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Development Of A New Mix Design Method And Specification Requirements For Asphalt Treated Bases

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DEVELOPMENT OF A NEW MIX DESIGN METHOD AND SPECIFICATION REQUIREMENTS FOR ASPHALT TREATED BASES

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Dedication

This thesis is dedicated to the most beloved people in my life who supported me and gave strength throughout my studies.

My Mother
Guillermina Sandoval

My Father
Humberto Antonio Hernandez

My Brothers
Humberto and Jose Luis Hernandez

And to my Wife
Paulina Espinosa de los Monteros

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Abstract

Asphalt stabilized bases in Texas are usually designed and constructed as per Item 292, "Asphalt Treatment (Plant Mixed)," of the 2004 Standard Specification book. This specification is a hybrid of base and hot mix asphalt concrete procedures and requirements, which are sometimes incompatible. In addition, this Item uses a specific Texas Gyratory compactor that is not readily available to all districts. Some districts have started using test method Tex-204-F, Part III, 'Mix Design for Large Stone Mixtures Using the Superpave Gyratory Compactor." However, this procedure was originally developed to design Type A and Type B hot mix at 96% density and produce a 6 in. by 4.5 in. specimen. Under Item 292, the unconfined compressive strength of the mix (as per Tex-126-E) is used to assess the quality of the mix. Specimens prepared under Tex-204-F are not the appropriate size for this type of testing. As such, the quality of the mix is assessed with the indirect tensile strength. A new mix design procedure is needed for this type of material that can use standard equipment such as the Superpave Gyratory Compactor (SGC) to mold the specimens for mix design.

In order to achieve the objective of this project, current TxDOT procedures such as Tex-126-E and Tex-204-F were evaluated and modified in order to propose new Tex-126-H and Tex-204-H specifications. A comprehensive parametric study comparing the results of the two proposed specifications was performed. The impact of the number of gyrations, curing temperature, binder grade, and asphalt content variation were evaluated using prepared laboratory specimens. Parameters including density, unconfined compressive strength, indirect tensile strength, and modulus using the existing and proposed specifications were compared. Based on these studies, a new method for determining the Optimum Asphalt Content (OAC) for ATBs was developed. The recommendations were then evaluated at six actual construction projects for reasonableness.

The most practical set-up for laboratory tests was achieved using Tex-204-H specifications, which proposes preparation of 6 in. diameter and 4.5 in. high specimens using 75

gyrations of the SGC. Furthermore, it is recommended to cure specimens for 24 hrs at room temperature (77°F) before conducting the indirect tensile strength because the results from this procedure were more sensitive to asphalt content while reducing the mix design period. The appropriate asphalt content should satisfy a target indirect tensile strength of at least 85 psi, and a relative density of 97%.

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Chapter One: Introduction

One of the alternative stabilized bases that are available to TxDOT districts is the asphalt-treated bases (ATB) that falls under Item 292 of current specifications. According to the National Asphalt Pavement Association (NAPA), ATB is a dense-graded HMA with a wide gradation band and lower asphalt content (2.5% - 4.5%) intended for use as a base course. Also according to NAPA, ATB should costs less than typical HMA mixes because it can be produced with less expensive aggregates and lower percentages of asphalt binder. ATB can also provide a waterproof barrier to prevent fines infiltration into the subgrade and pavement structure and an alternative to untreated base material.

A review of the specifications of all fifty highway agencies that are incorporated in the FHWA National Highway Specifications indicates that most agencies either favor designing the ATB's using the HMA specifications, or using emulsion instead of asphalt. Moreover, most mix designs are carried out by using Superpave Gyratory Compactor (SGC), the Hveem method or the Marshall method. Similarly, newest version of FAA Item P-403 has reverted to Marshall mix design with the additional requirements of the Tensile Strength Ratio (TSR) from ASTM D 4867.

Current TxDOT construction activities have pointed out that about half a dozen districts place ATB's, with the Houston and Beaumont Districts being by far the most frequent users. However, most districts utilize Tex-204-F to achieve their mix designs primarily because the compactor specified in Item 292 is not available to all districts. Therefore, an updated mix design for ATB is in high need. To develop modern test protocols for designing ATB's, the first consideration is to determine whether the ATB should be designed and used as a high-quality base (similar to stabilized or emulsion bases) or as a lower quality hot mix (as compared to Types A and B asphalt mixes). The mix design requirements for these two alternatives are different, which in turn will impact the mix design process from compaction of specimens to their performance testing. No matter which alternative (high-quality base or low-quality hot

mix) is pursued, for the ease of operation, the mix design should be compatible with current practices of TxDOT to minimize the learning curve for the technicians and the acquisition of new equipment.

One major criterion for the new mix design should be the compatibility between the parameters measured in the lab and the similar parameters obtained from the as-constructed layers. Currently, quality management of ATB's during construction is for the most part is based on the measured density under Tex-207-F using a nuclear density gauge on the field, or in the lab on cores or prepared samples retrieved from the site. There is some concern that the densities measured in those fashions may not be accurate or repeatable because of the nature of the base material (limited control on the gradation) used in ATB's. Since the accurate measurement of the maximum theoretical density of the mix measured as per Tex-227-F directly impacts the relative density of the ATB, the accuracy and precision of this test for use with the ATB needs also to be evaluated.

Based on this background, our goals in this project are to achieve the following items:

- Define criteria and procedures that are compatible with production and placement requirements of ATB,
- Develop a new simple, efficient, and consistent mix design method that uses commercially available equipment.
- Develop draft laboratory specification requirements for better quality control of asphalt treated base.

1.1 OBJECTIVES AND SCOPE

The main goal of this project was to develop a laboratory test protocol to help in selecting the optimum asphalt content and a guideline or draft specification for the construction of asphalt-treated bases. To achieve this goal, the following objectives were addressed:

1. Document different uses and establish the most appropriate uses of the ATB in Texas considering both the engineering and economical consideration.

2. Evaluate the reasonableness, strengths and weaknesses of current practices (e.g. Item 292, Tex-126-E, and Tex-204-F) in terms of mix design and construction practices.
3. Evaluate and establish the most appropriate performance indicators and ways of designing ATB's in the laboratory based on the intended use and identified performance indicators.
4. Evaluate the best method of compacting the specimens given the existing available equipment in TxDOT such as the Superpave Gyratory Compactor (SGC) instead of the Texas gyratory compactor (TGC) specified in Item 292.
5. Develop the compaction criteria in terms of energy and/or number of gyrations that is more representative of this type of mixes in the field.
6. Evaluate, and if necessary modify, the process of determining the optimum asphalt content for these mixes from the process enumerated in Tex-126-E or Tex-204-F.
7. Establish the minimum strength characteristics of ATB and the best way to measure them so that the value-added benefits of the binder added to the mix is best represented.
8. Incorporate tests (e.g. moisture susceptibility) to ensure long-term performance of the mix.
9. Evaluate and recommend the best construction practices with especial attention to the reasonableness and accuracy of the current quality management process.
10. Verify the mix design and the results from laboratory testing by evaluating the field performance.
11. Monitor and record the initial performance of pavement sections with ATB.

1.2 ORGANIZATION

Chapter Two contains a thorough literature review of studies addressing current compaction methods; specifications used in Texas and many other states and countries; performance of Asphalt Treated Base (ATB), and cost benefit of the use of the ATBs.

Chapter Three presents the result of district survey conducted at the beginning of this research. The districts that previously have used ATB were identified. Construction and laboratory specifications and compactors used by those districts were as well identified. The survey also collected the main uses of ATB, factor to motivate its use, type of binder grade and aggregate, and criteria and problems found during design or construction of ATB.

Chapter Four reflects the work done at the early middle of the project in order to compare current (Tex-126-E and Tex-204-F) and proposed (Tex-126-H, Tex-204-H) protocols. Materials were retrieved from different districts to prepare specimens using both: Texas and Superpave Gyratory compactors; the results were analyzed and compared. The properties of each material obtained were also addressed.

Chapter Five contains a detailed evaluation of the parameters that impact the performance of a mix for the proposed mix design protocols. The measured impact of the number of gyrations, curing temperature, binder grade, change in gradation, and asphalt content variation to the property of the mixes for the proposed procedures were addressed.

Chapter Six describes the mix design protocol selected. The compactor, specimen size, density calculation, number of gyrations determined to meet the new protocol requirements are presented. Strength parameter, curing and testing temperature, density and strength requirements and the new method for determining the optimum asphalt content are also showed.

Chapter Seven contains the information obtained from field investigation conducted at several sites in order to evaluate the results of the proposed protocol. The laboratory results and properties of the materials acquired from every field sites are also discussed.

Chapter Two: Review of Literature

The performance of a pavement depends on many factors such as the properties of the materials used, structural capacity of the pavement, construction method, traffic loading, and climatic conditions. For flexible pavements, the quality of the base layer is one of the most important factors. Previous research has found that much of the distress that flexible pavements experience can be traced to problems encountered in the base (Saeed et al., 2001). The use of a cost efficient base layer, that would extend the pavement life, that would require less thickness, or that would use local materials, is highly desirable. Asphalt-treated bases (ATB) fit this category.

