version of the balanced procedure shown in Figure 43, the trial asphalt contents replace Stages 2 and 3 in the balanced mixture design procedure, which is the only difference between them.

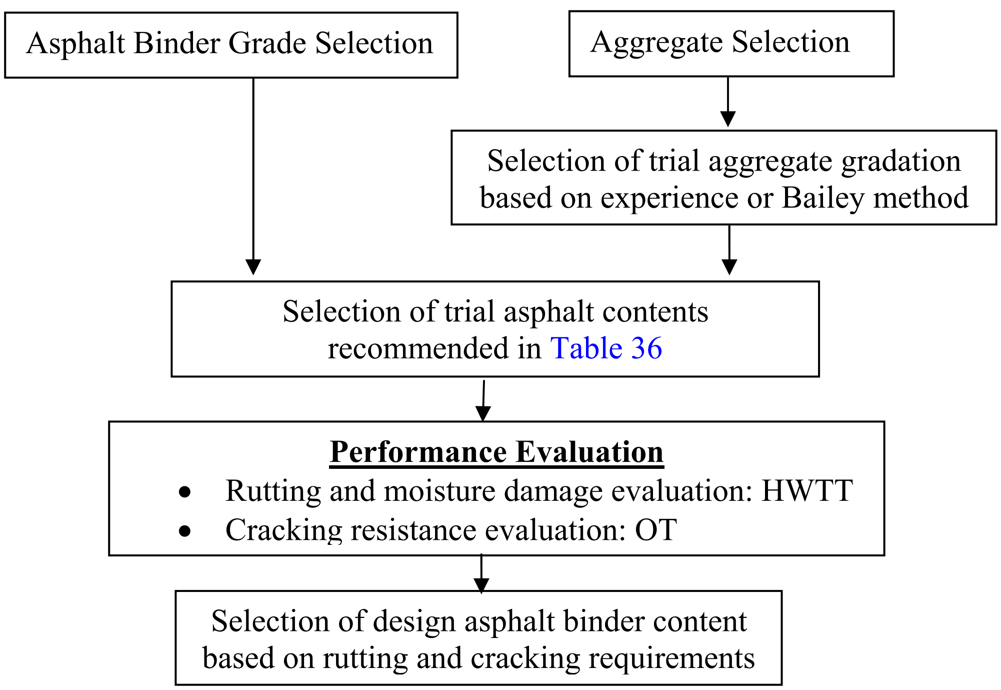


Figure 43. Simplified Version of Balanced HMA Mixture Design Procedure.

Step by step guidelines for using this simplified procedure are provided in the next sections.

7.2.1 Asphalt Binder PG Selection

Generally, asphalt binder type and grade are selected according to *TxDOT Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges* (*78*), ITEM 300, "ASPHALTS, OILS, AND EMULSIONS." Since more and more PG binders are being used, only PG binders are discussed in this chapter. The PG binders must meet certain requirements (see Table 38). High-temperature binder grade is specified at the yearly, 7-day average maximum pavement temperature, measured 20 mm (0.8 in) below the pavement surface. Low-temperature grade is specified through the yearly, 1-day minimum temperature at the pavement surface. The variability of both high- and low-temperature grading should be emphasized. The binder grade should be selected according to a desired level of reliability concept. Reliability can provide the probability that the binder will serve without substantial failure over the life of the pavement. In Texas, a computer program (PGEXCEL3.XLS) or maps showing climate grades can be used to assist in the PG selection process. In theory, the low-temperature performance (resistance to thermal cracking) is affected only by temperature (how cold does it get). This parameter is not affected by traffic levels or mixture type. However, the high-temperature performance, resistance to rutting, is affected by several traffic-related factors such as traveling speed and traffic volume. Slow moving traffic and high volume traffic may warrant an increase of one temperature grade on the high side. Additionally, mixture type may be another consideration for increasing the high-temperature portion of the binder grade. Some districts have used stiffer binders (higher high-temperature designation) to address flushing of CMHB type of mixture. Finally, when determining the appropriate base binder grade and considering possible increases to the high-temperature grade, there are economic considerations as well.

PG binder selection is a critical step for overall HMA design. However, other steps, such as aggregate selection and asphalt binder contents, are also important.

Table 38. Performance-Graded Binders (78).

Property and Test Method	Performance Grade																	
	PG 58			PG64				PG70				PG76				PG82		
	-22	-28	-34	-16	-22	-28	-34	-16	-22	-28	-34	-16	-22	-28	-34	-16	-22	-28
Average 7-day max pavement design temperature, ˚C[1]	PG 58			PG64				PG70				PG76				PG82		
Min pavement design temperature, ˚C[1]	>-22	>-28	>-34	>-16	>-22	>-28	>-34	>-16	>-22	>-28	>-34	>-16	>-22	>-28	>-34	>-16	>-22	>-28
ORIGINAL BINDER																		
Flash point, T48, Min, ˚C	230																	
Viscosity, T316:[2,3] Max. 3.0 Pa.s, test temperature, ˚C	135																	
Dynamic shear, T315:[4] G*/sin(δ), Min, 1.00 kPa Test temperature@10 rad/sec., ˚C	58			64				70				76				82		
Elastic recovery, D6084, 50 ˚F, % Min	–	–	30	–	–	30	50	–	30	50	60	30	50	60	70	50	60	70
ROLLING THIN-FILM OVEN (Tex-541-C)																		
Mass loss, Tex-541-C, Max, %	1.0																	
Dynamic shear, T315:[4] G*/sin(δ), Min, 2.20 kPa Test temperature@10 rad/sec., ˚C	58			64				70				76				82		
PRESSURE AGING VESSEL (PAV) (R28)																		
PAV aging temperature, ˚C	100																	
Dynamic shear, T315:[4] G*/sin(δ), Max, 5000 kPa Test temperature@10 rad/sec., ˚C	25	22	19	28	25	22	19	28	25	22	19	28	25	22	19	28	25	22

Table 38. Performance-Graded Binders (continued) *(78)*.

Property and Test Method	Performance Grade																	
	PG 58			PG64				PG70				PG76				PG82		
	-22	-28	-34	-16	-22	-28	-34	-16	-22	-28	-34	-16	-22	-28	-34	-16	-22	-28
Average 7-day max pavement design temperature, ˚C[1]	PG 58			PG64				PG70				PG76				PG82		
Min pavement design temperature, ˚C[1]	>-22	>-28	>-34	>-16	>-22	>-28	>-34	>-16	>-22	>-28	>-34	>-16	>-22	>-28	>-34	>-16	>-22	>-28
Creep stiffness, T313: [5, 6] S, max, 300 MPa, m-value, min, 0.300 Test temperature@60 sec., ˚C	-12	-18	-24	-6	-12	-18	-24	-6	-12	-18	-24	-6	-12	-18	-24	-6	-12	-18
Direct tension, T314: [6] Failure strain, min, 1.0% Test temperature @ 1.0 mm/min., ˚C	-12	-18	-24	-6	-12	-18	-24	-6	-12	-18	-24	-6	-12	-18	-24	-6	-12	-18

1. Pavement temperatures are estimated from air temperature using an algorithm contained in a Department-supplied computer program, may be provided by the Department or by following the procedure outlined in AAHTO MP2 and PP28.
2. This requirement may be waived at the Department's discretion if the supplier warrants that the asphalt binder can be adequately pumped, mixed, and compacted at temperatures that meet all applicable safety, environmental, and constructability requirements. At test temperatures where the binder is a Newtonian fluid, any suitable standard means of viscosity measurement may be used, including capillary (T201 or T202) or rotational viscometry (T316).
3. Viscosity at 135 ˚C is an indicator of mixing and compaction temperatures that can be expected in the lab and field. High values may indicate high mixing and compaction temperatures. Additionally, significant variation can occur from batch to batch. Contractors should be aware that variation could significantly impact their mixing and compaction operations. Contractors are therefore responsible for addressing any constructability issues that may arise.
4. For quality control of unmodified asphalt binder production, measurement of the viscosity of the original asphalt binder may be substituted from dynamic shear measurements of G*/sin (δ) at test temperatures where the asphalt is a Newtonian fluid. Any suitable standard means of viscosity measurement may be used, including capillary (T201 or T202) or rotational viscometry (T316).
5. Silicone beam molds, as described in AAHTO TP1-93, are acceptable for use.
6. If creep stiffness is below 300 MPa, direct tension test is not required. If creep stiffness is between 300 and 600 MPa, the direct tension failure strain requirement can be used instead of the creep stiffness requirement. The *m*-value requirement must be satisfied in both cases.

7.2.2 Aggregate Selection

General guidelines for aggregate selection and requirements on aggregates can be found in *TxDOT Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges* (*78*), ITEMS 341 (340), 344, and 346. For example, aggregate quality requirements for dense-graded HMA mixtures (Item 341 [340]) are listed in Table 39. In addition, another important aggregate property is (water/asphalt) absorption. Zhou and Scullion found that absorption has significant influence on cracking resistance of HMA mixtures (*5*). The higher the absorption, the poorer the cracking performance. The laboratory test results discussed in Chapters 5 and 6 further verified the influence of asphalt absorption by aggregates on cracking resistance. Also, it was found that asphalt absorption by aggregates is a long-term process. ***Further study on long-term asphalt absorption by aggregates is highly recommended.***

Table 39. Aggregate Quality Requirements (Dense-Graded HMA, Items 340, 341).

Property	Test Method	Requirement
Coarse Aggregate		
SAC	AQMP	As shown on plans
Deleterious material, %, max	Tex-217-F, Part I	1.5
Decantation, %, max	Tex-217-F, Part II	1.5
Micro-Deval abrasion, %, max	Tex-461-A	Note 1
Los Angeles abrasion, %, max	Tex-410-A	40
Magnesium sulfate soundness, 5 cycles, %, max	Tex-411-A	30[2]
Coarse aggregate angularity, 2 crushed faces, %, min	Tex 460-A, Part I	85[3]
Flat and elongated particles @ 5:1, %, max	Tex-280-F	10
Fine Aggregate		
Linear shrinkage, %, max	Tex-107-E	3
Combined aggregate[4]		
Sand equivalent, %, min	Tex-203-F	45

1. Not used for acceptance purposes. Used by the Engineer as an indicator of the need for further investigation.
2. Unless otherwise shown on the plans.
3. Unless otherwise shown on the plans. Only applies to crushed gravel.
4. Aggregates, without mineral filler, RAP, or additives, combined as used in the job-mix formula (JMF).

7.2.3 Trial Aggregate Gradation

TxDOT Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges (*78*), ITEMS 341 (340), 344, and 346 provide requirements for aggregate gradations. Tables 40, 41, and 42 present aggregate gradation requirements for dense-graded mixtures, performance-designed mixtures, and SMA, respectively. It should be noted that selection of aggregate gradation is still mainly based on experience. Alternatively, the Bailey method (*94, 95*) has recently become a popular concept for aggregate gradation, specifically for coarse aggregate gradations. This method can be used as a supplement to check the gradation selected based on past experience.

Table 40. Gradation Bands for Dense-Graded Mixtures *(78)*.

Sieve Size	A Coarse Base	B Fine Base	C Coarse Surface	D Fine Surface	F Fine Mixture
1-1/2"	98.0–100.0	–	–	–	–
1"	78.0–94.0	98.0–100.0	–	–	–
3/4"	64.0–85.0	84.0–98.0	95.0–100.0	–	–
1/2"	50.0–70.0	–	–	98.0–100.0	–
3/8"	–	60.0–80.0	70.0–85.0	85.0–100.0	98.0–100.0
#4	30.0–50.0	40.0–60.0	43.0–63.0	50.0–70.0	80.0–86.0
#8	22.0–36.0	29.0–43.0	32.0–44.0	35.0–46.0	38.0–48.0
#30	8.0–23.0	13.0–28.0	14.0–28.0	15.0–29.0	12.0–27.0
#50	3.0–19.0	6.0–20.0	7.0–21.0	7.0–20.0	6.0–19.0
#200	2.0–7.0	2.0–7.0	2.0–7.0	2.0–7.0	2.0–7.0

Table 41. Gradation Bands for Performance-Designed Mixtures *(78)*.

Sieve Size	SP-A Base	SP-B Intermediate	SP-C Surface	SP-D Fine Mixture	CMHB-C Coarse Surface	CMHB-F Fine Surface
2"	100.0	–	–	–	–	–
1-1/2"	98.0–100.0	100.0	–	–	–	–
1"	90.0–100.0	98.0–100.0	100.0	–	100.0	–
3/4"	Note 1	90.0–100.0	98.0–100.0	100.0	98.0–100.0	100.0
1/2"	–	Note 1	90.0–100.0	98.0–100.0	72.0–85.0	98.0–100.0
3/8"	–	–	Note 1	90.0–100.0	50.0–70.0	85.0–100.0
#4	–	–	–	Note 1	30.0–45.0	40.0–60.0
#8	19.0–45.0	23.0–49.0	28.0–58.0	32.0–67.0	17.0–27.0	17.0–27.0
#16	1.0–45.0	2.0–49.0	2.0–58.0	2.0–67.0	5.0–27.0	5.0–27.0
#30	1.0–45.0	2.0–49.0	2.0–58.0	2.0–67.0	5.0–27.0	5.0–27.0
#50	1.0–45.0	2.0–49.0	2.0–58.0	2.0–67.0	5.0–27.0	5.027.0
#200	1.0–7.0	2.0–8.0	2.0–10.0	2.0–10.0	5.0–9.0	5.0–9.0

Table 42. Gradation Bands for SMA *(78)*.

Master Gradation Bands (% Passing by Weight or Volume) and Volumetric Properties

Sieve Size	SMA-C Coarse	SMA-D Medium	SMA-F Fine	SMAR-C Coarse	SMAR-F Fine
3/4"	100.0	100.0	100.0	100.0	100.0
1/2"	80.0–90.0	85.0–99.0	100.0	72.0–85.0	100.0
3/8"	25.0–60.0	50.0–75.0	70.0–90.0	50.0–70.0	95.0– 100.0
#4	20.0–28.0	20.0–32.0	30.0–50.0	30.0–45.0	40.0–50.0
#8	14.0–20.0	16.0–28.0	20.0–30.0	17.0–27.0	17.0–27.0
#16	8.0–20.0	8.0–28.0	8.0–30.0	12.0–22.0	12.0–22.0
#30	8.0–20.0	8.0–28.0	8.0–30.0	8.0–20.0	8.0–20.0
#50	8.0–20.0	8.0–28.0	8.0–30.0	6.0–15.0	6.0–15.0
#200	8.0–12.0	8.0–12.0	8.0–14.0	5.0–9.0	5.0–9.0

7.2.4 Trial Asphalt Contents

As discussed in Section 6.6 of Chapter 6, the trial asphalt contents are related to surface area and asphalt absorption by aggregates. After choosing aggregate sources and aggregate gradation, surface area for each gradation can be easily estimated using Hveem's SA factors (*32*). An example of calculating the SA is presented in Appendix E. Normally, asphalt absorption by aggregates is calculated if the bulk specific gravity of aggregates is known. However, since a simple and accurate test method is currently not available for characterizing specific gravity and water absorption of aggregates, current TxDOT HMA mixture design does not calculate the asphalt absorption by aggregates. Therefore, asphalt absorption must be estimated by either historical data in the literature or past experience. If no information is available, 0.5 percent asphalt absorption can be assumed. With the known surface area and estimated asphalt absorption, trial asphalt contents for HMA mixtures with different PG binder can be chosen based upon the minimum asphalt contents that were tabulated previously in Tables 24 and 25 of Chapter 6. For the purpose of reference, Table 43 provides the recommended trial asphalt contents with three levels of asphalt absorption by aggregates: 0, 0.5, and 1.0 percent.

7.2.5 Performance Evaluation

Both the HWTT and the OT are conducted to evaluate the trial mixtures. Both tests were described in Chapter 4.

7.2.6 Selection of Balanced Asphalt Content

In contrast to the current volumetric-based methods, the design asphalt binder content in the balanced mixture design system is based on performance as measured in the laboratory rutting and cracking tests. Additionally, construction and other factors also need to be taken into account. For example, there is a ± 0.3 percent operational tolerance of asphalt content in current TxDOT *Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges* (*78*). If possible, this tolerance range should be considered when selecting an asphalt content balancing rut and crack resistance. In summary, selection of an asphalt content needs to consider at least three factors:

- rutting resistance requirement,
- cracking resistance requirement, and
- construction tolerance, if possible.

Table 43. Recommended Trial Asphalt Contents.

Asphalt absorption (%)	Surface area* (m^2/kg)	PG64-22			PG70-22			PG76-22		
0.0	4.5	4.6	5.0	5.4	4.7	5.1	5.5	4.7	5.1	5.5
	5.0	4.7	5.1	5.5	4.8	5.2	5.6	4.8	5.3	5.7
	5.5	4.9	5.3	5.7	5.0	5.4	5.8	5.0	5.4	5.8
	6.0	5.0	5.4	5.8	5.1	5.5	5.9	5.2	5.6	6.0
	6.5	5.0	5.4	5.8	5.2	5.6	6.0	5.3	5.7	6.2
	7.0	5.1	5.5	5.9	5.3	5.7	6.1	5.4	5.8	6.2
	7.5	5.1	5.5	5.9	5.3	5.7	6.1	5.9	5.9	6.3
0.5	4.5	5.0	5.4	5.8	5.1	5.5	5.9	5.2	5.6	6.0
	5.0	5.2	5.6	6.0	5.3	5.7	6.1	5.4	5.8	6.2
	5.5	5.3	5.7	6.1	5.4	5.8	6.2	5.5	5.9	6.3
	6.0	5.4	5.8	6.2	5.5	5.9	6.3	5.6	6.0	6.4
	6.5	5.5	5.9	6.3	5.7	6.1	6.5	5.8	6.2	6.6
	7.0	5.5	5.9	6.3	5.7	6.1	6.5	5.8	6.2	6.6
	7.5	5.6	6.0	6.4	5.8	6.2	6.6	5.9	6.3	6.7
1.0	4.5	5.5	5.9	6.3	5.6	6.0	6.4	5.6	6.0	6.4
	5.0	5.6	6.0	6.4	5.7	6.1	6.5	5.8	6.2	6.6
	5.5	5.8	6.2	6.6	5.9	6.3	6.7	5.9	6.3	6.7
	6.0	5.9	6.3	6.7	6.0	6.4	6.8	6.1	6.5	6.9
	6.5	5.9	6.3	6.7	6.1	6.5	6.9	6.2	6.6	7.0
	7.0	6.0	6.4	6.8	6.2	6.6	7.0	6.3	6.7	7.1
	7.5	6.0	6.4	6.8	6.2	6.6	7.0	6.3	6.7	7.1

Note: An example of surface area calculation is presented in Appendix E.

7.3 VERIFICATION AND DEMONSTRATION OF THE SIMPLIFIED VERSION OF HMA MIXTURE DESIGN PROCEDURE

Generally, mixtures with PG76-22 binder, as seen in Chapters 5 and 6, have more tolerance to asphalt content, primarily because these stiffer binders are much less susceptible to rutting. Therefore, with these binders there is potential to add more asphalt into the mixture to meet the cracking resistance requirement without losing rut resistance. However, there are potential problems for mixtures with PG70-22 or PG64-22 binder to meet rutting and cracking resistance requirements. Earlier work demonstrated that the acceptable zone of asphalt contents may be wide for a PG 76-22 binder and relatively small for the softer PG64-22 because of rutting problems at higher asphalt contents. Thus, this verification and demonstration focused on mixtures molded with either PG70-22 or PG64-22 binder.

Table 44 presents the key information about the two Superpave C mixtures, designated A and B. The HWTT and OT results are provided in Table 45. The maximum rut depth is less than 4.0 mm (0.16 in). Thus, both mixtures are very rut resistant. For the cracking requirement, the minimum asphalt contents for Mixtures A and B are around 5.9 and 6.5 percent, respectively. If construction tolerance is considered, the final asphalt contents for Mixtures A and B should be 6.2 and 6.8 percent, respectively. These two case studies show that the simplified version of HMA mixture design procedure proposed in this chapter is reasonable and useful.

Table 44. Basic Information of Trial Mixtures A and B.

Trial mixtures	Mixture A	Mixture B
Mixture type	Superpave C	Superpave C
Aggregate	Quartzite	Sandstone
Asphalt binder	PG70-22	PG64-22
SA (m^2/kg)	5.672	5.752
Estimated asphalt absorption (%)	0.5	1.2
Three trial asphalt contents (%)	5.4, 5.8, 6.2	6.0, 6.4, 6.8

Table 45. HWTT and OT Results on Trial Mixtures A and B.

Asphalt binder PG	Aggregate type	Sample no.	Trial asphalt content (%)	Estimated asphalt absorption (%)	Calculated asphalt absorption (%)	HWTT rut depth (mm)	OT (cycles)
70-22	Quartzite	1	5.4	0.5	0.82	2.3	170
		2	5.4				220
		1	5.8			2.2	230
		2	5.8				320
		1	6.2			3.3	490
		2	6.2				700
64-22	Sandstone	1	6.0	1.2	1.59	3.0	160
		2	6.0				210
		1	6.4			3.2	195
		2	6.4				260
		1	6.8			3.9	563
		2	6.8				600

Note: No. of passes of Mixture A (PG70-22 binder) is 15,000; for Mixture B it is 10,000 passes.

7.4 SUMMARY

This chapter presented a simplified version of the HMA mixture design procedure. In this simplified procedure, the volumetric-based method of selecting the OAC is replaced by a table which suggests a range of acceptable asphalt contents. The final design asphalt content is selected by balancing the conflicting demands of the HWTT and the OT. Guidelines for each component of this procedure are provided. Also, two case studies are provided to demonstrate and verify this procedure. More research is needed to study aggregate absorption, which has a significant influence on rutting and cracking resistance of HMA mixtures.

CHAPTER 8

CONCLUSIONS AND RECOMMENDATIONS

8.1 CONCLUSIONS

- A methodology of integrating the OT into the current TxDOT HMA mixture design process was developed. The OT was integrated into Stage 4: evaluate mixture properties. Eleven mixtures commonly used in Texas were designed following the current TxDOT mixture design process. Optimum asphalt contents for these 11 mixtures were determined based on 96 percent density. Rutting and cracking resistance of these 11 mixtures at optimum asphalt content were evaluated under the HWTT and OT. It was found that those dense-graded and Superpave mixtures designed following the current TxDOT mixture design method were rut resistant, but generally not crack resistant. All three SMA mixtures were both rut and crack resistant. These observations are consistent with past experience.
- A balanced HMA mixture design procedure considering rutting and cracking resistance requirements was proposed in this report. The HWTT was used to evaluate rut resistance of the HMA mixtures. Meanwhile, crack resistance of HMA mixtures was evaluated by the OT device. This balanced design procedure incorporates minor changes to the current TxDOT design procedure at Stage 4 (evaluate mixture properties). The changes include 1) employing the OT to evaluate crack resistance of mixtures, and 2) varying asphalt contents around the "optimum" asphalt content determined in Stages 2 and 3 (volumetric design). Finally, based on the HWTT and the OT results plus consideration of construction tolerance, a method of selecting a balanced asphalt content was proposed.
- Seven mixtures including dense-graded and Superpave mixtures were used to verify and demonstrate the balanced HMA mixture design procedure. For most cases, balanced asphalt contents were determined without a problem. This simply involved adding between 0.5 and 1.0 percent more asphalt to the OAC designed with existing procedures. The results demonstrate the efficiency of the proposed HMA mixture design procedure. It was also verified that aggregate absorption had significant impact on cracking and rutting resistance of HMA mixtures.
- Statistical analyses were conducted on the OT results. It was found that PG, VBE, FT, and SA have significant impact on crack resistance of mixtures. Note that the influence of asphalt absorption by aggregates was included in the VBE and FT. The influence of air void content was not significant on crack resistance.
- Similarly, statistical analyses indicated that the following factors had significant influence on rutting resistance: 1) PG, 2) VMA, 3) FT, 4) SA, and 5) air void content. Specifically, asphalt binder PG had a dominant influence

on rutting resistance. Mixtures that used a PG76-22 binder had a much better rutting resistance than those with the PG64-22 binder. This finding is consistent with the NCAT test track results and is in line with theoretical expectations.

- Based on extensive laboratory testing results, the minimum asphalt contents for different HMA mixtures to pass the cracking criteria were recommended. Similarly, the maximum asphalt contents without failing the rutting requirement were recommended. Actually, these minimum and maximum asphalt contents are the lower and upper limits of asphalt content within which HMA mixtures can meet both rutting and cracking resistance requirements. The reasonableness of these recommended limits was preliminarily verified by field performance data from IH20 in the Atlanta District, WesTrack, and NCAT test track.
- A simplified version of the balanced HMA mixture design procedure was also proposed. The simplification focused on Stages 2 and 3 (volumetric design) where an optimum asphalt content was determined at 96 percent density. Instead of volumetric design, trial asphalt contents for different mixtures were recommended for performance evaluation. Two case studies were presented to verify and demonstrate this simplified version of the balanced HMA mixture design procedure. The results indicated that this simplified version of the balanced HMA mixture design procedure is reasonable.
- It was also found that FT had considerable impact on both rutting and cracking resistance of HMA mixtures. Currently, the digital imaging technique is the only technique available to quantitatively measure FT distribution. Three mixtures with identical gradation, asphalt binder, and asphalt content were chosen to verify the reasonableness of the measured FT using this digital image technique. The measured FT distributions were not consistent with the OT test results. The poorest crack resistant mixture did not show the thinnest FT. More research is needed to accurately measure FT.