According to the National Asphalt Pavement Association (NAPA), (http://training.ce.washington.edu/wsdot/Modules/02_pavement_types/02-3_body.htm), ATB is a dense-graded hot mix asphalt (HMA) with a wide gradation band with a lower asphalt content that can be used as a base course. Among the features that make it different from HMA are (Wong et al., 2004):

1. HMA layer may receive direct impact from traffic and experience more serious weather conditions than those experienced by the ATB
2. Thickness of ATB is greater than the one of the HMA
3. Asphalt layer can be superimposed to improve surface performance, or removed to construct a new surface layer when the damage has attained certain level

These differences make the ATB's design vital, since it must preserve necessary performance in the entire pavement life.

One of the main problems with asphaltic pavements that include a stabilized base course is the incidence of transverse cracking in the stabilized base and the related propagation of cracks to the surface, which diminishes the life of pavements. In this case, ATB may be more flexible and resistant to fatigue cracking as compared to cement stabilized bases (Dykman et al., 2003).

Besides new construction, ATB can be beneficial in rehabilitation projects. According to Dykman et al. (2003), the crucial factors in choosing ATB as a way of rehabilitation are:

- Quick construction time, therefore decreased delays and diminished traffic disturbance
- Low permeability to moderate pore water pressure effect (Cedergren, 1977)
- Minimal moisture sensitivity as a consequence of moisture ingress
- Quite flexible, consequently decrease the possibility of reflective cracking (Marks and Heisman, 1985.)

Research in ATBs has been limited even though ATBs have been used as structural pavement layers for more than 40 years (Dykman et al., 2003). McDowell and Smith (1969) performed the first comprehensive study in design and construction of ATB (also known as black base). An important objective of their study was to observe the effects of rate of loading on the unconfined compressive strength (UCS) of ATB. McDowell and Smith used loading rates of 6, 8, 10 and 15 in./min, noticing that when testing at fast rates of loading a definite improvement in compressive strength of asphalt treated materials over untreated materials was obtained. They stated that a fast rate of loading test should become part of the analysis of asphalt mixtures.

McDowell and Smith (1969) also studied the effects of moisture absorption on strength and its relation to total percent voids. They used pressure pycnometer to obtain saturation in the least amount of time. They concluded that mixtures having less than 5.5% total voids will possibly not lose strength due to absorption of moisture. While McDowell and Smith investigation was taking place, the Texas Gyratory Compactor (TGC) was revised so that 6 in. in diameter by 8 in. in height specimens could be prepared.

2.1 COMPACTION METHODS

An analysis of the specifications of all fifty highway agencies that are incorporated in the Federal Highway Administration (FHWA) National Highway Specifications indicates that most highway agencies either support designing the ATBs using the HMA specification (e.g., Illinois,

Indiana, Washington), or utilizing emulsion rather than asphalt (e.g., Delaware, Maine, Maryland). However, several transportation agencies are upgrading their conventional compaction methods such as TGC or Marshall with SGC for routine mix design. According to Button et al. (2006) the advantages of using the SGC include the following items:

- Its ability to estimate the compatibility of mixes from density during the compaction process
- Its ability to identify weak aggregate structures that collapse very quickly to lower air voids
- Improved reproducibility of samples due to mostly mechanical control of compaction process
- Ability to simulate field compacted mixes relatively better than other compaction methods

Gyratory compaction was originally created in 1939 by the Texas Highway Department to help in the molding and design of asphalt mixtures (Harman et al., 2002). According to Sebesta et al. (2004), a more advanced Texas Gyratory Compactor has also been used to compact soil samples which were subsequently used in laboratory swell tests.

Gyratory compactors were designed to simulate the orientation of aggregate, degradation of aggregate, field compaction, and traffic degradation that occurs in HMA during production, compaction and traffic loading (Collins et al., 1997). Dykman et al. (2003) state that gyratory compaction can simulate the action of a roller in the field due to its capability to rotate the principal stresses. On the other hand, gyratory compaction can create totally different compaction characteristics. These characteristics depend on the adjustment and calibration of several parameters that affect the degree of compaction of laboratory HMA specimens (Mokwa et al., 2008).

According to Button et al. (2006), the lower angle of gyration of the SGC (1.25°) imparts significantly less mechanical energy into the specimen per gyration as compared to the TGC (5.8° gyration angle). Different angles of gyration have different influence on the orientation of

the aggregates, particularly the larger aggregates. The differences between specimens prepared using the TGC and SGC such as: air void structure, aggregate orientation, voids in mineral aggregate (VMA) and density gradient, will not likely be consistent because these differences will depend on the shear resistance of the mixture.

Mokwa et al. (2008) used confining pressures ranging from 30 psi to 90 psi in preparing specimens with a SGC. They found that mixes with smaller particles sizes exhibited higher rates of densification as a result of higher confining pressures. The confining pressure applied to the specimen according to Tex-126-E varies from 20 psi to 60 psi for the TGC.

Aguiar-Moya et al. (2007) indicate that the basis for the number of gyrations is that the compactive effort obtained in the laboratory should produce the same outcome on the asphalt mixtures (to increase density) as traffic loads for in-place asphalt mixes. As more weight is assigned to fatigue resistance, the optimum number of gyrations decreases, therefore producing mixes with higher binder contents. Similarly, as more significance is given to rutting, the optimal number of gyrations increases. Button et al. (2006) indicate that the design gyrations in SGC can be reduced below the initial recommendations without compromising rutting resistance of HMA mixtures.

Although the SGC can produce the same volume of air voids as the TGC in a given mixture type, the resulting optimum asphalt contents and engineering properties of the compacted mixtures may be measurably different because of different aggregate orientations and different density gradients within the specimens (Button et al., 1994; Von Quintus et al., 1991).

According to NAPA, http://training.ce.washington.edu/wsdot/Modules/05_mix_design/05-3_ body.htm) the Hveem method has been proven to produce quality HMA from which long-lasting pavements can be constructed. Hveem method has the following six main steps:

1. Aggregate selection
2. Asphalt binder selection
3. Sample preparation using the California Kneading Compactor (Figure 2.1)

4. Stability and cohesion determination using a Stabilometer and Cohesiometer (Figure 2.2)
5. Density and voids calculations, and
6. Optimum asphalt binder content selection.

The basic concept of the Marshall mix design method was originally developed by Bruce Marshall of the Mississippi Highway Department around 1939 and then refined by the U.S. Army. The Marshall method is very popular because of its relatively simplicity, economical equipment and proven record. Similar to Hveem method, the Marshall method consists of the following six basic steps:

1. Aggregate selection
2. Asphalt binder selection
3. Sample preparation using the Marshall Hammer (Figure 2.3)
4. Stability and Flow Test using the Marshall Stability testing apparatus (Figure 2.4)
5. Density and voids calculations, and
6. Optimum asphalt binder content selection

Figure 2.1: Kneading Compactor.

a) Stabilometer b) Cohesiometer

Figure 2.2: Hveem Machine used for Hveem Mix Design Method.

According to Wong et al. (2004) conventional Marshall method is not appropriate for ATB mixture design for the reason that the maximum size of aggregate and the thickness of ATB may not be comparable to those of asphalt layer.

Figure 2.3: Marshall Hammer.

Figure 2.4: Marshall Stability.

2.2 TxDOT Specifications

An evaluation of TxDOT construction activities pointed out that about six out of twenty-five districts place ATBs, with Houston and Beaumont Districts being the most recurrent users. In Texas, ATBs are traditionally designed and constructed as per Item 292 "Asphalt Treatment (Plant Mixed)," of the 2004 Standard Specification book. Since the compactor under Item 292 is not available to all districts, some districts have started using Tex-204-F, Part III 'Mix Design for Large Stone Mixtures Using the Superpave Gyratory Compactor."

2.2.1 Item 292: Asphalt Treatment (Plant-Mixed)

Table 2.1 shows the mix requirements for Item 292. This specification is a hybrid of base and hot mix asphalt concrete procedures and requirements. Under Item 292, the aggregates basically have the same gradation and quality as Item 247 for untreated base, which is less rigorous than those utilized in Type A/B mixes under Item 341 or 344. Aggregate quality requirements for Item 292 are reflected in Table 2.2. Item 292 permits the use of crushed concrete in the mix, which is usually unacceptable in the Type A/B mixes. It seems that the main incentive for the utilization of Item 292, as stated by NAPA, may be to incorporate local materials (raw or recycled) in the local construction.

Table 2.1: Mix Requirements for ATB as per Item 292.

Master Gradation Bands Tex-200-F, Part I, % Passing by Weight				
Sieve Size	**Grade 1**	**Grade 2**	**Grade 3**	**Grade 4**
1-3/4"		100	100	As shown on the plans
1-1/2"	100	90-100		
1"	90-100			
3/8"	45-70			
#4	30-55	25-55		
#40	15-30	15-40	15-40	
Strength Requirements				
Slow strength, psi, min.[1]	50	40	30	30 [2]

1. At optimum asphalt content
2. Unless a higher minimum strength is shown on the plans

Under Item 292, 3% to 9% binder is suggested for the ATB; however, based on our review of several current mix designs from several districts, the optimum binder content is about 4% to 5%. For Type A/B mixes the optimum binder content is typically 5% to 6%. The unconfined compressive strength of the mix (as per Tex-126-E) is used to assess the quality of the mix. Since the placement of the mixes under Item 292 and 341/344 is similar (with less strict field quality management for Item 292), it seems that ATB cost should be slightly less than Type A/B mixes.

2.2.2 Tex-126-E: Molding, Testing, and Evaluating Bituminous Black Base Materials

This method is used to mold an asphalt stabilized (black base) material, and to determine the relationship between asphalt content and density (this relationship is called an asphalt-density curve) and similarly an unconfined compressive strength-density curve. The compacted black base specimens are made in duplicates and are tested for their unconfined compressive strengths

Table 2.2: Aggregate Quality Requirements.