8.2 RECOMMENDATIONS

- Field experimental sections: A series of field experimental sections is needed to validate and further refine the balanced HMA mixture design procedure. Districts should be contacted to determine their willingness to place experimental sections into projects planned for this season. In this experiment TTI will redesign the existing mixture using the balanced approach presented in this report. In most cases this will simply involve increasing the design asphalt content. The district will then construct short experimental sections to compare with the proposed mixture. Samples will be taken both immediately after construction and 1 year after construction so that a laboratory evaluation can be made on the in-situ properties. Visual observations of surface condition as well as nondestructive testing will be carried out.

- Water/asphalt absorption by aggregate: Asphalt absorption by the aggregate has a tremendous influence on both rutting and cracking resistance performance of HMA mixtures. Estimated asphalt absorption is very important to correctly select the trial asphalt binder contents. A simple method is urgently needed to quickly and accurately estimate the asphalt absorption by the aggregate.

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APPENDIX A:

AGGREGATE GRADATION FOR EACH MIX

Table A1. Gradation of Limestone TXI.

BIN FRACTIONS: TXI																							
Aggregate:		Bin No.1		Bin No.2		Bin No.3		Bin No.4		Bin No.5		Bin No.6		Bin No.7									
		D-Rock		Screenings		DonnaFill																	
Individual Bin (		61.0	Percent	30.0	Percen	9.0	Percen	0.0	Percen	0.0	Percent		Percent		Percent								
Sieve Size:	Sieve Size: (mm)	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum. % Passing	Lower & Upper SP_C Specification Limits		Within Spec's	Restricted Zone SP_C			Within Spec's
1/2"	12.500	100.0	61.0	100.0	30.0	100.0	9.0	0.0	0.0	0.0	0.0					100.0	98.0	100.0	Yes				
3/8"	9.500	97.3	59.4	100.0	30.0	100.0	9.0	0.0	0.0	0.0	0.0					98.4	85.0	100.0	Yes				
No. 4	4.750	40.2	24.5	99.5	29.8	100.0	9.0	0.0	0.0	0.0	0.0					63.4	50.0	70.0	Yes				
No. 8	2.360	3.2	2.0	83.8	25.2	99.9	9.0	0.0	0.0	0.0	0.0					36.1	35.0	46.0	Yes				
No. 16	1.180	0.7	0.4	53.4	16.0	99.8	9.0	0.0	0.0	0.0	0.0					25.4							
No. 30	0.600	0.6	0.4	34.4	10.3	97.6	8.8	0.0	0.0	0.0	0.0					19.5	15.0	29.0	Yes				
No. 50	0.300	0.6	0.4	19.3	5.8	66.6	6.0	0.0	0.0	0.0	0.0					12.1	7.0	20.0	Yes				
No. 100	0.150	0.6	0.4	6.8	2.1	42.9	3.9	0.0	0.0	0.0	0.0					6.3							
No. 200	0.075	0.5	0.3	1.2	0.4	25.5	2.3	0.0	0.0	0.0	0.0					3.0	2.0	7.0	Yes				

Table A2. Gradation of Limestone TCS.

BIN FRACTIONS: TCS																							
Aggregate:		Bin No.1		Bin No.2		Bin No.3		Bin No.4		Bin No.5		Bin No.6		Bin No.7									
		KellyPit D-Rock		TCS D-Rock		TXI Field Sand		KellyPit Man Sand		TCS Man Sand													
Individual Bin (		36.0	Percent	19.0	Percen	6.0	Percen	12.0	Percen	27.0	Percent		Percent		Percent								
Sieve Size:	Sieve Size: (mm)	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum. % Passing	Lower & Upper SP_C Specification Limits		Within Spec's	Restricted Zone SP_C			Within Spec's
1/2"	12.500	100.0	36.0	100.0	19.0	100.0	6.0	100.0	12.0	100.0	27.0	0.0	0.0	0.0	0.0	100.0	98.0	100.0	Yes				
3/8"	9.500	98.7	35.5	85.3	16.2	100.0	6.0	100.0	12.0	100.0	27.0	0.0	0.0	0.0	0.0	96.7	85.0	100.0	Yes				
No. 4	4.750	28.6	10.3	14.4	2.7	99.8	6.0	99.4	11.9	99.7	26.9	0.0	0.0	0.0	0.0	57.9	50.0	70.0	Yes				
No. 8	2.360	0.9	0.3	2.2	0.4	99.8	6.0	82.9	9.9	85.6	23.1	0.0	0.0	0.0	0.0	39.8	35.0	46.0	Yes				
No. 16	1.180	0.3	0.1	1.5	0.3	99.7	6.0	53.5	6.4	59.8	16.1	0.0	0.0	0.0	0.0	28.9							
No. 30	0.600	0.2	0.1	1.4	0.3	99.1	5.9	31.9	3.8	40.7	11.0	0.0	0.0	0.0	0.0	21.1	15.0	29.0	Yes				
No. 50	0.300	0.2	0.1	1.3	0.2	84.0	5.0	17.1	2.1	24.7	6.7	0.0	0.0	0.0	0.0	14.1	7.0	20.0	Yes				
No. 100	0.150	0.2	0.1	1.2	0.2	26.4	1.6	8.6	1.0	12.7	3.4	0.0	0.0	0.0	0.0	6.3							
No. 200	0.075	0.1	0.0	1.1	0.2	7.0	0.4	6.0	0.7	8.3	2.2	0.0	0.0	0.0	0.0	3.6	2.0	7.0	Yes				

Table A3. Gradation of Sandstone-FM529, Houston.

BIN FRACTIONS: Sandstone-FM529 Houston		Bin No. 1		Bin No.2		Bin No.3		Bin No.4		Bin No.5		Bin No.6		Bin No.7		Combined Gradation						
Aggregate Source:		Jones Mill		Jones Mill		Jones Mill																
Aggregate Number:																						
Sample ID:		D-Rock		F-Rock		Screenings																
Rap?, Asphalt%:																Total Bin						
Individual Bin (%):		25.0	Percent	35.0	Percent	40.0	Percent	0.0	Percent	0.0	Percent		Percent		Percent	100.0%	Lower & Upper Type_D Specification Limits		Within Specs	Restricted Zone Type_D		Within Specs
Sieve Size:	Sieve Size: (mm)	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum. % Passing						
1/2"	12.500	99.1	24.8	100.0	35.0	100.0	40.0	100.0	0.0	100.0	0.0	0.0	0.0	0.0	0.0	99.8	98.0	100.0	Yes			
3/8"	9.500	97.0	24.3	96.5	33.8	100.0	40.0	100.0	0.0	100.0	0.0	0.0	0.0	0.0	0.0	98.0	85.0	100.0	Yes			
No. 4	4.750	10.1	2.5	34.1	11.9	97.5	39.0	99.4	0.0	99.7	0.0	0.0	0.0	0.0	0.0	53.5	50.0	70.0	Yes			
No. 8	2.360	6.9	1.7	8.9	3.1	99.8	39.9	82.9	0.0	85.6	0.0	0.0	0.0	0.0	0.0	44.8	35.0	46.0	Yes			
No. 16	1.180	4.9	1.2	5.7	2.0	69.5	27.8	53.5	0.0	59.8	0.0	0.0	0.0	0.0	0.0	31.0						
No. 30	0.600	2.7	0.7	3.1	1.1	45.0	18.0	31.9	0.0	40.7	0.0	0.0	0.0	0.0	0.0	19.8	15.0	29.0	Yes			
No. 50	0.300	2.1	0.5	2.5	0.9	29.6	11.8	17.1	0.0	24.7	0.0	0.0	0.0	0.0	0.0	13.2	7.0	20.0	Yes			
No. 100	0.150	1.9	0.5	2.1	0.7	19.2	7.7	8.6	0.0	12.7	0.0	0.0	0.0	0.0	0.0	8.9						
No. 200	0.075	1.8	0.5	1.2	0.4	12.0	4.8	6.0	0.0	8.3	0.0	0.0	0.0	0.0	0.0	5.7	2.0	7.0	Yes			

Table A4. Gradation of Superpave C Sandstone_L.

BIN FRACTIONS: SandStone With Lime																						
Aggregate:		Bin No.1 Meridian C-Rock		Bin No.2 Meridian D-Rock		Bin No.3 Meridian Screenings		Bin No.4 DonnaFill		Bin No.5 Lime		Bin No.6		Bin No.7								
Individual Bin (		22.0	Percent	56.0	Percent	13.0	Percen	8.0	Percent	1.0	Percent		Percent		Percent							
Sieve Size:	Sieve Size: (mm)	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum. % Passing	Lower & Upper SP_C Specification Limits		Within Spec's	Restricted Zone SP_C		Within Spec's
3/4"	19.000	100	22.0	100	56.0	100	13.0	100.0	8.0	100.0	1.0	0.0	0.0	0.0	0.0	100.0	98.0	100.0	Yes			
1/2"	12.500	64	14.1	100	56.0	100	13.0	100.0	8.0	100.0	1.0	0.0	0.0	0.0	0.0	92.1	90.0	100.0	Yes			
3/8"	9.500	17	3.7	96	53.8	100	13.0	100.0	8.0	100.0	1.0	0.0	0.0	0.0	0.0	79.5	28.0	90.0	Yes			
No.4"	4.750	1.0	0.2	49	27.4	97	12.6	100.0	8.0	100.0	1.0	0.0	0.0	0.0	0.0	49.3						
No. 8	2.360	0.8	0.2	20	11.2	72.8	9.5	99.9	8.0	100.0	1.0	0.0	0.0	0.0	0.0	29.8	28.0	58.0	Yes	###	39.1	Yes
No. 16	1.180	0.5	0.1	12.0	6.7	53	6.9	99.8	8.0	100.0	1.0	0.0	0.0	0.0	0.0	22.7	2.0	58.0	Yes	###	31.6	Yes
No. 30	0.600	0.4	0.1	10	5.6	42.9	5.6	97.6	7.8	100.0	1.0	0.0	0.0	0.0	0.0	20.1	2.0	58.0	Yes	###	23.1	No
No. 50	0.300	0.3	0.1	8	4.5	36.0	4.7	66.6	5.3	100.0	1.0	0.0	0.0	0.0	0.0	15.6	2.0	58.0	Yes	###	15.5	Yes
No.100	0.150	0.2	0.0	6.0	3.4	23.1	3.0	42.9	3.4	100.0	1.0	0.0	0.0	0.0	0.0	10.8						
No. 200	0.075	0.1	0.0	4	2.2	14.6	1.9	25.5	2.0	100.0	1.0					7.2	2.0	10.0	Yes			

Table A5. Gradation of Superpave C Sandstone_NL.

BIN FRACTIONS: SandStone Without Lime																						
Aggregate:		Bin No.1		Bin No.2		Bin No.3		Bin No.4		Bin No.5		Bin No.6		Bin No.7								
		Meridian C-Rock		Meridian D-Rock		Meridian Screenings		DonnaFill		Lime												
Individual Bin (		22.0	Percent	58.0	Percent	4.0	Percen	16.0	Percent	0.0	Percent		Percent		Percent							
Sieve Size:	Sieve Size: (mm)	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum. % Passing	Lower & Upper SP_C Specification Limits		Within Spec's	Restricted Zone SP_C		Within Spec's
3/4"	19.000	100	22.0	100	58.0	100	4.0	100.0	16.0	100.0	0.0	0.0	0.0	0.0	0.0	100.0	98.0	100.0	Yes			
1/2"	12.500	64	14.1	100	58.0	100	4.0	100.0	16.0	100.0	0.0	0.0	0.0	0.0	0.0	92.1	90.0	100.0	Yes			
3/8"	9.500	27	5.9	92.7	53.8	100	4.0	99.2	15.9	100.0	0.0	0.0	0.0	0.0	0.0	79.6	28.0	90.0	Yes			
No.4"	4.750	1.0	0.2	45	26.1	97	3.9	97.4	15.6	100.0	0.0	0.0	0.0	0.0	0.0	45.8						
No. 8	2.360	0.8	0.2	20	11.6	72.8	2.9	92.3	14.8	100.0	0.0	0.0	0.0	0.0	0.0	29.5	28.0	58.0	Yes	###	39.1	Yes
No. 16	1.180	0.5	0.1	12.3	7.1	53	2.1	71.1	11.4	100.0	0.0	0.0	0.0	0.0	0.0	20.7	2.0	58.0	Yes	###	31.6	Yes
No. 30	0.600	0.4	0.1	11.2	6.5	42.9	1.7	53.5	8.6	100.0	0.0	0.0	0.0	0.0	0.0	16.9	2.0	58.0	Yes	###	23.1	Yes
No. 50	0.300	0.3	0.1	10.9	6.3	36.0	1.4	41.5	6.6	100.0	0.0	0.0	0.0	0.0	0.0	14.5	2.0	58.0	Yes	###	15.5	Yes
No.100	0.150	0.2	0.0	9.6	5.6	23.1	0.9	32.7	5.2	100.0	0.0	0.0	0.0	0.0	0.0	11.8						
No. 200	0.075	0.1	0.0	8	4.6	14.6	0.6	27.9	4.5	100.0	0.0					9.7	2.0	10.0	Yes			

Table A6. Gradation of Superpave C Gravel-1.