Property	Test Method	Specification Requirement
Wet ball mill, % max	Tex-116-E	50
Max increase, % passing #40		20
Liquid Limit, max	Tex-104-E	40
Plasticity Index, max	Tex-106-E	10
Sand Equivalent, % min	Tex-203-F	40

at 140°F. The specimens are subjected to two types of deformation rates: a slow (0.15 in./min) and a fast deformation rate (10 in./min). Specimens tested at a slow deformation rate yield relatively lower strengths as compared to the fast deformation rate. From the strength-density relationship, the minimum density that would satisfy the unconfined strength requirements as per Item 292 is taken as the minimum allowable density.

2.2.3 Tex 204 Part III: Mix Design for Large Stone Mixtures Using Superpave Gyratory Compactor (SGC)

This procedure was originally developed to design Type A and Type B hot mixes at 96% density from a 6 in. by 4.5 in. specimens as per Table 2.3. Depending on the PG grade of the binder, the material, mixing, and compaction temperatures are according to Table 2.4. The specimens are molded to either 100 gyrations, or as shown on the plans, or as per Table 2.5.

The specimens prepared under Tex-204-F are not the appropriate sizes for unconfined compressive strength (as per Tex-126-E). As such, the quality of the mix assessed with the indirect tensile strength at a deformation rate of 2 in./min. at a temperature of 77 ± 2°F according to Tex-226-F.

Table 2.3: Minimum Size of Samples as per Tex 204 Part III.

Nominal Maximum Size of Particles, Passing Sieve	Minimum Weight of Sample for Test, lb
Coarse Aggregate	
2"	8
1-1/2"	8
1"	6
3/4"	4
1/2"	3
3/8"	2
Fine Aggregate	
#4	1.1
#8	1.1

Table 2.4: Material, Mixing, and Compacting Temperatures as per Tex 204 Part III.

PG Grade	Asphalt Material Temperature, °F	Mixing Temperature, °F	Compaction Temperature, °F
64-22	290	290	250
64-28	300	300	275
70-22	300	300	275
70-28	325	325	300
76-16	325	325	300
76-22	325	325	300

Table 2.5: Compaction Parameters as per Tex 204 Part III.

Design ESALs[1] (million)	Compaction Parameters			Typical Roadway Application [2]
	$N_{initial}$	N_{des}	$N_{maximum}$	
<0.3	6	50	75	Applications include roadways with very light traffic volumes such as local roads, county roads, and city streets where truck traffic is prohibited or at a very minimal level. Traffic on these roadways is local in nature, not regional, intrastate, or interstate. Special purpose roadways serving recreational sites or areas may also be applicable to this level.
0.3 to <3	7	75	115	Applications include many collector roads or access streets. Medium-trafficked city streets and the majority of country roadways may be applicable to this level.
3 to <30	8	100	160	Applications may include many 2-lane, multilane, divided, and partially or completely controlled access roadways. Among these are medium to highly trafficked city streets, many state routes, US highways, and some rural interstates.
≥30	9	125	205	Applications include the vast majority of the US Interstate System, both rural and urban in nature. Special applications such as truck-weighing stations or truck-climbing lanes on 2-lane roadways may also be applicable to this level.

[1] Design ESALs are the anticipated project traffic level expected on the design lane over a 20-yr. period. Regardless of the actual design life of the roadway, determine the design ESALs for 20 yr., and choose the appropriate N_{des} level

[2] Typical Roadway Applications as defined by *A Policy on Geometric Design of Highway and Streets*, AASHTO

2.3 OTHER DOTS CURRENT SPECIFICATIONS

As mentioned before some agencies either support designing the ATBs using the HMA specifications, or utilizing emulsion rather than asphalt. Other agencies specify design processes for asphalt treated permeable bases, which are out of the scope of this project. However, few states have specifications for the ATB design.

2.3.1 Alaska

According to Alaska Department of Transportation (AKDOT) the ATBs are designed under Section 306 "Asphalt Treated Base Course." The selection of aggregate is mainly determined by AASHTO requirements presented on Table 2.6. In Alaska, the aggregate requirements shown on Table 2.7 are dictated by AASHTO T 27/T 11.

Table 2.6: Aggregate Mix Requirements.

Property	Base Course	Test Method
L.A. Wear, %	50, max.	AASHTO T 96
Degradation Value	45, min.	ATM 313
Fracture, %	70, min.	WAQTC FOP for AASHTO TP 61
Liquid Limit	---	WAQTC FOP for AASHTO T 89
Plastic Index	6, max.	WAQTC FOP for AASHTO T 90
Sodium Sulfate Loss, %	9, max. (5 cycles)	AASHTO T 104

An important point mentioned in the Alaskan specification is the weather limitations, which states not to place the asphalt mixture on a wet or frozen surface, or when weather conditions will prevent proper handling, compacting or finishing of the mixture. It also states not to place the asphalt mixture unless the air temperature is above 40 °F, as measured in the shade and away from any heat sources.

2.3.2 Arkansas

Arkansas Department of Transportation's (ARDOT) design of ATB falls under Section 417 "Open Graded Asphalt Base Course." Besides the size of the aggregates requirements, there is a criterion for asphalt content for each grade type as shown on Table 2.8.

Table 2.7: Aggregate Base Gradations in Alaska.

Sieve	Gradation	
	C-1	D-1
1-1/2"	100	--
1"	70-100	100
3/4"	60-90	70-100
3/8"	45-75	50-80
#4	30-60	35-65
#8	22-52	20-50
#50	8-30	8-30
#200	0-6	0-6

2.3.3 Washington

Washington Department of Transportation (WSDOT) has its own specification for ATB design. This design has two general requirements:

1. Los Angeles Wear, 500 Rev. → 30% max.
2. Degradation Factor → 15 min.

When the aggregates are mixed within the limits of Table 2.9 and mixed in the laboratory with the designated grade of asphalt, the mixture shall be capable of meeting the following test values:

- Stabilometer Value → 30 min.
- Cohesiometer Value → 50 min.
- Modified Lottman Stripping Test → 80% min.
- Sand Equivalent Value → 30 min.

Table 2.8: Mix Requirements for ATB Design in Arkansas.

Sieve Size	Type 1	Type 2	Type 3	Type 4
3"	100			
2 1/2"	95-100			
2"		100		
1-1/2"	30-70 [±7]	75-90 [±7]		
1"				100
3/4"	0-15 [±7]	50-70 [±7]	100	90-100
1/2"			90-100	
3/8"	0-2			20-55 [±5]
#4		8-20 [±5]	0-15 [±5]	0-10
#8			0-3	0-5
#100		0-5		
Asphalt Content	1.5 - 4.0	1.5 - 4.0	1.5 - 4.0	2.5 - 3.0

Note: The number in brackets is the allowable tolerance from the mix design value

Table 2.9: Aggregate Requirements.

Sieve Size	Percent Passing
2"	100
1/2"	56-100
1/4"	40-78
#10	22-57
#40	8-32
#200	2.0-9.0
Asphalt Content	2.5 - 4.5

2.4 INTERNATIONAL REVIEW

2.4.1 Australia

A study in Queensland, Australia by Dykman et al. (2003) mentioned that due to the population growth, more emphasis is being placed on maintenance and rehabilitation processes. Due to the relatively thin granular base courses used, pavement rehabilitation techniques now prefer modification than stabilization. This involves using treatments that balance the materials' strength and ductility in order to produce strong but flexible granular layer so that fatigue cracking can be minimized. ATB is considered as an alternative for this purpose. Dykman et al. (2003) used Marshall and gyratory-compacted specimens. ATB mixes appeared to be fairly rut resistant and retained a high stiffness. This characteristic of ATB may improve the load distribution capacity of the pavement, leading to more effective protection of the underlying layers. Dykman et al. (2003) and Ullman and Nolan (1991) conclude that the following benefits can be obtained from ATB, if used properly:

- ATB can be placed with conventional equipment
- Fast construction
- Less cost than conventional hot-mix asphalt

- High stability
- Possibility of using marginal aggregates
- Potential for recycling

2.4.2 Singapore

Wong et al. (2004) stated that ATB has better strength and stability compared to common macadam base, and has good flexibility and durability. In the cases of high traffic volume and increasing axle loading on certain roadways, using ATB can be useful in respect to pavement performance. According to Wong et al., the determination of asphalt content to get the best performance is through establishing a balance between friction and cohesion.

Wong et al. (2004) also recommended a procedure that considered the variations of the maximum density, indirect tensile strength and unconfined compressive strength of a blend with asphalt content to obtain the optimum mix design. Through static creep and fatigue tests, they demonstrated that mixture using their design method had good resistance to permanent deformation and fatigue at the bottom of the base. They prepared their specimens using an SGC, with the number of gyrations being based on the locking point of the mix. As a conclusion from that study, Wong et al. (2004) recommended the use of compression test and indirect tensile test for ATB design since these tests are simple and convenient.

2.5 PERFORMANCE OF ATB

Some of the parameters that play an important role in the performance of the ATB as a system are the structural integrity of the section, the internal stability of the layer, the environmental conditions, and most importantly the quality of construction.