BIN FRACTIONS: SPC_Gravel																						
Aggregate:		Bin No.1		Bin No.2		Bin No.3		Bin No.4		Bin No.5		Bin No.6		Bin No.7								
		Fordyce C-Rock		Fordyce D-Rock		Fordyce Man Sand		lime stone Screenings														
Individual Bin (		26.0	Percent	42.0	Percent	7.0	Percen	25.0	Percent		Percent		Percent		Percent							
Sieve Size:	Sieve Size: (mm)	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum. % Passing	Lower & Upper SP_C Specification Limits		Within Spec's	Restricted Zone SP_C		Within Spec's
3/4"	19.000	100	26.0	100	42.0	100	7.0	100.0	25.0	100.0	0.0	0.0	0.0	0.0	0.0	100.0	98.0	100.0	Yes			
1/2"	12.500	70.8	18.4	100	42.0	100	7.0	100.0	25.0	100.0	0.0	0.0	0.0	0.0	0.0	92.4	90.0	100.0	Yes			
3/8"	9.500	36	9.4	96.6	40.6	100	7.0	99.2	24.8	100.0	0.0	0.0	0.0	0.0	0.0	81.7	28.0	90.0	Yes			
No.4"	4.750	0.5	0.1	32	13.4	98.8	6.9	97.4	24.4	100.0	0.0	0.0	0.0	0.0	0.0	44.8						
No. 8	2.360	0.1	0.0	6	2.5	87.5	6.1	92.3	23.1	100.0	0.0	0.0	0.0	0.0	0.0	31.7	28.0	58.0	Yes	###	39.1	Yes
No. 16	1.180	0.1	0.0	1.2	0.5	60.1	4.2	71.1	17.8	100.0	0.0	0.0	0.0	0.0	0.0	22.5	2.0	58.0	Yes	###	31.6	Yes
No. 30	0.600	0.1	0.0	0.2	0.1	34.2	2.4	53.5	13.4	100.0	0.0	0.0	0.0	0.0	0.0	15.9	2.0	58.0	Yes	###	23.1	Yes
No. 50	0.300	0.1	0.0	0.2	0.1	16.0	1.1	41.5	10.4	100.0	0.0	0.0	0.0	0.0	0.0	11.6	2.0	58.0	Yes	###	15.5	Yes
No.100	0.150	0.1	0.0	0.2	0.1	4.4	0.3	32.7	8.2	100.0	0.0	0.0	0.0	0.0	0.0	8.6						
No. 200	0.075	0.1	0.0	0.2	0.1	4.2	0.3	27.9	7.0	100.0	0.0	0.0	0.0	0.0	0.0	7.4	2.0	10.0	Yes			

Table A7. Gradation of Superpave C Quartzite_MD_L.

BIN FRACTIONS: Quartzite Match Design with Lime																							
		Bin No.1		Bin No.2		Bin No.3		Bin No.4		Bin No.5		Bin No.6		Bin No.7									
Aggregate:		Marietta C-Rock		Marietta D-Rock		Maritta Screenings		DonnaFill		Lime													
Individual Bin (		18.0	Percent	47.0	Percent	26.0	Percen	8.0	Percent	1.0	Percent		Percent		Percent								
Sieve Size:	Sieve Size: (mm)	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum. % Passing	Lower & Upper SP_C Specification Limits		Within Spec's	Restricted Zone SP_C		Within Spec's	
3/4"	19.000	100	18.0	100	47.0	100	26.0	100.0	8.0	100.0	1.0	0.0	0.0	0.0	0.0	100.0	98.0	100.0	Yes				
1/2"	12.500	65	11.7	99.295	46.7	100	26.0	100.0	8.0	100.0	1.0	0.0	0.0	0.0	0.0	93.4	90.0	100.0	Yes				
3/8"	9.500	24	4.3	80.617	37.9	100	26.0	100.0	8.0	100.0	1.0	0.0	0.0	0.0	0.0	77.2	28.0	90.0	Yes				
No.4"	4.750	3.0	0.5	20	9.4	99.2	25.8	100.0	8.0	100.0	1.0	0.0	0.0	0.0	0.0	44.7							
No. 8	2.360	1	0.2	3	1.4	78.4	20.4	99.9	8.0	100.0	1.0	0.0	0.0	0.0	0.0	31.0	28.0	58.0	Yes	###	39.1	Yes	
No. 16	1.180	0.5	0.1	1.3	0.6	48.6	12.6	99.8	8.0	100.0	1.0	0.0	0.0	0.0	0.0	22.3	2.0	58.0	Yes	###	31.6	Yes	
No. 30	0.600	0.4	0.1	1.1029	0.5	31.8	8.3	97.6	7.8	100.0	1.0	0.0	0.0	0.0	0.0	17.7	2.0	58.0	Yes	###	23.1	Yes	
No. 50	0.300	0.3	0.1	1.0147	0.5	21.9	5.7	66.6	5.3	100.0	1.0	0.0	0.0	0.0	0.0	12.5	2.0	58.0	Yes	###	15.5	Yes	
No.100	0.150	0.2	0.0	0.9	0.4	15.1	3.9	42.9	3.4	100.0	1.0	0.0	0.0	0.0	0.0	8.8							
No. 200	0.075	0.1	0.0	0.75	0.4	11.5	3.0	25.5	2.0	100.0	1.0					6.4	2.0	10.0	Yes				

Table A8. Gradation of SMA C, Houston.

		BIN FRACTIONS: SMA-C, Houston																				
		Bin No.1		Bin No.2		Bin No.3		Bin No.4		Bin No.5		Bin No.6		Bin No.7								
Aggregate Source:		MartinMar		MartinMar		MartinMar																
Aggregate Number:		Mill Creek		Mill Creek		Mill Creek		Urimin														
Sample ID:		5/8"		Jones Mill D		Screenings		Mineral filler								Combined Gradation						
Rap?, Asphalt %:																Total Bin						
Individual Bin (%):		71.0	Percent	10.0	Percent	10.0	Percent	9.0	Percent		Percent		Percent		Percent	100.0%						
Sieve Size:	Sieve Size: (mm)	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum. % Passing	Lower & Upper SMA_C Specification Limits		Within Specs	Restricted Zone SMA_C		Within Specs
3/4"	19.000	100	71.0	100	10.0	100	10.0	100.0	9.0	100	0.0	100.0	0.0	0.0	0.0	100.0	100.0	100.0	Yes			
1/2"	12.500	75.6	53.7	97.1	9.7	100	10.0	100.0	9.0	100	0.0	100.0	0.0	0.0	0.0	82.4	80.0	90.0	Yes			
3/8"	9.500	36.6	26.0	78.4	7.8	100	10.0	100.0	9.0	100	0.0	100.0	0.0	0.0	0.0	52.8	25.0	60.0	Yes			
No.4"	4.750	3.9	2.8	20.9	2.1	100	10.0	100.0	9.0	100	0.0	100.0	0.0	0.0	0.0	23.9	20.0	28.0	Yes			
No. 8	2.360	2.1	1.5	3.2	0.3	83.5	8.4	100.0	9.0	83.5	0.0	100.0	0.0	0.0	0.0	19.2	14.0	20.0	Yes			
No. 16	1.180	1.9	1.3	1.8	0.2	53.5	5.4	100.0	9.0	53.5	0.0	100.0	0.0	0.0	0.0	15.9	8.0	20.0	Yes			
No. 30	0.600	1.7	1.2	1.7	0.2	35.4	3.5	100.0	9.0	35.4	0.0	100.0	0.0	0.0	0.0	13.9	8.0	20.0	Yes			
No. 50	0.300	1.5	1.1	1.6	0.2	22.8	2.3	99.9	9.0	22.8	0.0	99.9	0.0	0.0	0.0	12.5	8.0	20.0	Yes			
No. 200	0.075	1.2	0.9	1.4	0.1	12.0	1.2	68.7	6.2	12	0.0	68.7	0.0	0.0	0.0	8.4	8.0	12.0	Yes			

Table A9. Gradation of SMA D, IH635, Dallas.

		BIN FRACTIONS: SMA-D IH635 Dallas																				
		Bin No.1		Bin No.2		Bin No.3		Bin No.4		Bin No.5		Bin No.6		Bin No.7								
Aggregate Source:		Mill Creek		Mill Creek		Mill Creek		Mill Creek		Mill Creek												
Aggregate Number:		Type C		5/8 Chips		Grade 4		Screenings		Crusher		Unimin										
Sample ID:		Granite		Granite		Granite		Granite		RUN		Mineral Filler				Combined Gradation						
Rap?, Asphalt%:																Total Bin						
Individual Bin (%):		30.0	Percent	25.0	Percent	20.0	Percent	5.0	Percent	12.0	Percent	8.0	Percent		Percent	100.0%	Lower & Upper SMA_D Specification Limits			Restricted Zone SMA_D		
Sieve Size:	Sieve Size: (mm)	Cum.% Passing	Wt/ Cum. %	Cum.% Passing	Wt/ Cum. %	Cum.% Passing	Wt/ Cum. %	Cum.% Passing	Wt/ Cum. %	Cum.% Passing	Wt/ Cum. %	Cum.% Passing	Wt/ Cum. %	Cum.% Passing	Wt/ Cum. %	Cum. % Passing			Within Specs			Within Specs
3/4"	19.000	100.0	30.0	100.0	25.0	100.0	20.0	100.0	5.0	100.0	12.0	100.0	8.0			100.0	100.0	100.0	Yes			
1/2"	12.500	62.2	18.7	88.1	22.0	100.0	20.0	100.0	5.0	94.5	11.3	100.0	8.0			85.0	85.0	99.0	Yes			
3/8"	9.500	18.0	5.4	49.0	12.3	78.8	15.8	100.0	5.0	88.6	10.6	100.0	8.0			57.0	50.0	75.0	Yes			
No.4"	4.750	3.0	0.9	4.5	1.1	3.4	0.7	98.3	4.9	71.5	8.6	100.0	8.0			24.2	20.0	32.0	Yes			
No.8	2.360	2.1	0.6	1.3	0.3	1.4	0.3	83.5	4.2	54.0	6.5	100.0	8.0			19.9	16.0	28.0	Yes			
No. 16	1.180	2.0	0.6	1.2	0.3	0.9	0.2	58.0	2.9	39.0	4.7	100.0	8.0			16.7	8.0	28.0	Yes			
No. 30	0.600	1.7	0.5	1.2	0.3	0.7	0.1	40.7	2.0	27.9	3.3	100.0	8.0			14.3	8.0	28.0	Yes			
No. 50	0.300	1.2	0.4	1.1	0.3	0.4	0.1	27.0	1.4	19.9	2.4	99.9	8.0			12.4	8.0	28.0	Yes			
No. 100	0.150	1.1	0.3	1.1	0.3	0.3	0.1	17.9	0.9	14.4	1.7	94.9	7.6			10.9						
No. 200	0.075	1.1	0.3	1.0	0.3	0.2	0.0	12.5	0.6	10.6	1.3	68.7	5.5			8.0	8.0	12.0	Yes			

Table A10. Gradation of SMA D, US96, Beaumont.

		BIN FRACTIONS: SMA-D US96, Beaumont																				
		Bin No.1		Bin No.2		Bin No.3		Bin No.4		Bin No.5		Bin No.6		Bin No.7								
Aggregate Source:																						
Aggregate Number:																						
Sample ID:		Limestone		Granite		Limestone		domarill								Combined Gradation						
Rap?, Asphalt%:																Total Bin						
Individual Bin (%):		15.0	Percent	64.0	Percent	10.0	Percent	11.0	Percent	0.0	Percent		Percent		Percent	100.0%	Lower & Upper SMA_D Specification Limit			Restricted Zone SMA_D		
Sieve Size:	Sieve Size: (mm)	Cum.% Passing	Wt/ Cum. %	Cum.% Passing	Wt/ Cum. %	Cum.% Passing	Wt/ Cum. %	Cum.% Passing	Wt/ Cum. %	Cum.% Passing	Wt/ Cum. %	Cum.% Passing	Wt/ Cum. %	Cum.% Passing	Wt/ Cum. %	Cum. % Passing			Within Specs			Within Specs
3/4"	19.000	100.0	15.0	100.0	64.0	100.0	10.0	100.0	11.0	100.0	0.0	100.0	0.0			100.0	100.0	100.0	Yes			
1/2"	12.500	50.0	7.5	98.0	62.7	100.0	10.0	100.0	11.0	94.5	0.0	100.0	0.0			91.2	85.0	99.0	Yes			
3/8"	9.500	30.0	4.5	57.0	36.5	100.0	10.0	100.0	11.0	88.6	0.0	100.0	0.0			62.0	50.0	75.0	Yes			
No.4"	4.750	3.0	0.5	8.0	5.1	94.0	9.4	100.0	11.0	71.5	0.0	100.0	0.0			26.0	20.0	32.0	Yes			
No.8	2.360	3.0	0.5	5.0	3.2	69.0	6.9	100.0	11.0	54.0	0.0	100.0	0.0			21.6	16.0	28.0	Yes			
No. 16	1.180	1.0	0.2	3.0	1.9	48.0	4.8	100.0	11.0	39.0	0.0	100.0	0.0			17.9	8.0	28.0	Yes			
No.30	0.600	0.8	0.1	1.0	0.6	34.0	3.4	100.0	11.0	27.9	0.0	100.0	0.0			15.2	8.0	28.0	Yes			
No.50	0.300	0.7	0.1	1.0	0.6	25.0	2.5	98.0	10.8	19.9	0.0	99.9	0.0			14.0	8.0	28.0	Yes			
No. 100	0.150	0.6	0.1	1.0	0.6	20.0	2.0	78.0	8.6	14.4	0.0	94.9	0.0			11.3						
No. 200	0.075	0.5	0.1	1.0	0.6	17.0	1.7	58.0	6.4	10.6	0.0	68.7	0.0			8.8	8.0	12.0	Yes			

Table A11. Gradation of Limestone Fort Worth (FW).

BIN FRACTIONS: FortWorth																						
Aggregate:		Bin No.1		Bin No.2		Bin No.3		Bin No.4		Bin No.5		Bin No.6		Bin No.7								
		KellyPit D-Rock		TXI D-Rock		TXI Field Sand		KellyPit Man Sand		Lime												
Individual Bin (		31.0	Percent	26.0	Percen	9.0	Percen	33.0	Percen	1.0	Percent		Percent		Percent							
Sieve Size:	Sieve Size: (mm)	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum. % Passing	Lower & Upper SP_C Specification Limits		Within Spec's	Restricted Zone SP_C		Within Spec's
1/2"	12.500	100.0	31.0	100.0	26.0	100.0	9.0	100.0	33.0	100.0	1.0	0.0	0.0	0.0	0.0	100.0	98.0	100.0	Yes			
3/8"	9.500	98.7	30.6	97.3	25.3	100.0	9.0	100.0	33.0	100.0	1.0	0.0	0.0	0.0	0.0	98.9	85.0	100.0	Yes			
No. 4	4.750	35.0	10.9	40.2	10.5	99.8	9.0	99.4	32.8	100.0	1.0	0.0	0.0	0.0	0.0	64.1	50.0	70.0	Yes			
No. 8	2.360	0.9	0.3	3.2	0.8	99.8	9.0	82.9	27.4	100.0	1.0	0.0	0.0	0.0	0.0	38.5	35.0	46.0	Yes			
No. 16	1.180	0.3	0.1	0.7	0.2	99.7	9.0	53.5	17.7	100.0	1.0	0.0	0.0	0.0	0.0	27.9						
No. 30	0.600	0.2	0.1	0.6	0.2	99.1	8.9	31.9	10.5	100.0	1.0	0.0	0.0	0.0	0.0	20.7	15.0	29.0	Yes			
No. 50	0.300	0.2	0.1	0.6	0.2	84.0	7.6	17.1	5.6	100.0	1.0	0.0	0.0	0.0	0.0	14.4	7.0	20.0	Yes			
No. 100	0.150	0.2	0.1	0.6	0.2	26.4	2.4	8.6	2.8	100.0	1.0	0.0	0.0	0.0	0.0	6.4						
No. 200	0.075	0.1	0.0	0.5	0.1	7.0	0.6	6.0	2.0	100.0	1.0	0.0	0.0	0.0	0.0	3.8	2.0	7.0	Yes			

Table A12. Gradation of Superpave C Gravel-2.

BIN FRACTIONS: SPC_Gravel2																						
Aggregate:		Bin No.1		Bin No.2		Bin No.3		Bin No.4		Bin No.5		Bin No.6		Bin No.7								
		Fordyce C-Rock		Fordyce D-Rock		Fordyce Man Sand		lime stone Screenings														
Individual Bin (		26.0	Percent	42.0	Percent	13.0	Percen	19.0	Percent		Percent		Percent		Percent							
Sieve Size:	Sieve Size: (mm)	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum. % Passing	Lower & Upper SP_C Specification Limits		Within Spec's	Restricted Zone SP_C		Within Spec's
3/4"	19.000	100	26.0	100	42.0	100	13.0	100.0	19.0	0.0	0.0	0.0	0.0	0.0	0.0	100.0	98.0	100.0	Yes			
1/2"	12.500	70.8	18.4	100	42.0	100	13.0	100.0	19.0	0.0	0.0	0.0	0.0	0.0	0.0	92.4	90.0	100.0	Yes			
3/8"	9.500	19.4	5.0	96.6	40.6	100	13.0	99.2	18.8	0.0	0.0	0.0	0.0	0.0	0.0	77.5	28.0	90.0	Yes			
No.4"	4.750	0.5	0.1	46.6	19.6	98.8	12.8	97.4	18.5	0.0	0.0	0.0	0.0	0.0	0.0	51.1						
No. 8	2.360	0.1	0.0	10.9	4.6	87.5	11.4	92.3	17.5	0.0	0.0	0.0	0.0	0.0	0.0	33.5	28.0	58.0	Yes	###	39.1	Yes
No. 16	1.180	0.1	0.0	1.2	0.5	60.1	7.8	71.1	13.5	0.0	0.0	0.0	0.0	0.0	0.0	21.9	2.0	58.0	Yes	###	31.6	Yes
No. 30	0.600	0.1	0.0	0.2	0.1	34.2	4.4	53.5	10.2	0.0	0.0	0.0	0.0	0.0	0.0	14.7	2.0	58.0	Yes	###	23.1	Yes
No. 50	0.300	0.1	0.0	0.2	0.1	16.0	2.1	41.5	7.9	0.0	0.0	0.0	0.0	0.0	0.0	10.1	2.0	58.0	Yes	###	15.5	Yes
No.100	0.150	0.1	0.0	0.2	0.1	4.4	0.6	32.7	6.2	0.0	0.0	0.0	0.0	0.0	0.0	6.9						
No. 200	0.075	0.1	0.0	0.2	0.1	4.2	0.5	27.9	5.3	0.0	0.0	0.0	0.0	0.0	0.0	6.0	2.0	10.0	Yes			

Table A13. Gradation of Superpave C Granite.

BIN FRACTIONS: Granite																						
		Bin No.1		Bin No.2		Bin No.3		Bin No.4		Bin No.5		Bin No.6		Bin No.7								
Aggregate:		Granite C-Rock		Granite 5/8" Chip		Granite Grade4		Granite Screenings				DonnaFill										
Individual Bin (		0.0	Percent	20.0	Percent	46.0	Percen	26.0	Percent	0.0	Percent	8.0	Percent		Percent							
Sieve Size:	Sieve Size: (mm)	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum.% Passing	Wtd Cum. %	Cum. % Passing	Lower & Upper SP_C Specification Limits		Within Spec's	Restricted Zone SP_C		Within Spec's
3/4"	19.000	100	0.0	100	20.0	100	46.0	100.0	26.0	100.0	0.0	100.0	8.0	0.0	0.0	100.0	98.0	100.0	Yes			
1/2"	12.500	62.2	0.0	89.6	17.9	99.8	45.9	100.0	26.0	94.5	0.0	100.0	8.0	0.0	0.0	97.8	90.0	100.0	Yes			
3/8"	9.500	18	0.0	67	13.4	73.3	33.7	100.0	26.0	88.6	0.0	100.0	8.0	0.0	0.0	81.1	28.0	90.0	Yes			
No.4"	4.750	3.0	0.0	29	5.8	14	6.4	91.6	23.8	71.5	0.0	100.0	8.0	0.0	0.0	44.1						
No. 8	2.360	2.1	0.0	5.6	1.1	5.4	2.5	71.6	18.6	54.0	0.0	99.9	8.0	0.0	0.0	30.2	28.0	58.0	Yes	###	39.1	Yes
No. 16	1.180	2	0.0	3.6	0.7	3.3	1.5	51.8	13.5	39.0	0.0	99.8	8.0	0.0	0.0	23.7	2.0	58.0	Yes	###	31.6	Yes
No. 30	0.600	1.7	0.0	2.8	0.6	2.5	1.2	35.6	9.3	27.9	0.0	97.6	7.8	0.0	0.0	18.8	2.0	58.0	Yes	###	23.1	Yes
No. 50	0.300	1.2	0.0	2.3	0.5	1.9	0.9	23.5	6.1	19.9	0.0	66.6	5.3	0.0	0.0	12.8	2.0	58.0	Yes	###	15.5	Yes
No.100	0.150	1.1	0.0	1.9	0.4	1	0.5	14.8	3.8	14.4	0.0	42.9	3.4	0.0	0.0	8.1						
No. 200	0.075	1.1	0.0	1.6	0.3	0.7	0.3	10.4	2.7	10.6	0.0	25.5	2.0			5.4	2.0	10.0	Yes			

APPENDIX B:
HMA MIXTURES VOLUMETRIC DESIGN SUMMARY SHEET

Volumetric Design Summary Sheet: Dense-Graded Type D-TXI+PG64-22

TEXAS DEPARTMENT OF TRANSPORTATION

HMACP MIXTURE DESIGN : SUMMARY SHEET

File Version: 01/28/04 14:02:18

SAMPLE ID:	6422-Txi	SAMPLE DATE:	
LOT NUMBER:		LETTING DATE:	
STATUS:		CONTROLLING CSJ:	
COUNTY:		SPEC YEAR:	
SAMPLED BY:		SPEC ITEM:	
SAMPLE LOCATION:		SPECIAL PROVISION:	
MATERIAL:		MIX TYPE:	Type_D
PRODUCER:	US82-WITCHITA FALLS		
AREA ENGINEER:		PROJECT MANAGER:	

COURSE\LIFT:		STATION:		DIST. FROM CL:	

Target Density:	**96**	Percent
Number of Gyrations:		

Asphalt Content (%)	Specific Gravity Of Specimen (Ga)	Maximum Specific Gravity (Gr)	Effective Gravity (Ge)	Theo. Max. Specific Gravity (Gt)	Density from Gt (Percent)	VMA (Percent)
4.00	2.411	2.580	2.751	2.575	93.6	15.7
4.50	2.442	2.557	2.747	2.556	95.5	15.1
5.00	2.446	2.534	2.744	2.536	96.4	15.4
5.50	2.465	2.513	2.741	2.517	97.9	15.2

Effective Specific Gravity:	2.746

Optimum Asphalt Content:	**4.8**
VMA @ Optimum AC:	**15.2**

Interpolated Values	
Specific Gravity (Ga):	**2.444**
Max. Specific Gravity (Gr):	**2.545**
Theo. Max. Gravity (Gt):	**2.546**

Remarks:

Volumetric Design Summary Sheet: Dense-Graded Type D-TXI+PG76-22

TEXAS DEPARTMENT OF TRANSPORTATION

HMACP MIXTURE DESIGN : SUMMARY SHEET

File Version: 01/28/04 14:02:18

SAMPLE ID:	7622-Txi	SAMPLE DATE:	
LOT NUMBER:		LETTING DATE:	
STATUS:		CONTROLLING CSJ:	
COUNTY:		SPEC YEAR:	
SAMPLED BY:		SPEC ITEM:	
SAMPLE LOCATION:		SPECIAL PROVISION:	
MATERIAL:		MIX TYPE:	Type_D
PRODUCER:	US82-WICHITA FALLS		
AREA ENGINEER:		PROJECT MANAGER:	

COURSE\LIFT:		STATION:		DIST. FROM CL:	

Target Density:	**96**	Percent
Number of Gyrations:		

Asphalt Content (%)	Specific Gravity Of Specimen (Ga)	Maximum Specific Gravity (Gr)	Effective Gravity (Ge)	Theo. Max. Specific Gravity (Gt)	Density from Gt (Percent)	VMA (Percent)
4.00	2.416	2.593	2.767	2.589	93.3	16.0
4.50	2.454	2.574	2.768	2.569	95.5	15.1
5.00	2.461	2.554	2.768	2.549	96.6	15.3
5.50	2.471	2.527	2.758	2.530	97.7	15.5
6.00	2.482	2.501	2.750	2.511	98.9	15.5

Effective Specific Gravity:	2.762

Optimum Asphalt Content:	**4.7**
VMA @ Optimum AC:	**15.2**

Interpolated Values	
Specific Gravity (Ga):	**2.458**
Max. Specific Gravity (Gr):	**2.565**
Theo. Max. Gravity (Gt):	**2.560**

Remarks:

Volumetric Design Summary Sheet: Dense-Graded Type D-TCS+PG64-22

TEXAS DEPARTMENT OF TRANSPORTATION

HMACP MIXTURE DESIGN : SUMMARY SHEET

File Version: 01/28/04 14:02:18

SAMPLE ID:	6422-TCS	SAMPLE DATE:	
LOT NUMBER:		LETTING DATE:	
STATUS:		CONTROLLING CSJ:	
COUNTY:		SPEC YEAR:	
SAMPLED BY:		SPEC ITEM:	
SAMPLE LOCATION:		SPECIAL PROVISION:	
MATERIAL:		MIX TYPE:	Type_D
PRODUCER:	TCS		
AREA ENGINEER:		PROJECT MANAGER:	