2.5.1 Structural Integrity

The structural integrity of a flexible pavement section is controlled by several parameters. In most classical structural design programs (such as FPS19), the design thickness of the layers is (directly or indirectly) estimated based on the criteria that the stresses at the interfaces of the hot mix and base, and the base and subgrade, are low enough so that the cracking and rutting will not be an issue. The traffic volume is also a major consideration. For a given traffic condition, the thicker the layers overlying the base, the thicker the base layer and the stiffer the subgrade are, the lower the base layer stresses will be. This indicates that not only the quality of base should be considered, the stiffness of the subgrade, and the thickness of the hot mix should also be considered. The complex modulus or diameteral resilient modulus tests can be performed for this purpose.

2.5.2 Internal Stability

The internal stability is defined as the excessive deformation of the base under the load. This manifests as rutting primarily associated with the base layer. To address this issue, repeated load permanent deformation lab tests as advocated by the FHWA should be used in conjunction with the appropriate models that predicts the rutting of the hot-mix, base, and subgrade layers individually.

2.5.3 Environmental Conditions

The main environmental parameter of interest is the adverse effects of moisture and temperature on the strength and modulus of the base. Therefore, the importance of considering the impact of moisture on the performance of the material should not be neglected. Hamburg wheel tracking device, perhaps with some relaxed requirements, can be used for this purpose. Alternatively, two inter-related methods can be used to assess the impact of moisture on the performance of the base: Tube Suction Test (Tex-145) and the Free-Free Resonant Column (proposed Tex-147) or V-meter (proposed Tex-259). The Tube Suction Test (TST) qualitatively provides an estimate of the water-retention of the base material that can be correlated to the

potential of damage to the base due to softening. The Free-Free Resonant Column (FFRC) or V-meter test is a quantitative nondestructive lab method that can be performed on a specimen for its modulus. In both methods, each specimen is oven-dried for two days and then allowed to soak moisture through capillary saturation. The modulus of the specimen is measured every day in conjunction with the tube suction test. The residual modulus corresponds to the modulus measured after the specimen soaked moisture for several days. Since the same specimen is used throughout for both TST and FFRC tests, the variation in moisture content with time can also be obtained by weighing the specimens daily. In that manner, the moisture retention properties of the material and its impact on modulus can be measured. Of course separate specimens should be prepared and tested for strength and modulus at optimum and after moisture conditioning to determine the retained strength and retained modulus for conventional design.

2.5.4 Quality of Construction

No matter how much attention is focused on the mix design, the performance of the mix is directly related to the quality of construction. The necessity to perform a thorough evaluation of the component materials, and a thorough testing regimen and an aggressive quality control/ quality assurance program is well understood by TxDOT and is incorporated in the appropriate specifications. One of the major quality management tool used is the density of the in-place mat. To successfully assess the density of the mat, two parameters are necessary: The theoretical maximum specific gravity of the mixture (Tex-227-F) and the bulk specific gravity of the cores obtained from the finished mat (or density with the NDG) as per Tex-207-F. Both of these methods have been the subject of numerous studies by the federal and state highway agencies.

There is some concern that the densities measured in those fashions may not be accurate or repeatable because of the nature of the base material (limited control on the gradation) used in ATBs. The most recent and comprehensive study of this matter has been carried out under a multi-year, multi-phase project (NCHRP 9-26) that was completed in 2007 by the AASHTO Materials Reference Laboratory (AMRL). The report from Phase 1 of Project included precision

estimates of selected volumetric properties of HMA using non-absorptive aggregates by (Spellberg and Savage, 2003). The report from Phase 2 discusses the results of an investigation into the cause of variations in HMA bulk specific gravity test results using non-absorptive aggregates by Spellberg and Savage (2004). The report from Phase 3 includes a robust technique developed by AMRL for analyzing proficiency sample data for the purpose of obtaining reliable single-operator and multi-laboratory estimates of precision (Holsinger et al., 2005). According to Azari et al. (2006) the report from Phase 4 includes the precision estimates of selected volumetric properties of HMA using absorptive aggregates, and the effect of aging period on the volumetric properties of the absorptive aggregates. The Phase 5 report includes the update precision estimates for AASHTO Standard Test Method T269, "Percent Air Voids in Compacted Dense and Open Asphalt Mixtures." These reports are summarized below.

The inter-laboratory repeatability and reproducibility estimates of several common volumetric properties were established. A coarse (maximum aggregate size of 19.0 mm) mix and a fine mix (maximum aggregate size of 12.5 mm) were considered. The following two different methods were considered for the theoretical maximum specific gravity:

- ASTM D6857 "Specific Gravity and Density of Bituminous Paving Mixtures Using Automatic Vacuum Sealing Method" and
- ASTM D2041 (equivalent to Tex-227-F Part II) "Standard Test Method for Theoretical Maximum Specific Gravity and Density of Bituminous Paving Mixtures."

Based on the results from 26 laboratories, the values obtained from ASTM D6857 were found to be lower than those obtained following ASTM D2041 (equivalent to Tex-227-F Part II). The author mentioned that the variability in ASTM D6857 can be attributed to some problems while performing the test like: the bags touching the sides of the bath during the weighing, or an incomplete evacuation of air during sealing process. This variability consequently produced a statistically significant difference in the repeatability and reproducibility of both tests.

The bulk specific gravity was also measured using the following two other methods:

- ASTM D6752 (equivalent to Tex-207-F Part VI) "Bulk Specific Gravity and Density of Compacted Bituminous Mixtures Using Automatic Vacuum Sealing Method" and
- AASHTO T166 (equivalent to Tex-207-F Part I) "Bulk Specific Gravity of Compacted Asphalt Mixtures Using Saturated Surface-Dry Specimens."

The specimens tested under ASTM D6752 (equivalent to Tex-207-F Part VI) had a lower average specific gravity as determined from tests performed using AASHTO T166 (equivalent to Tex-207-F Part I). Using student T-test statistical analyses, there was a good reproducibility for both coarse and fine specimens tested under AASHTO T166 (equivalent to Tex-207-F Part I); however the F-test statistical analyses showed a statistically significant difference between the repeatability of both mixes. Variability was attributed to incomplete evacuation of air during sealing process and pinholes developing in the bags after the vacuum sealing process. This study also stated that there was a significant difference between the densities of different compactors. For example, the specimens compacted with a Troxler 4140 compactor had a lower density than the specimens compacted using a Pine AFGC125X. The results of this inter-laboratory study also indicated that the within laboratory variation in the bulk specific gravity test results was much greater than the variation in the maximum theoretical specific gravity test results.

The second phase of this study focused on the variability of the bulk specific gravity. Instead of each laboratory compacting its own specimens as done in Phase I, the necessary specimens were produced at the same facility to minimize the compaction errors. Data from 21 laboratories, each one testing specimens with maximum size of 9.5-mm, 12.5-mm, and 19.0-mm showed that overall the AASHTO T166 (equivalent to Tex-207-F Part I) measurements were precise. Comparing data from Phases I and II, the authors conclude that 90% of the variation in AASHTO T166 (equivalent to Tex-207-F Part I) bulk density test results can be attributed to the mixing and compaction process.

The next phase of this inter-laboratory study focused on the development of precision statements of absorptive aggregates applicable to the following tests:

- AASHTO T312 (equivalent to Tex-241-F Part I) "Preparing and Determining Density of HMA Specimens by Means of the SGC",
- AASHTO T166 (equivalent to Tex-207-F Part I) "Bulk Specific Gravity of Compacted Asphalt Mixtures Using Saturated Surface-Dry Specimens",
- ASTM D2041 (equivalent to Tex-227-F Part II) "Theoretical Maximum Specific Gravity and Density of Bituminous Paving Mixtures", and
- ASTM D6752 (equivalent to Tex-207-F Part VI) "Bulk Specific Gravity and Density of Compacted Bituminous Mixtures Using Automatic Vacuum Sealing Method"

According to the authors, the precision statement for AASHTO T312 (equivalent to Tex-241-F Part I) should differ using absorptive aggregates and non-absorptive aggregates based on the significant difference between repeatability and reproducibility estimates for relative density measurements obtained in this study. The conclusion of the authors was that the repeatability and reproducibility estimates of the theoretical maximum specific gravity are significantly different using absorptive and non-absorptive aggregates. They suggested following ASTM D2041 Section 11 "Supplemental Procedure for Mixtures Containing Porous Aggregate" (equivalent to Tex-227-F Part II) for absorptive aggregates mixtures.

For Bulk Specific Gravity AASHTO T166 (equivalent to Tex-207-F Part I) and ASTM D6752 (equivalent to Tex-207-F Part VI) were compared. The average specific gravity obtained using ASTM D6752 (equivalent to Tex-207-F Part VI) was significantly lower than the average specific gravity determined by AASHTO T166. However, the repeatability and reproducibility estimates of both tests were comparable for absorptive and non-absorptive mixtures. The authors also stated that it was appropriate to combine repeatability and reproducibility estimates of ASTM D6752 (equivalent to Tex-207-F Part VI) and AASHTO T166 (equivalent to Tex-207-F Part I) for coarse and fine mixtures.

Another objective of this phase was to examine the effect of aging time on bulk specific gravity, maximum specific gravity, percent air voids, and percent absorbed asphalt of mixtures

with absorptive aggregates. Mixtures with more absorptive aggregates were found to be more susceptible to aging. Since the increase in aging time would allow more absorption and additional hardening of asphalt binder, this would reduce the compactability and consequently increase of percent air voids in the compacted mixtures. They recommended that the laboratory aging time should not exceed 4 hrs to prevent a significant change in the original mixture design.

The final phase of the study consisted of updating the estimates for AASHTO Standard Test Method T269 "Percent Air Voids in Compacted Dense and Open Asphalt Mixtures." The specimens used in that study were compacted by means of a Marshall apparatus, a California Kneading Compactor, a Gyratory Shear Compactor, and ta Superpave Gyratory Compactor. The authors recommended the precision limits for AASHTO T269 to be determined based on precision of percent air voids since this parameter is considered the controlling property for design and quality control of asphalt mixtures. Based on this study the Marshall apparatus and Superpave Gyratory Compactor precision estimates were in good agreement with the previous estimates presented in AASHTO T269 Section 8.3 "Criteria for Judging the Acceptability of Percent Air Voids Test Results that are obtained by Using T166 and T209 for Non Porous Aggregates". On the other hand, precision estimates computed using the California Kneading and Shear Gyratory Compactors were significantly larger than the ones previously stated on Section 8.3. Table 2.10 shows the precision estimates of this study based on the repeatability and reproducibility of the data.

2.6 COST-BENEFIT

According to NAPA, ATB should cost less than typical HMA mixes since it can be produced with less expensive aggregates and lower percentages of asphalt binder. Among the advantages of ATB, as stated by NAPA, on a site that must export material (excess cut), an ATB pavement can save a considerable amount of excavation, hauling and disposal costs. Furthermore, on a site that must import material (excess fill), ATB can be used to build the pavement over more marginal subgrades. Another benefit from ATB is that it provides a water

proof barrier to prevent fines infiltration into the subgrade and pavement structure. According to NAPA, if water accumulates in the subgrade, the repetition of pavement loading may cause subgrade fines to migrate into the base and pavement structure. This can clog the base layer, which blocks drainage and create voids in the subgrade into which the pavement may settle. One additional valuable characteristic of ATB is the alternative to untreated base material. According to NAPA, ATB is structurally up to three times as strong as an untreated granular base. NAPA states that it may be feasible to use thinner layers for the same structural support.

Table 2.10: Precision Estimates.

Compaction Method	Specimen Diameter, in	Standard Deviation (1s)[a]	Acceptable Ranged of Two Test Results (d2s)[a]
Single Operator Precision:			
Marshall Apparatus[b]	4	0.48	1.36
California Kneading Compactor	4	0.52	1.47
Gyratory Shear Compactor	4	0.50	1.42
Superpave Gyratory Compactor	6	0.47	1.33
Multilaboratory Precision:			
Marshall Apparatus[b]	4	1.08	3.06
California Kneading Compactor	4	1.39	3.94
Gyratory Shear Compactor	4	1.49	4.22
Superpave Gyratory Compactor	6	1.01	2.86

[a] These values represent the 1s and d2s limits described in ASTM Practice C670.
[b] The results reported for specimens compacted using T245 were determined as the average of three specimens.
Note: The precision estimates given in the table are based on he analysis of test results from three pairs of AMRL proficiency samples. The data analyzed consisted of results from 20 to 578 laboratories for each of the three pairs of samples. The analysis included three binder grades: PG 70-22, PG 64-10, PG 64-22. Average results for air voids ranged from 2.37% to 7.95%. The details of this analysis are in NCHRP Final Report, NCHRP Project No. 9-26, Phase V.

In accordance with feedback from local authorities in Queensland, Australia, ATBs have performed very well over a period of ten years and in particular, its value for money when compared to other base stabilization treatments (Dykman et al.,2003). According to McDowell and Smith (1969) the economic benefit of ATB over other hot mixes is generally dependent on the use of local base materials, which do not have to be washed and sieved before batching.

Chapter Three: District Survey

A survey was conducted to identify the activities related to the use of Asphalt Treated Base (ATB) throughout Texas, and to identify possible sites to be incorporated in this study. The questionnaire is included in the appendix at the end of this report.

Survey responses were received from the following 18 districts: Abilene, Amarillo, Atlanta, Austin, Brownwood, Bryan, Childress, Dallas, Fort Worth, Houston, Laredo, Lufkin, Paris, Pharr, San Angelo, San Antonio, Wichita Falls, and Yoakum. The responses to the survey questions so far are summarized in this section.

Question 1: Have you used or are you using Asphalt Treated Base (ATB) in construction/ rehabilitation projects in your district?

Six districts (Abilene, Houston, Paris, Pharr, San Angelo, and Wichita Falls) reported they had used or were using ATB in their projects. Based on information from lettings in Site Manager, the districts that had recently used Item 292 are as per Table 3.1. The districts that have used Item 345 are summarized in Table 3.2.

Out of the responses obtained, six districts (Abilene, Houston, Paris, Pharr, San Angelo, and Wichita Falls) reported they have used or are using ATB in their projects.

Table 3.1: Districts that have used ATB in their construction based on Item 292.

Year	Tons of Asphalt Treated Base Material							
	Abilene	Beaumont	Houston	Paris	San Angelo	San Antonio	Waco	Wichita Falls
2005		2,882	16,602					
2006		139,815	149,159		248	217		688
2007		106,890	87,341	49,530	1,222		318	
2008	9,273	59,872	430,632		13,430	3,123		

Table 3.2: Districts that have used ATB in their construction based on Item 345.

Year	Tons of Asphalt Treated Base Material					
	Beaumont	El Paso	Houston	San Angelo	San Antonio	Waco
2005	251,204		74,751	13,359	1,394	16,616
2006	122,402		220,980			133
2007	162,887	441	146,029	3,103		
2008	43,679		353,711			

Based on information from lettings in Site Manager, the districts that have recently used Item 292 are as per Table 3.1. The districts that have used Item 345 (similar but discontinued version of Item 292) are summarized in Table 3.2.

Houston and Beaumont are the districts with the highest quantities of ATB. Personal contact with the Wichita Falls and San Antonio Districts indicated that they had changed ordered the ATBs reflected in the tables to Type A/B HMA mixes.

Question 2: If yes, how many such projects have been completed in the last 5 years or are scheduled to be constructed in the near future in your district?

The six districts that responded positively to the previous question have at least worked with one ATB project, with Houston being the leader with approximately 20 ATB projects.

Question 3: Which specification do you use for the design of ATB?

Of the districts that use ATB, 50% follow only Item 292 for the design of ATB. Abilene District uses only Tex-204-F for the design, and Pharr district applies both design procedures (Figure 3.1).

If you use Item 292 or Tex-204-Part III, do you waive any of the requirements?

Abilene waives Tex 242-F “Hamburg Wheel-Tracking Test”, Tex 226-F “Indirect Tensile Strength”, and if virgin base is used, it should meet triaxial requirements as per Tex 126-F “Molding, Testing, and Evaluating Bituminous Black Base Material”. Houston and Pharr waive Tex-126-E and the strength requirements, respectively.

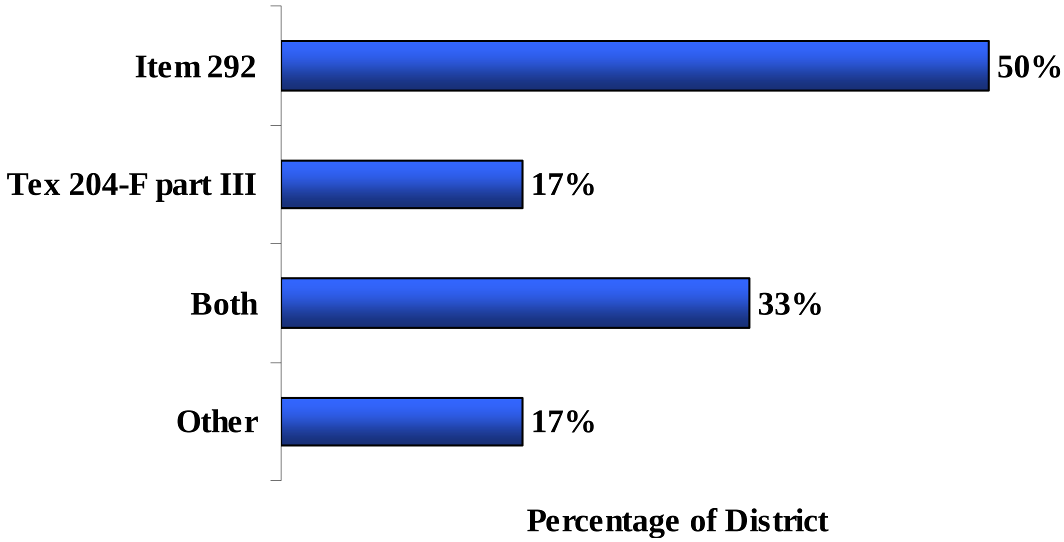


Figure 3.1: Specifications Used for ATB Design.

Question 4: Which compactor do you use for the design of ATB?

As shown in Figure 3.2, half of the districts that handle ATB use the Superpave Gyratory Compactor (SGC) for their design. Only San Angelo district uses the 6 in. gyratory press mentioned in Tex-126-E.