COURSE\LIFT:		STATION:		DIST. FROM CL:	

Target Density:	**96**	Percent
Number of Gyrations:		

Asphalt Content (%)	Specific Gravity Of Specimen (Ga)	Maximum Specific Gravity (Gr)	Effective Gravity (Ge)	Theo. Max. Specific Gravity (Gt)	Density from Gt (Percent)	VMA (Percent)
4.00	2.259	2.478	2.631	2.468	91.5	17.2
4.50	2.289	2.446	2.614	2.450	93.4	16.5
5.00	2.308	2.432	2.619	2.432	94.9	16.3
5.50	2.318	2.409	2.611	2.415	96.0	16.3

Effective Specific Gravity:	2.619

Optimum Asphalt Content:	**5.5**
VMA @ Optimum AC:	**16.3**

Interpolated Values	
Specific Gravity (Ga):	**2.318**
Max. Specific Gravity (Gr):	**2.408**
Theo.Max.Gravity (Gt):	**2.415**

Remarks:

Volumetric Design Summary Sheet: Superpave C-Sandstone+PG64-22

TEXAS DEPARTMENT OF TRANSPORTATION

HMACP MIXTURE DESIGN : SUMMARY SHEET

File Version: 01/28/04 14:02:18

SAMPLE ID:	6422-Sandstone_L	SAMPLE DATE:	
LOT NUMBER:		LETTING DATE:	
STATUS:		CONTROLLING CSJ:	
COUNTY:		SPEC YEAR:	
SAMPLED BY:		SPEC ITEM:	
SAMPLE LOCATION:		SPECIAL PROVISION:	
MATERIAL:		MIX TYPE:	SP_C
PRODUCER:	IH20-Atlanta		
AREA ENGINEER:		PROJECT MANAGER:	

COURSE\LIFT:		STATION:		DIST. FROM CL:	

Target Density:	**96**	Percent
Number of Gyrations:	100	

Asphalt Content (%)	Specific Gravity Of Specimen (Ga)	Maximum Specific Gravity (Gr)	Effective Gravity (Ge)	Theo. Max. Specific Gravity (Gt)	Density from Gt (Percent)	VMA (Percent)
4.50	2.270	2.396	2.554	2.400	94.6	15.3
5.00	2.286	2.383	2.558	2.383	95.9	15.1
5.50	2.299	2.371	2.564	2.367	97.1	15.1

Effective Specific Gravity:	2.559

Optimum Asphalt Content:	**5.0**
VMA @ Optimum AC:	**15.1**

Interpolated Values	
Specific Gravity (Ga):	**2.287**
Max. Specific Gravity (Gr):	**2.382**
Theo. Max.Gravity (Gt):	**2.382**

Remarks:

Volumetric Design Summary Sheet: Superpave C-Sandstone+PG70-22

TEXAS DEPARTMENT OF TRANSPORTATION

HMACP MIXTURE DESIGN : SUMMARY SHEET

File Version: 01/28/04 14:02:18

SAMPLE ID:	70-22-Sandstone_NL	SAMPLE DATE:	
LOT NUMBER:		LETTING DATE:	
STATUS:		CONTROLLING CSJ:	
COUNTY:		SPEC YEAR:	
SAMPLED BY:		SPEC ITEM:	
SAMPLE LOCATION:		SPECIAL PROVISION:	
MATERIAL:		MIX TYPE:	SP_C
PRODUCER:	IH20-Atlanta		
AREA ENGINEER:		PROJECT MANAGER:	

COURSE\LIFT:		STATION:		DIST. FROM CL:	

Target Density:	**96**	Percent
Number of Gyrations:	100	

Asphalt Content (%)	Specific Gravity Of Specimen (Ga)	Maximum Specific Gravity (Gr)	Effective Gravity (Ge)	Theo. Max. Specific Gravity (Gt)	Density from Gt (Percent)	VMA (Percent)
4.50	2.262	2.407	2.567	2.405	94.0	15.8
5.00	2.284	2.391	2.568	2.389	95.6	15.4
5.50	2.315	2.370	2.562	2.373	97.6	14.7
6.00	2.319	2.354	2.562	2.356	98.4	15.0

Effective Specific Gravity:	2.565

Optimum Asphalt Content:	**5.1**
VMA @ Optimum AC:	**15.3**

Interpolated Values	
Specific Gravity (Ga):	**2.290**
Max. Specific Gravity (Gr):	**2.387**
Theo. Max. Gravity (Gt):	**2.386**

Remarks:

Volumetric Design Summary Sheet: Superpave C-Gravel-1+PG76-22

TEXAS DEPARTMENT OF TRANSPORTATION

HMACP MIXTURE DESIGN : SUMMARY SHEET

File Version: 01/28/04 14:02:18

SAMPLE ID:	76-22-Gravel-1	SAMPLE DATE:	
LOT NUMBER:		LETTING DATE:	
STATUS:		CONTROLLING CSJ:	
COUNTY:		SPEC YEAR:	
SAMPLED BY:		SPEC ITEM:	
SAMPLE LOCATION:		SPECIAL PROVISION:	
MATERIAL:		MIX TYPE:	SP_C
PRODUCER:	IH20-Atlanta		
AREA ENGINEER:		PROJECT MANAGER:	

COURSE\LIFT:		STATION:		DIST. FROM CL:	

Target Density:	**96**	Percent
Number of Gyrations:	100	

Asphalt Content (%)	Specific Gravity Of Specimen (Ga)	Maximum Specific Gravity (Gr)	Effective Gravity (Ge)	Theo. Max. Specific Gravity (Gt)	Density from Gt (Percent)	VMA (Percent)
4.50	2.279	2.443	2.610	2.437	93.5	16.4
5.00	2.296	2.428	2.613	2.420	94.9	16.2
5.50	2.303	2.395	2.593	2.403	95.8	16.4
6.00	2.336	2.381	2.597	2.386	97.9	15.7

Effective Specific Gravity:	2.603

Optimum Asphalt Content:	**5.5**
VMA @ Optimum AC:	**16.3**

Interpolated Values	
Specific Gravity (Ga):	**2.306**
Max. Specific Gravity (Gr):	**2.394**
Theo. Max. Gravity (Gt):	**2.402**

Remarks:

Volumetric Design Summary Sheet: Superpave C-Quartzite+PG76-22

TEXAS DEPARTMENT OF TRANSPORTATION

HMACP MIXTURE DESIGN : SUMMARY SHEET

File Version: 01/28/04 14:02:18

SAMPLE ID:	76-22-Quartzite_MD_L	SAMPLE DATE:	
LOT NUMBER:		LETTING DATE:	
STATUS:		ONTROLLING CSJ:	
COUNTY:		SPEC YEAR:	
SAMPLED BY:		SPEC ITEM:	
SAMPLE LOCATION:		ECIAL PROVISION:	
MATERIAL:		MIX TYPE:	SP_C
PRODUCER:	IH20-Atlanta		
AREA ENGINEER:		OJECT MANAGER:	

COURSE\LIFT:		STATION:		DIST. FROM CL:	

Target Density:	**96**	Percent
Number of Gyrations:	100	

Asphalt Content (%)	Specific Gravity Of Specimen (Ga)	Maximum Specific Gravity (Gr)	Effective Gravity (Ge)	Theo. Max. Specific Gravity (Gt)	Density from Gt (Percent)	VMA (Percent)
4.50	2.338	2.496	2.674	2.494	93.7	16.4
5.00	2.343	2.476	2.672	2.476	94.6	16.7
5.50	2.368	2.455	2.668	2.458	96.3	16.2
6.00	2.374	2.441	2.673	2.440	97.3	16.5

Effective Specific Gravity:	2.672

Optimum Asphalt Content:	**5.4**
VMA @ Optimum AC:	**16.3**

Interpolated Values	
Specific Gravity (Ga):	**2.363**
Max. Specific Gravity (Gr):	**2.459**
Theo. Max. Gravity (Gt):	**2.461**

Remarks:

Volumetric Design Summary Sheet: SMA C-Granite+PG76-22

TEXAS DEPARTMENT OF TRANSPORTATION

HMACP MIXTURE DESIGN : SUMMARY SHEET

File Version: 01/28/04 14:02:18

SAMPLE ID:	7622TR-Granite	SAMPLE DATE:	
LOT NUMBER:		LETTING DATE:	
STATUS:		ONTROLLING CSJ:	
COUNTY:		SPEC YEAR:	
SAMPLED BY:		SPEC ITEM:	
SAMPLE LOCATION:		ECIAL PROVISION:	
MATERIAL:		MIX TYPE:	SMA_C
PRODUCER:	HOUSTON		
AREA ENGINEER:		OJECT MANAGER:	

COURSE\LIFT:		STATION:		DIST. FROM CL:	

Target Density:	**96**	Percent
Number of Gyrations:	75	

Asphalt Content (%)	Specific Gravity Of Specimen (Ga)	Maximum Specific Gravity (Gr)	Effective Gravity (Ge)	Theo. Max. Specific Gravity (Gt)	Density from Gt (Percent)	VMA (Percent)
5.50	2.39	2.52	2.751	2.520	94.9	17.9
6.00	2.40	2.50	2.752	2.501	96.1	18.0
6.50	2.42	2.49	2.758	2.482	97.7	17.7

Effective Specific Gravity:	2.754

Optimum Asphalt Content:	**6.0**
VMA @ Optimum AC:	**18.0**

Interpolated Values	
Specific Gravity (Ga):	**2.402**
Max. Specific Gravity (Gr):	**2.501**
Theo. Max. Gravity (Gt):	**2.502**

Remarks:

Volumetric Design Summary Sheet: SMA D (IH635)-Granite+PG76-22

TEXAS DEPARTMENT OF TRANSPORTATION

HMACP MIXTURE DESIGN : SUMMARY SHEET

File Version: 01/28/04 14:02:18

SAMPLE ID:	7622-Granite	SAMPLE DATE:	
LOT NUMBER:		LETTING DATE:	
STATUS:		CONTROLLING CSJ:	
COUNTY:		SPEC YEAR:	
SAMPLED BY:		SPEC ITEM:	
SAMPLE LOCATION:		SPECIAL PROVISION:	
MATERIAL:		MIX TYPE:	SMA_D
PRODUCER:	IH635-DALLAS		
AREA ENGINEER:		PROJECT MANAGER:	

COURSE\LIFT:		STATION:		DIST. FROM CL:	

Target Density:	**96**	Percent
lumber of Gyrations:	75	

Asphalt Content (%)	Specific Gravity Of Specimen (Ga)	Maximum Specific Gravity (Gr)	Effective Gravity (Ge)	Theo. Max. Specific Gravity (Gt)	Density from Gt (Percent)	VMA (Percent)
5.50	2.391	2.518	2.751	2.520	94.9	17.9
6.00	2.403	2.499	2.752	2.501	96.1	18.0
6.50	2.424	2.485	2.758	2.482	97.7	17.7

Effective Specific Gravity:	2.754

Optimum Asphalt Content:	**6.0**
VMA @ Optimum AC:	**18.0**

Interpolated Values	
Specific Gravity (Ga):	**2.402**
Max. Specific Gravity (Gr):	**2.501**
Theo. Max. Gravity (Gt):	**2.502**

Remarks:

Volumetric Design Summary Sheet: SMA D (US96)-Granite+PG76-22

TEXAS DEPARTMENT OF TRANSPORTATION

HMACP MIXTURE DESIGN : SUMMARY SHEET

File Version: 01/28/04 14:02:18

SAMPLE ID:	7622-Granite	SAMPLE DATE:	
LOT NUMBER:		LETTING DATE:	
STATUS:		CONTROLLING CSJ:	
COUNTY:		SPEC YEAR:	
SAMPLED BY:		SPEC ITEM:	
SAMPLE LOCATION:		SPECIAL PROVISION:	
MATERIAL:		MIX TYPE:	SMA_D
PRODUCER:	US96-BEAUMONT		
AREA ENGINEER:		PROJECT MANAGER:	

COURSE\LIFT:		STATION:		DIST. FROM CL:	

Target Density:	**96**	Percent
Number of Gyrations:	75	

Asphalt Content (%)	Specific Gravity Of Specimen (Ga)	Maximum Specific Gravity (Gr)	Effective Gravity (Ge)	Theo. Max. Specific Gravity (Gt)	Density from Gt (Percent)	VMA (Percent)
5.50	2.285	2.454	2.669	2.454	93.1	19.1
6.00	2.303	2.436	2.669	2.436	94.5	18.9
6.50	2.338	2.419	2.669	2.419	96.7	18.1
7.00	2.352	2.402	2.670	2.402	97.9	18.0

Effective Specific Gravity:	2.669

Optimum Asphalt Content:	**6.3**
VMA @ Optimum AC:	**18.3**

Interpolated Values	
Specific Gravity (Ga):	**2.327**
Max. Specific Gravity (Gr):	**2.424**
Theo. Max. Gravity (Gt):	**2.424**

Remarks:

APPENDIX C:
OVERLAY TESTER RESULTS

Table C1. Overlay Tester Results − All Dense-Graded Type D Mixtures.