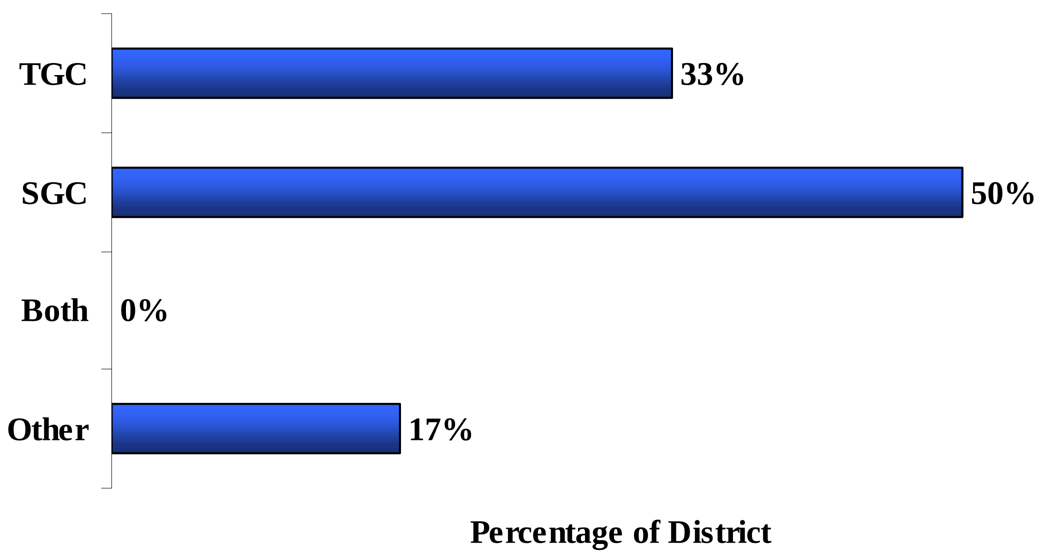


Figure 3.2: Compactors Used for ATB Design.

Question 5: What are the main uses of ATB in your district?

Most of the districts make use of ATB as an alternative to stabilized base and to Type A/B hot mix asphalt (HMA). Pharr District also applies ATB to reduce the pavement structure by eliminating the lime-treated subgrade at high volume intersection.

Question 6: What factors motivate you to select ATB for projects in your district over other alternatives?

The respondents indicated their main reasons of using ATB are the following items: (1) more economical, (2) easier to construct, (3) short curing time, (4) stronger than stabilized bases, and (5) rut resistance (see Figure 3.4).

Question 7: What typical aggregate types does your district use on ATB projects?

Based on the responses to the questionnaire, the majority of the districts use limestone as aggregate for ATB, just Paris district uses sandstone as aggregate.

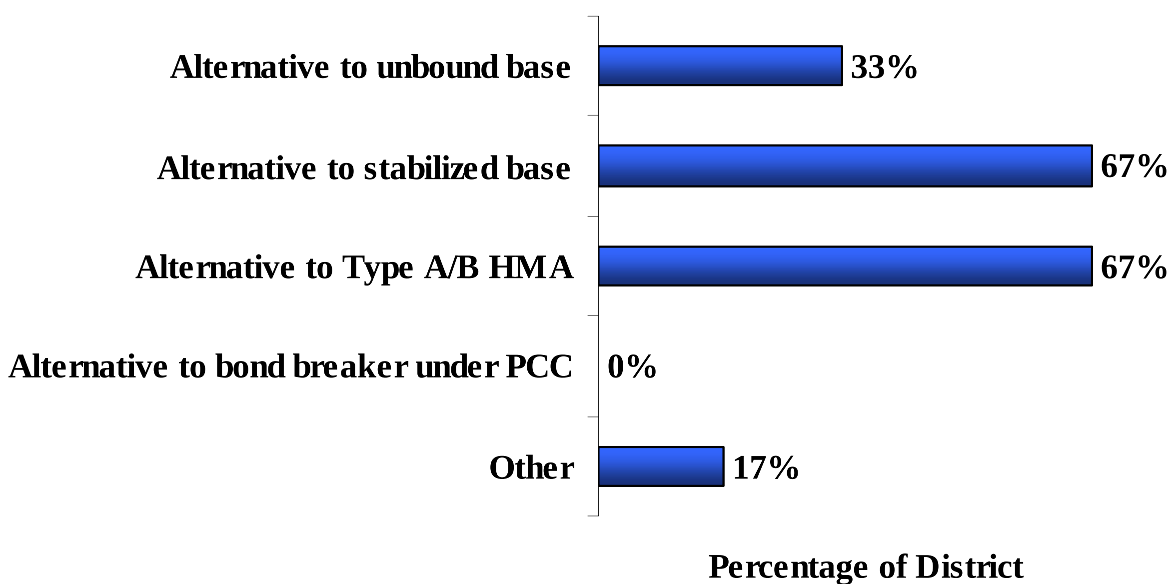


Figure 3.3: Main Uses of ATB.

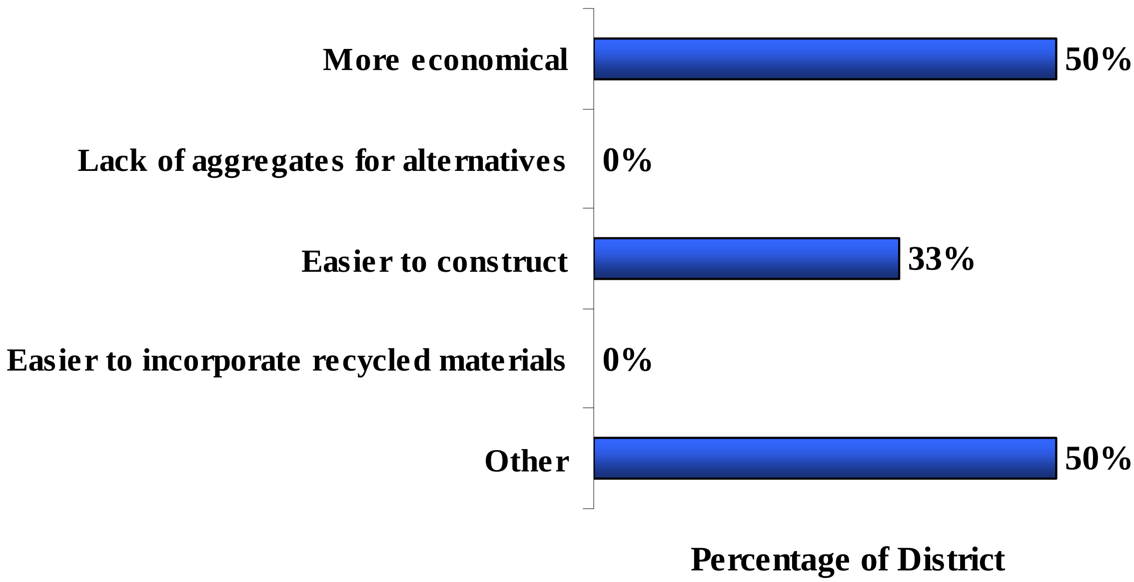


Figure 3.4: Factors for Selection of ATB in Projects.

Question 8: Do you add RAP or Crushed Concrete to your ATB?

50% of the districts using ATB add RAP to ATB, the other half do not add RAP or crushed concrete.

Question 9: As per Item 292, what are the major types and grades of the materials you use in your district?

Grade 1 as per Item 292 is the most used throughout the state, but Abilene and Houston Districts prefer to use Grade 2.

Question 10: What binder grades does your district use on ATB projects?

As shown in Figure 3.5, PG 64-22 is used for ATB by all districts in Texas. However, occasionally PG 70-22 and PG 76-22 are specified.

Question 11: What criteria are used to determine the amounts of binder?

Most of the districts that responded positively to the use of ATB in projects determine the amount of binder following TxDOT specifications, mainly density. San Angelo district bases the amount of binder on Area Engineer's preference and experience.

Question 12: What construction specifications do you use for your projects?

Half of the districts follow Item 292 for construction purposes. Pharr District states a lift thickness no greater than 4 in. Paris District requires 5% to 9% air voids calculated by the Theoretical Maximum Specific Gravity.

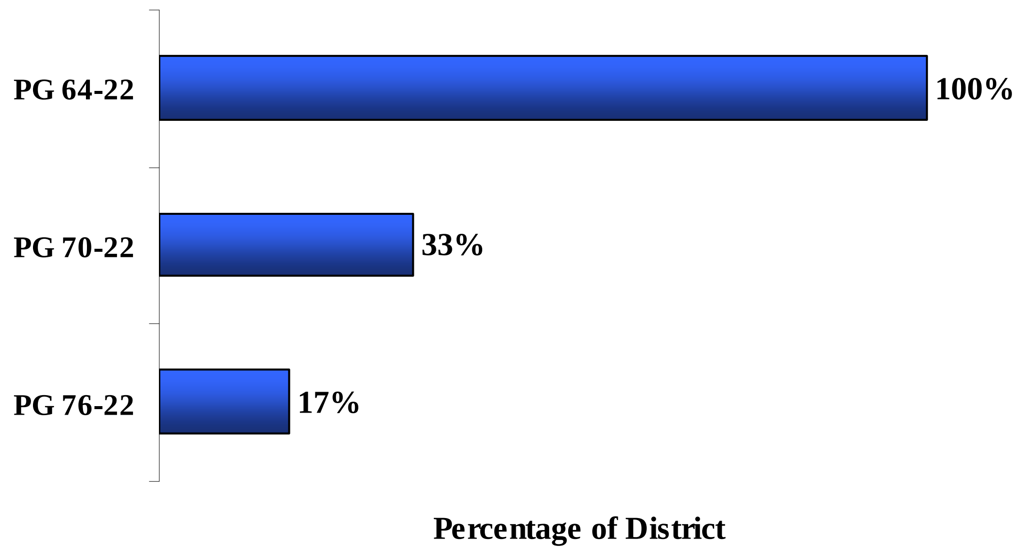


Figure 3.5: Binder Used for ATB Projects.

Question 13: What types of problems, if any, have you encountered with design or construction of ATB?

Based on the questionnaire, the districts were satisfied with the performance of their projects. However, segregation is mentioned as a more frequent problem presented in ATB than in HMA.

The Houston and Paris Districts staff was visited to obtain insight in their use of ATB, their current mix design processes and their concerns. The insight gained by the research staff from these two districts was quite evaluable.

Based on the questionnaire and interaction with the PMC, the candidate districts that were considered for this study are shown in Table 3.3.

Table 3.3: Candidate Districts Considered for This Study.

District	**Aggregate Type**
El Paso	Dolomite
Beaumont	Limestone
Paris	Sandstone
Wichita Falls	Limestone
Houston LP-610	Limestone
Houston SH-99	Limestone

Chapter Four: Comprehensive Evaluation of Alternative Protocols

The current methods commonly used by the districts were described in Chapter 2. This section contain a deep explanation of those methods along with two alternative proposed methods that are more in line with the operational requirements of TxDOT. This chapter addresses the Index properties; OAC based on many parameters; density, strength and modulus comparisons for prepared specimens following the current and proposed protocols.

Table 4.1 shows a summary of the different mix design methods used in this study. The main differences between mix design methods are; how the OAC is calculated, the size of the specimens, curing temperature and the strength test. To set the criteria for each method and to select a preferred method, a comparative study of the properties of different mixes was carried out.

Table 4.1: Summary of Mix Design Methods.

Baseline Mix Design	**Tex-126-E**	**Tex-204-F**	**Tex-126-H**	**Tex-204-H**
Compactor	TGC	SGC	SGC	SGC
Specimen Size, in.	6x8	6x4.5	6x8	6x4.5
OAC based on	Asphalt Density Curve	Volumetric Properties	Asphalt Density Curve	Asphalt Density Curve
Curing and Testing Temperature, °F	140	77	140	77
Strength Test	UCS	IDT	UCS	IDT

4.1 TEX-126-E

In this method, asphalt stabilized (black base) materials are molded using a Texas Gyratory Compactor (TGC). Several specimens with different asphalt contents are prepared and tested to develop an asphalt content-density curve and an asphalt content-unconfined

compressive strength (UCS) curve. The optimum asphalt content (OAC) is determined from the asphalt content-density curve. The compacted black base specimens are made in duplicates and are tested for their unconfined compressive strengths at 140°F in this test method. The specimens are subjected to two types of deformation rates: slow (0.15 in./min) and fast (10 in./min.) Specimens tested at a slow deformation rate yield relatively lower UCS's as compared to the fast deformation rate ones. From the UCS-density relationship, the minimum density that would satisfy the UCS requirements as per Item 292 is taken as the minimum allowable density.

4.2 TEX-204-F

This procedure was originally developed to design Type A and Type B hot mix at 96% density using 6 in. x 4½ in. specimens. Unless otherwise indicated, the specimens are molded to 100 gyrations using an SGC. The quality of the mix assessed with the indirect tensile strength at a deformation rate of 2 in./min and a nominal temperature of 77oF.

4.3 PROPOSED TEX-126-H

The proposed Tex-126-H (hybrid) is similar to Tex-126-E, with one exception. While Tex-126-E uses a Texas Gyratory Compactor (TGC), Tex-126-H advocates the use of a Superpave Gyratory Compactor (SGC) to produce the specimens. The rest of the protocol is the same as Tex-126-E, where the asphalt-density curve and UCS-density curve are developed to assess the OAC and field density.

4.4 PROPOSED TEX-204-H

The second proposed method, Tex-204-H (Hybrid), is similar to Tex-204-F. The only difference is the way the OAC is selected. For Tex-204-H, the requirements for volumetric properties such as VFA are waived because it is not possible to modify the gradation to achieve the required volumetric properties. In Tex-204-H protocol, attention is paid to asphalt-density curve, similar to Tex-126-H, and the IDT strength.

4.5 Comparison of Properties from Different Approaches

Mix designs were performed on six materials following the four approaches indicated above to highlight their similarities and differences. These six materials were collected from different sites from El Paso, Beaumont, Paris, Wichita Falls and two sites from Houston, LP-610 and SH-99. To demonstrate different approaches a local material from El Paso is used as an example. Results from the other materials are then summarized.

4.5.1 Index Properties

The gradation curves of all materials as received are shown in Figure 4.1. To prepare the materials for mix design, the entire stock of the material was sieved first to develop a global gradation curve. This gradation curve was used throughout the study. The acceptable range as per Item 292 Grade 1 is also shown in the figure. Houston SH-99 and Paris materials do not fulfill those requirements.

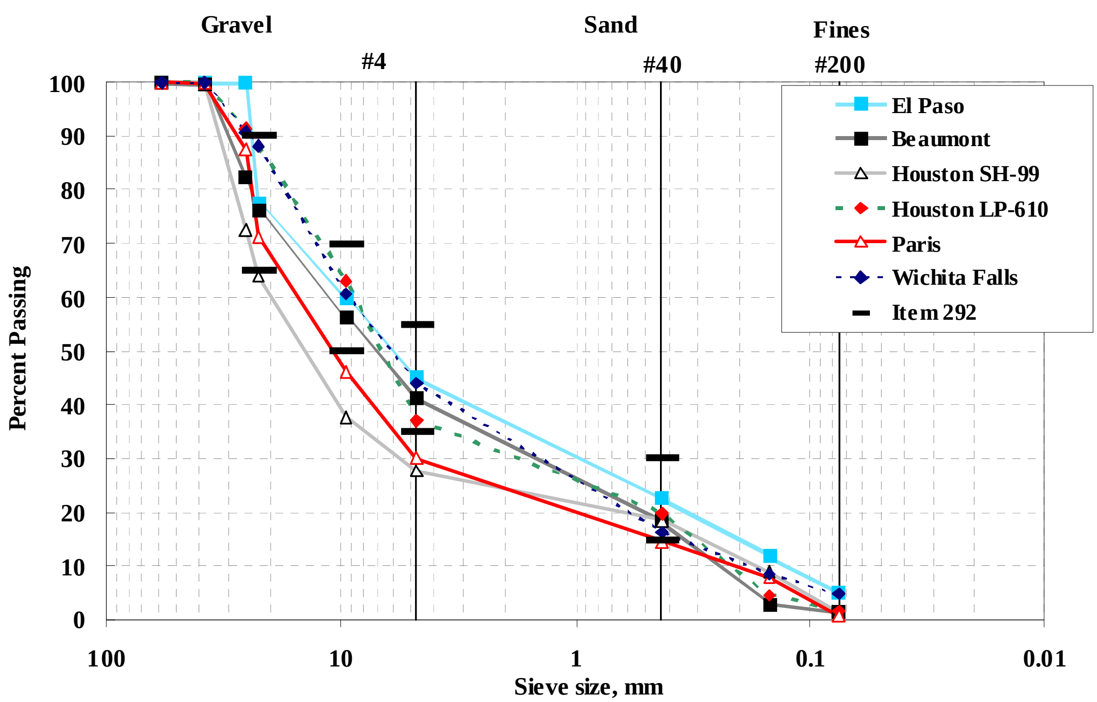


Figure 4.1: Gradation Curves from Different Materials Used in This Study.

Figure 4.2 illustrates the material constituents for all materials. Houston SH-99 and Paris have higher concentrations of gravel (72% and 70%, respectively) while El Paso material has the lowest gravel concentration (55%). Coarse sand contents range from 9% to 28%, with Houston SH-99 containing the lowest concentration and Wichita Falls the highest. The fine sand contents of all materials are similar, ranging from 12% to 18%. El Paso and Wichita Falls have the highest fines concentrations (5%) while other materials contain 1% to 2% fines.

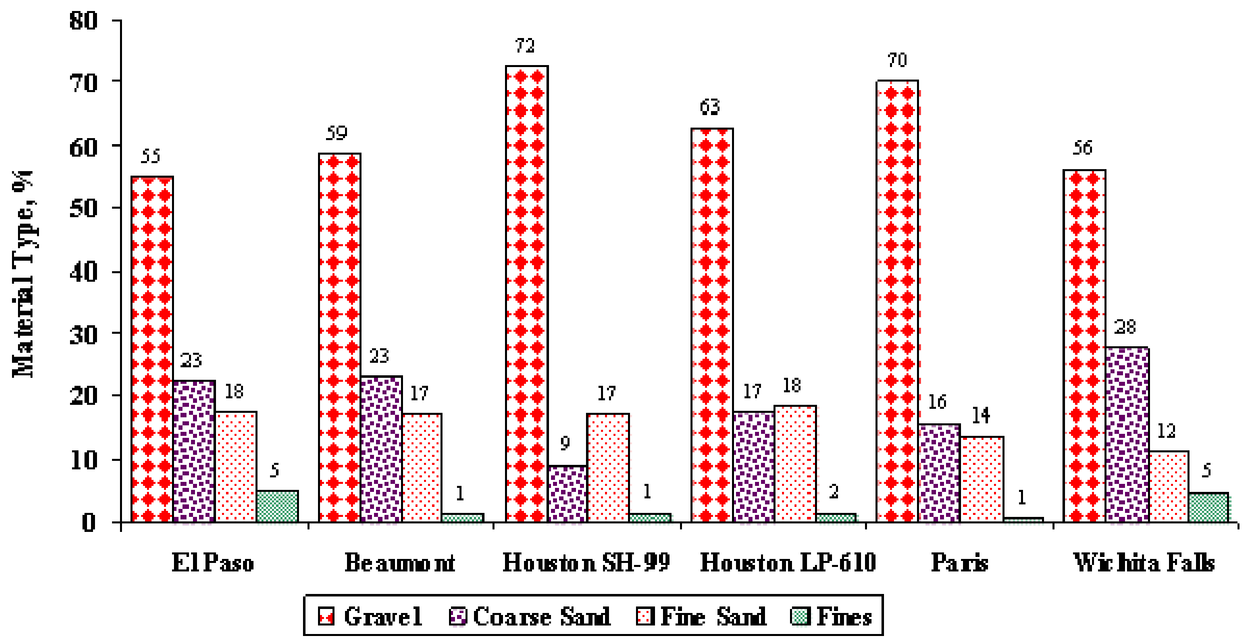


Figure 4.2: Material Constituents for the Sites Being Studied.