Asphalt binder PG	Aggregate type	Sample no.	AC (%)	Air void (%)	VMA(%)		VFA(%)		VBE(%)	Asphalt absorption (%)	SA (m2/kg)	FT (microns)	OT (no. of cycles)
					TxDOT	SuperPave	TxDOT	SuperPave	SuperPave				
64-22	Limestone-TCS	1	4.0	6.4	15.3	13.3	58.3	52.1	6.9	0.93	4.873	6.42	7
		2	4.0	6.5	15.5	13.5	57.7	51.4	6.9	0.93	4.873	6.42	6
		1	4.5	6.3	16.3	14.3	61.4	56.1	8.0	0.93	4.873	7.51	6
		2	4.5	5.9	15.9	13.9	63.0	57.8	8.1	0.93	4.873	7.51	10
		1	5.0	5.7	16.8	14.9	66.1	61.7	9.2	0.93	4.873	8.60	32
		2	5.0	6.4	17.4	15.5	63.3	58.8	9.1	0.93	4.873	8.60	15
		1	5.5	6.6	18.6	16.7	64.5	60.5	10.1	0.93	4.873	9.70	42
		2	5.5	7.4	19.3	17.4	61.5	57.4	10.0	0.93	4.873	9.70	52
64-22	Limestone-FW	1	4.5	5.5	15.8	15.4	65.1	64.1	9.9	0.22	4.451	9.76	14
		2	4.5	5.8	16.1	15.6	64.1	63.0	9.8	0.22	4.451	9.76	10
		1	5.0	5.4	16.8	16.3	67.9	67.0	11.00	0.22	4.451	10.96	82
		2	5.0	5.3	16.7	16.3	68.1	67.2	11.00	0.22	4.451	10.96	284
		1	5.5	6.6	18.9	18.4	65.1	64.2	11.8	0.22	4.451	12.17	569
		2	6.0	6.4	19.7	19.2	67.7	67.0	12.9	0.22	4.451	13.39	1423
64-22	Limestone-TXI	1	4.5	5.3	15.8	15.7	66.8	66.5	10.4	0.15	4.433	10.00	20
		2	4.5	5.9	16.4	16.3	63.8	63.5	10.3	0.15	4.433	10.00	38
		3	4.5	6.6	17.0	16.6	61.5	60.5	10.0	0.2	4.433	9.83	29
		4	4.5	6.2	16.7	16.3	62.8	61.8	10.1	0.2	4.433	9.83	91
		1	5.0	6.6	18.1	18.0	63.4	63.1	11.3	0.15	4.433	11.23	231
		2	5.0	6.5	18.0	17.8	64.0	63.7	11.4	0.15	4.433	11.03	88
		3	5.0	6.1	17.6	17.2	65.7	64.8	11.2	0.2	4.433	11.03	104
		4	5.0	6.2	17.7	17.3	65.2	64.4	11.1	0.2	4.433	11.03	176
		1	5.5	8.8	21.0	20.9	58.3	58.1	12.1	0.1	4.433	12.54	846
		2	5.5	6.0	18.6	18.2	68.0	67.2	12.2	0.2	4.433	12.24	780

Table C1. Overlay Tester Results − All Dense-Graded Type D Mixtures (Continued).

Asphalt binder PG	Aggregate type	Sample no.	AC (%)	Air void (%)	VMA(%)		VFA(%)		VBE(%)	Asphalt absorption (%)	SA (m2/kg)	FT (microns)	OT (no. of cycles)
					TxDOT	SuperPave	TxDOT	SuperPave	SuperPave				
76-22	Limestone-TXI	1	4.5	7.2	17.6	17.3	59.0	58.3	10.1	0.14	4.433	9.97	126
		2	4.5	6.4	16.9	16.6	61.9	61.3	10.2	0.14	4.433	9.97	132
		1	5.0	5.0	16.7	16.4	70.0	69.4	11.4	0.14	4.433	11.17	306
		2	5.0	6.5	18.0	17.7	64.0	63.4	11.2	0.14	4.433	11.17	286
		1	5.5	6.9	19.4	19.1	64.5	64.0	12.2	0.14	4.433	12.39	620
		2	5.5	7.3	19.7	19.4	63.2	62.6	12.2	0.14	4.433	12.39	592
		1	6.0	5.8	19.5	19.2	70.2	69.7	13.4	0.14	4.433	13.62	1320
		2	6.0	8.0	23.1	22.8	56.8	56.3	12.8	0.14	4.433	13.62	1410

Table C2. Overlay Tester Results − All Superpave C Mixtures.

Asphalt binder PG	Aggregate type	Sample No.	AC (%)	Air void (%)	VMA(%)		VFA(%)		VBE (%) SuperPave	Asphalt absorption (%)	SA (m2/kg)	FT (microns)	OT (no. of cycles)
					TxDOT	SuperPave	TxDOT	SuperPave					
64-22	Sandstone-L	1	4.5	6.7	16.4	14.2	59.0	52.7	7.5	1.07	6.037	5.84	20
		2	4.5	7.1	16.8	14.6	57.6	51.2	7.5	1.07	6.037	5.84	16
		1	5.0	7.1	17.7	15.6	60.1	54.5	8.5	1.07	6.037	6.72	76
		2	5.0	7.1	17.8	15.6	59.9	54.4	8.5	1.07	6.037	6.72	143
		3	5.0	2.8	14.0	11.1	80.0	74.8	8.3	1.37	6.037	6.24	84
		4	5.0	3.3	14.4	11.5	77.4	71.7	8.3	1.37	6.037	6.24	136
		1	6.0	0.8	14.3	11.5	94.5	93.1	10.7	1.37	6.037	8.03	322
		2	6.0	1.1	14.6	11.7	92.6	90.8	10.7	1.37	6.037	8.03	334
64-22	Quartzite_ MD_L	1	5.0	0.9	12.7	11.6	92.7	92.0	10.7	0.49	6.368	7.25	63
		2	5.0	1.9	13.6	12.5	86.1	84.9	10.6	0.49	6.368	7.25	92
		1	6.0	0.1	13.8	12.8	100.0	100.0	13.1	0.49	6.368	8.94	302
		2	6.0	0.1	14.0	12.9	100.0	100.0	13.1	0.49	6.368	8.94	256
76-22	Gravel-1	1	4.5	5.0	15.1	14.5	66.6	65.2	9.4	0.29	5.866	7.28	2
		2	4.5	5.0	15.1	14.4	66.9	65.4	9.4	0.29	5.866	7.28	4
		1	5.0	4.4	15.6	15.0	71.6	70.4	10.5	0.29	5.866	8.19	2
		2	5.0	4.6	15.7	15.1	70.8	69.6	10.5	0.29	5.866	8.19	8
		1	5.5	6.0	18.0	17.4	66.8	65.6	11.4	0.29	5.866	9.10	90
		2	5.5	5.7	17.7	17.1	67.9	66.7	11.4	0.29	5.866	9.10	120
		1	6.0	6.1	19.1	18.5	68.1	67.1	12.4	0.29	5.866	10.03	240
		2	6.0	4.7	17.9	17.3	73.5	72.6	12.6	0.29	5.866	10.03	260
76-22	Gravel-2	1	5.0	6.2	17.2	16.5	63.8	62.4	10.3	0.32	5.11	9.34	224
		2	5.0	6.4	17.3	16.7	63.0	61.5	10.3	0.32	5.11	9.34	194
		1	5.5	6.0	18.0	17.3	66.8	65.5	11.3	0.32	5.11	10.39	250
		2	5.5	6.0	18.0	17.3	66.5	65.2	11.3	0.32	5.11	10.39	619
		1	6.0	4.6	17.8	17.1	74.1	73.1	12.5	0.32	5.11	11.46	743
		2	6.0	5.0	18.1	17.4	72.5	71.5	12.5	0.32	5.11	11.46	585

Table C2. Overlay Tester Results − All Superpave C Mixtures (Continued).

Asphalt binder PG	Aggregate type	Sample no.	AC (%)	Air void (%)	VMA(%)		VFA(%)		VBE (%) SuperPave	Asphalt absorption (%)	SA (m2/kg)	FT (microns)	OT (no. of cycles)
					TxDOT	SuperPave	TxDOT	SuperPave					
76-22	Granite	1	4.5	4.4	14.9	14.3	70.3	69.1	9.9	0.30	5.313	8.02	195
		2	4.5	5.2	15.6	15.0	66.7	65.5	9.8	0.30	5.313	8.02	110
		1	5.0	4.4	16.0	15.0	72.5	70.8	10.6	0.42	5.313	8.72	100
		2	5.0	5.3	16.7	15.8	68.5	66.7	10.6	0.42	5.313	8.72	122
		1	5.5	3.6	16.3	15.8	78.0	77.2	12.2	0.30	5.313	10.04	340
		2	5.5	3.4	16.1	15.6	79.1	78.4	12.2	0.30	5.313	10.04	370
		3	5.5	3.8	16.5	15.6	77.2	75.8	11.8	0.42	5.313	9.73	331
		4	5.5	4.4	17.0	16.1	74.2	72.7	11.7	0.42	5.313	9.73	565
		1	6.0	4.1	17.8	16.9	77.1	75.9	12.9	0.42	5.313	10.75	496
		2	6.0	4.6	18.2	17.4	75.0	73.7	12.8	0.42	5.313	10.75	430
		1	6.5	4.5	19.2	18.7	76.3	75.7	14.2	0.30	5.313	12.10	1711
		2	6.5	4.9	19.5	19.0	74.9	74.3	14.1	0.30	5.313	12.10	1500
64-22	Granite	1	4.5	5.5	15.9	15.2	65.2	63.7	9.7	0.34	5.313	7.95	34
		2	4.5	7.0	17.2	16.5	59.4	57.8	9.6	0.34	5.313	7.95	91
		1	5.0	4.3	15.8	15.2	73.1	72.0	10.9	0.34	5.313	8.95	104
		2	5.0	4.4	15.9	15.3	72.6	71.5	10.9	0.34	5.313	8.95	176
		1	5.5	3.8	16.5	15.8	77.1	76.2	12.1	0.34	5.313	9.96	442
		2	5.5	3.7	16.6	16.0	77.1	76.2	12.3	0.34	5.313	10.12	520

APPENDIX D:

HAMBURG WHEEL TRACKING TEST RESULTS

Table D1. Hamburg Wheel Tracking Test Results − Dense-Graded Type D Mixtures.

Asphalt binder PG	Aggregate type	Sample no.	AC (%)	Air void (%)	VMA(%)		VFA(%)		VBE(%)	Asphalt absorption (%)	SA (m^2/kg)	FT (microns)	Gsb	G* (Pa)	Hamburg-RD (mm)
					TxDOT	SuperPave	TxDOT	SuperPave	SuperPave						
64-22	Limestone-TCS	1	4.5	7.7	17.5	15.6	56.2	50.8	7.9	0.93	4.873	7.51	2.559	16497	3.7
		2	4.5	7.3	17.1	15.2	57.6	52.3	7.9	0.93	4.873	7.51	2.559	16497	3.7
		3	4.5	8.0	17.8	15.9	55.0	49.6	7.9	0.93	4.873	7.51	2.559	16497	3.7
		4	4.5	7.9	17.7	15.7	55.5	50.1	7.9	0.93	4.873	7.51	2.559	16497	3.7
		1	5.0	7.6	18.4	16.6	58.9	54.2	9.0	0.93	4.873	8.60	2.559	16497	6.8
		2	5.0	7.5	18.4	16.5	59.2	54.6	9.0	0.93	4.873	8.60	2.559	16497	6.8
		3	5.0	7.8	18.7	16.8	58.0	53.3	9.0	0.93	4.873	8.60	2.559	16497	6.8
		4	5.0	8.4	19.1	17.3	56.3	51.6	8.9	0.93	4.873	8.60	2.559	16497	6.8
		1	5.5	8.4	20.2	18.3	58.3	54.1	9.9	0.93	4.873	9.70	2.559	16497	9.6
		2	5.5	7.5	19.4	17.5	61.3	57.2	10.0	0.93	4.873	9.70	2.559	16497	9.6
		3	5.5	8.2	20.0	18.1	59.1	54.9	9.9	0.93	4.873	9.70	2.559	16497	9.6
		4	5.5	8.7	20.4	18.6	57.4	53.2	9.9	0.93	4.873	9.70	2.559	16497	9.6
		1	6.0	7.5	20.4	18.5	63.1	59.4	11.0	0.93	4.873	10.82	2.559	16497	13.4
		2	6.0	8.5	21.2	19.4	59.9	56.1	10.9	0.93	4.873	10.82	2.559	16497	13.4
		3	6.0	8.4	21.1	19.3	60.3	56.6	10.9	0.93	4.873	10.82	2.559	16497	13.4
64-22	Limestone-TXI	1	4.5	7.5	17.8	17.6	58.0	57.6	10.2	0.07	4.433	10.12	2.752	16497	3.9
		2	4.5	7.8	18.1	17.9	56.9	56.5	10.1	0.07	4.433	10.12	2.752	16497	3.9
		3	4.5	6.8	17.2	16.8	60.6	59.5	10.0	0.2	4.433	9.83	2.752	16497	4.7
		4	4.5	6.8	17.2	16.8	60.4	59.4	10.0	0.2	4.433	9.83	2.752	16497	4.7
		1	5.0	9.2	20.3	20.2	55.0	54.7	11.0	0.07	4.433	11.32	2.752	16497	5.8
		2	5.0	9.3	20.4	20.3	54.6	54.3	11.0	0.07	4.433	11.32	2.752	16497	5.8
		3	5.0	6.6	18.1	17.7	63.6	62.7	11.1	0.2	4.433	11.03	2.752	16497	6.0
		4	5.0	6.9	18.4	17.9	62.6	61.7	11.1	0.2	4.433	11.03	2.752	16497	6.0
		1	5.5	9.2	21.3	21.2	57.1	56.9	12.1	0.07	4.433	12.54	2.752	16497	18.5
		2	5.5	8.3	20.6	20.5	59.7	59.5	12.2	0.07	4.433	12.54	2.752	16497	18.5
		3	5.5	7.0	19.5	19.1	64.1	63.3	12.1	0.2	4.433	12.24	2.752	16497	18.0
		4	5.5	6.8	19.4	18.9	64.8	63.9	12.1	0.2	4.433	12.24	2.752	16497	18.0

Table D1. Hamburg Wheel Tracking Test Results − Dense-Graded Type D Mixtures (Continued).

Asphalt binder PG	Aggregate type	Sample no.	AC (%)	Air void (%)	VMA(%)		VFA(%)		VBE(%)	Asphalt absorption (%)	SA (m^2/kg)	FT (microns)	Gsb	G* (Pa)	Hamburg-RD (mm)
					TxDOT	SuperPave	TxDOT	SuperPave	SuperPave						
64-22	Limestone-FW	1	4.5	6.5	16.7	16.2	61.3	60.2	9.8	0.22	4.451	9.76	2.676	16497	7.3
		2	4.5	6.6	16.8	16.3	60.6	59.5	9.7	0.22	4.451	9.76	2.676	16497	7.3
		1	5.0	6.7	18.0	17.5	62.5	61.5	10.8	0.22	4.451	10.96	2.676	16497	22.0
		2	5.0	6.9	18.1	17.7	61.7	60.8	10.8	0.22	4.451	10.96	2.676	16497	22.0
		1	5.5	7.9	20.0	19.6	60.4	59.5	11.7	0.22	4.451	12.17	2.676	16497	49.0
		2	5.5	8.2	20.3	19.9	59.4	58.5	11.6	0.22	4.451	12.17	2.676	16497	49.0
		1	6.0	8.7	21.7	21.3	59.8	59.0	12.6	0.22	4.451	13.39	2.676	16497	58.0
		2	6.0	8.1	21.2	20.8	61.6	60.8	12.6	0.22	4.451	13.39	2.676	16497	58.0
76-22	Limestone-TXI	1	4.5	7.9	18.2	17.9	56.4	55.7	10.0	0.14	4.433	9.97	2.752	25543	5.4
		2	4.5	7.9	18.2	17.9	56.7	56.0	10.0	0.14	4.433	9.97	2.752	25543	5.4
		3	4.5	7.9	18.2	17.9	56.4	55.7	10.0	0.14	4.433	9.97	2.752	25543	5.4
		4	4.5	7.9	18.2	17.9	56.7	56.0	10.0	0.14	4.433	9.97	2.752	25543	5.4
		1	5.0	7.1	18.6	18.3	61.6	61.1	11.2	0.14	4.433	11.17	2.752	25543	3.9
		2	5.0	7.2	18.6	18.4	61.3	60.7	11.1	0.14	4.433	11.17	2.752	25543	3.9
		3	5.0	6.5	18.0	17.7	63.8	63.3	11.2	0.14	4.433	11.17	2.752	25543	3.9
		4	5.0	6.5	18.0	17.7	64.0	63.4	11.2	0.14	4.433	11.17	2.752	25543	3.9
		1	5.5	6.9	19.5	19.2	64.3	63.7	12.2	0.14	4.433	12.39	2.752	25543	7.4
		2	5.5	6.7	19.3	19.0	65.0	64.5	12.2	0.14	4.433	12.39	2.752	25543	7.4
		3	5.5	7.7	20.1	19.8	61.6	61.1	12.1	0.14	4.433	12.39	2.752	25543	7.4
		4	5.5	7.4	19.8	19.5	62.9	62.3	12.2	0.14	4.433	12.39	2.752	25543	7.4
		1	6.0	7.4	20.9	20.6	64.6	64.1	13.2	0.14	4.433	13.62	2.752	25543	10.2
		2	6.0	6.8	20.4	20.1	66.5	65.9	13.3	0.14	4.433	13.62	2.752	25543	10.2
		3	6.0	7.0	20.6	20.3	65.8	65.3	13.2	0.14	4.433	13.62	2.752	25543	10.2
		4	6.0	7.3	20.8	20.5	64.9	64.4	13.2	0.14	4.433	13.62	2.752	25543	10.2

Table D2. Hamburg Wheel Tracking Test Results − Superpave C Mixtures.

Asphalt binder PG	Aggregate type	Sample no.	AC (%)	Air void (%)	VMA(%)		VFA(%)		VBE(%)	Asphalt absorption (%)	SA (m2/kg)	FT (microns)	Gsb	G* (Pa)	Hamburg-RD (mm)
					TxDOT	SuperPave	TxDOT	SuperPave	SuperPave						
64-22	Sandstone_ L	1	4.5	7.5	17.1	15.0	56.0	49.7	7.4	1.07	6.037	5.84	2.481	16497	4.0
		2	4.5	7.7	17.3	15.2	55.3	48.9	7.4	1.07	6.037	5.84	2.481	16497	4.0
		1	5.0	8.3	18.8	16.6	56.0	50.4	8.4	1.07	6.037	6.72	2.481	16497	5.9
		2	5.0	8.7	19.1	17.0	54.7	49.0	8.3	1.07	6.037	6.72	2.481	16497	5.9
76-22	Gravel-1	1	4.5	5.9	15.9	15.3	62.7	61.2	9.3	0.29	5.866	7.28	2.584	25543	1.0
		2	4.5	5.4	15.4	14.8	65.1	63.6	9.4	0.29	5.866	7.28	2.584	25543	1.0
		1	5.0	6.4	17.4	16.7	63.1	61.7	10.3	0.29	5.866	8.19	2.584	25543	2.2
		2	5.0	6.5	17.4	16.8	62.7	61.3	10.3	0.29	5.866	8.19	2.584	25543	2.2
		1	5.5	7.7	19.5	18.9	60.4	59.1	11.2	0.29	5.866	9.10	2.584	25543	3.0
		2	5.5	7.4	19.2	18.6	61.6	60.4	11.2	0.29	5.866	9.10	2.584	25543	3.0
		1	6.0	8.1	20.8	20.2	61.1	60.0	12.1	0.29	5.866	10.03	2.584	25543	2.9
		2	6.0	6.9	19.8	19.2	64.9	63.9	12.3	0.29	5.866	10.03	2.584	25543	2.9
76-22	Gravel-2	1	5.0	7.5	18.3	17.7	58.9	57.4	10.1	0.32	5.11	9.34	2.578	25543	3.5
		2	5.0	7.7	18.5	17.9	58.2	56.7	10.1	0.32	5.11	9.34	2.578	25543	3.5
		1	5.5	9.0	20.6	19.9	56.4	54.9	10.9	0.32	5.11	10.39	2.578	25543	3.4
		2	5.5	7.6	19.4	18.7	60.8	59.4	11.1	0.32	5.11	10.39	2.578	25543	3.4
		1	6.0	6.0	19.0	18.3	68.4	67.2	12.3	0.32	5.11	11.46	2.578	25543	2.3
		2	6.0	6.1	19.0	18.4	68.1	67.0	12.3	0.32	5.11	11.46	2.578	25543	2.3
64-22	Granite	1	4.5	7.5	17.6	17.0	57.5	56.0	9.5	0.34	5.313	7.95	2.680	16497	3.5
		2	4.5	6.9	17.1	16.5	59.5	58.0	9.6	0.34	5.313	7.95	2.680	16497	3.5
		1	5.0	5.0	16.5	15.9	69.5	68.3	10.8	0.34	5.313	8.95	2.680	16497	6.2
		2	5.0	5.4	16.8	16.2	67.9	66.7	10.8	0.34	5.313	8.95	2.680	16497	6.2
		1	5.5	5.7	18.1	17.5	68.7	67.6	11.8	0.34	5.313	9.96	2.680	16497	10.5
		2	5.5	5.3	17.8	17.2	70.3	69.2	11.9	0.34	5.313	9.96	2.680	16497	10.5

Table D2. Hamburg Wheel Tracking Test Results − Superpave C Mixtures (Continued).

Asphalt binder PG	Aggregate type	Sample no.	AC (%)	Air void (%)	VMA(%)		VFA(%)		VBE(%)	Asphalt absorption (%)	SA (m2/kg)	FT (microns)	Gsb	G* (Pa)	Hamburg-RD (mm)
					TxDOT	SuperPave	TxDOT	SuperPave	SuperPave						
76-22	Granite	1	4.5	5.7	16.0	15.5	64.3	63.0	9.8	0.30	5.313	8.02	2.680	25543	2.1
		2	4.5	5.6	15.9	15.4	64.9	63.6	9.8	0.30	5.313	8.02	2.680	25543	2.1
		1	5.0	5.9	17.3	16.4	65.8	63.9	10.5	0.42	5.313	8.72	2.680	25543	1.8
		2	5.0	5.3	16.8	15.8	68.4	66.6	10.5	0.42	5.313	8.72	2.680	25543	1.8
		1	5.5	5.3	17.8	16.9	70.3	68.7	11.6	0.42	5.313	9.73	2.680	25543	2.8
		2	5.5	5.4	17.9	17.0	69.8	68.3	11.6	0.42	5.313	9.73	2.680	25543	2.8
		3	5.5	5.4	17.8	17.3	69.8	68.9	11.9	0.30	5.313	10.04	2.680	25543	3.1
		4	5.5	5.5	17.9	17.4	69.5	68.6	11.9	0.30	5.313	10.04	2.680	25543	3.1
		1	6.0	5.7	19.2	18.4	70.2	68.8	12.6	0.42	5.313	10.75	2.680	25543	3.5
		2	6.0	5.7	19.2	18.3	70.4	69.0	12.6	0.42	5.313	10.75	2.680	25543	3.5
		1	6.5	5.7	20.2	19.7	71.7	71.1	14.0	0.30	5.313	12.10	2.680	25543	6.5
		2	6.5	5.6	20.1	19.6	72.0	71.3	14.0	0.30	5.313	12.10	2.680	25543	6.5

APPENDIX E:
SURFACE AREA CALCULATION

Generally, the surface area of the total aggregate is calculated based on the gradation of the aggregate or blend of aggregates. This calculation consists of multiplying the total percent passing each sieve size by a "surface area factor" as set forth in Table E1. Sum these products and the total will represent the equivalent surface area of the sample in term of m^2/kg (ft^2/lb). It is important to note that all the surface area factors must be used in the calculation. Also, if a different series of sieves is used, different surface area factors are necessary.

Table E1. Surface Area Factors *(32)*.

Total percent passing sieve No.		Maximum size	4.75 mm	2.36 mm	1.18 mm	600 µm	300 µm	150 µm	75 µm
			No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200
Surface area factor*	m^2/kg	.41	.41	.82	1.64	2.87	6.14	12.29	32.77
	($ft^2/lb.$)	(2)	(2)	(4)	(8)	(14)	(30)	(60)	(160)

The following example demonstrates the calculation of surface area by this method.

Sieve size		Percent passing	×	Surface area factor		=	Surface area	
SI- mm	US			m^2/kg	ft^2/lb		m^2/kg	ft^2/lb
19.0	¾ in	100		.41	2		.41	2
9.5	3/8 in	90						
4.75	No. 4	75		.41	2		.31	1.5
2.36	No. 8	60		.82	4		.49	2.4
1.18	No. 16	45		1.64	8		.74	3.6
0.06	No. 30	35		2.87	14		1.00	4.9
0.03	No. 50	25		6.14	30		1.54	7.5
0.015	No. 100	18		12.29	60		2.21	10.8
0.075	No. 200	6		32.77	160		1.97	9.6
				Surface Area =			8.67 m^2/kg	42.3 ft^2/lb