Material classifications as per Unified Soil Classification System (USCS) and American Association of State Highway and Transportation Officials (AASHTO) Soil Classification System, as well as the Atterberg limits are summarized in Table 4.2. Under the USCS, all materials classified as GP (poorly-graded gravel) except for Houston LP-610 and Wichita Falls which were categorized as GW (well-graded gravel). All materials are classified as A-1-a under the AASHTO system. All materials seem to be non-plastic except Wichita Falls.

Table 4.2: Soil Classification and Plasticity Index for Bases under Study.

Material Source	Classification		Atterberg Limits	
	USCS	AASHTO	Liquid Limit	Plasticity Index
Beaumont	GP	A-1-a	Non Plastic	
El Paso	GP	A-1-a	Non Plastic	
Houston SH-99	GP	A-1-a	Non Plastic	
Houston LP-610	GW	A-1-a	Non Plastic	
Paris	GP	A-1-a	Non Plastic	
Wichita Falls	GW	A-1-a	21	4

The sand equivalency, wet ball mill and aggregates hardness for each material are presented in Table 4.3. The materials used in this study met the sand equivalency requirement except for Wichita Falls materials. All material met the wet ball mill requirements; Beaumont material was near to reach the maximum acceptable value. All materials exhibit Aggregate Crushing Value (ACV) and Aggregate Impact Value (AIV) 30% or less.

Table 4.3: Sand Equivalency and Wet Ball Mill and Hardness for Bases under Study.

Material Source	Sand Equivalency, %	Wet Ball Mill, %	Aggregate Crushing Value	Aggregate Impact Value
Beaumont	79	49	15	8
El Paso	53	28	16	9
Houston SH-99	82	11	25	16
Houston LP-610	80	16	29	18
Paris	52	12	24	14
Wichita Falls	26	31	30	20
Item 292 Limits	Min 40	Max 50	-	-

4.5.2 Results from Tex-126-E Protocol

Three sets of 6 in. by 8 in. specimens, prepared with the TGC at nominal asphalt contents of 3%, 4.5% and 6%, were tested to determine the optimum asphalt content (OAC). Based on the consultation with the PMC, the maximum AC content was limited to 6% for economic reasons. A minimum AC content of 3% was selected for constructability.

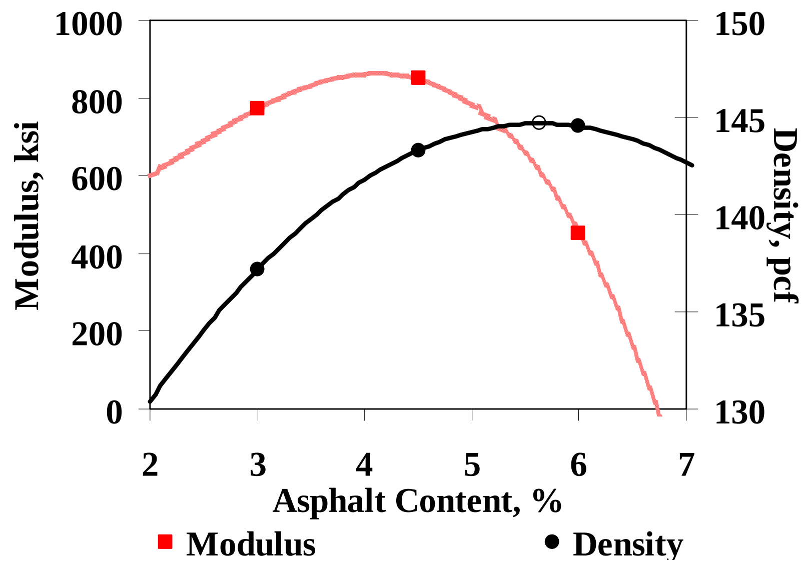
The compacted specimens were weighed and their heights and diameters were measured to calculate their densities. After the specimens equilibrated to 140°F oven for 48 hrs, they were tested as quickly as possible to avoid any heat loss for their moduli using the Free-Free Resonant Column (FFRC) tests, followed by UCS. Figure 4.3 shows the results from the El Paso material.

The OAC as per Tex-126-E is about 5.6%. The strength requirements for ATB as per Item 292 (50 psi) are met by all three AC contents used in this study.

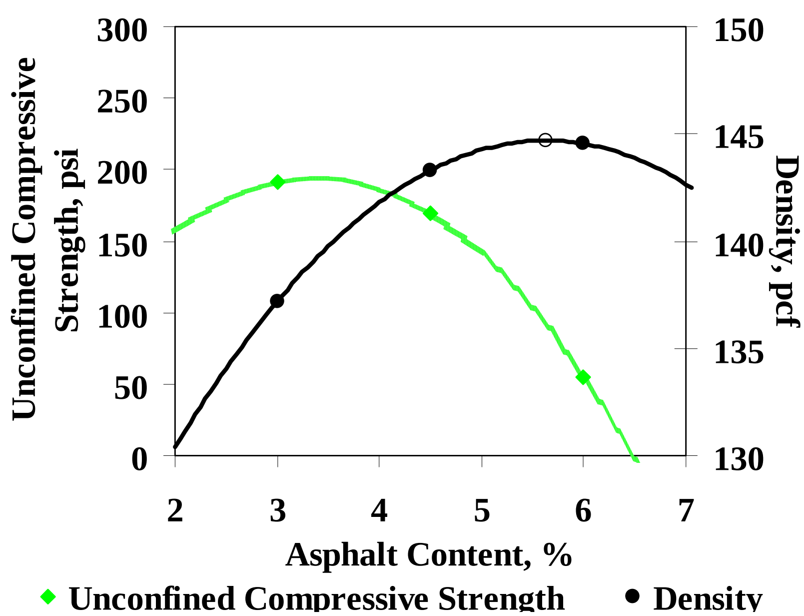
Two alternative OACs can also be extracted from Figure 4.3, one being at the AC content when the modulus is maximum and the other when the UCS is maximum. The OACs based on modulus and UCS are 4.1% and 3.0%, respectively.

Table 4.4 shows the density, UCS and modulus for each sets of specimens tested at 3%, 4.5% and 6% asphalt contents for all materials. The variations in density with AC content are 6% or less for all materials. Almost all materials exhibited their lowest strengths and moduli at an AC content of 6%.

The OAC values based on density are compared to those from modulus and strength in Table 4.5 for all materials. The OACs based on density are typically higher as compared to those based on modulus and strength. The OACs based on densities fall between 4.3% and 6%.



a) Asphalt Content–Density/Modulus Curves



b) Asphalt Content-Density/Strength Curves

Figure 4.3: Density/Modulus /Strength vs. Asphalt Content as per Tex-126-E.

4.5.3 Results from Tex-126-H Protocol

The same process followed to obtain the OACs using Tex-126-H specifications. A Pine SGC at 100 gyrations was used to prepare the specimens. Figure 4.4 shows the asphalt content-density, asphalt content-modulus and asphalt content-strength curves for El Paso material as per

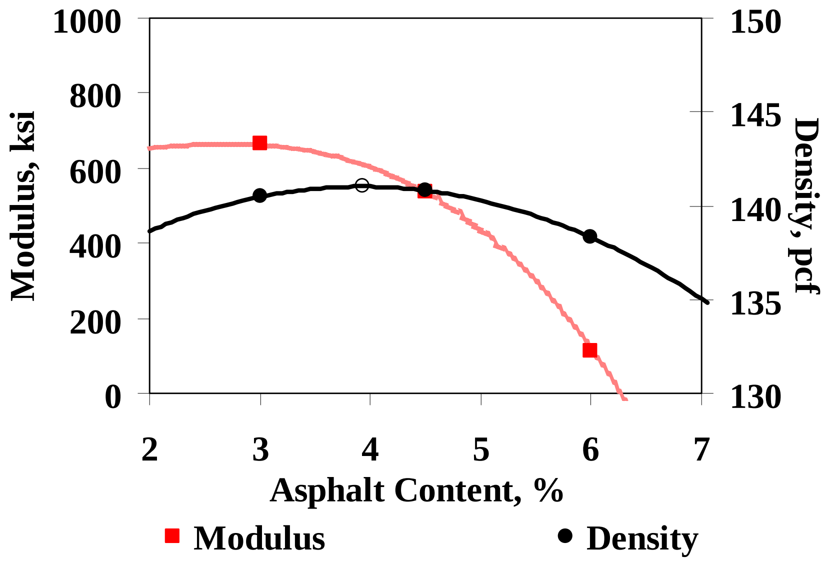
Table 4.4: Density, Strength and Modulus at Different Asphalt Contents as per Tex-126-E.

Parameter	Asphalt Content	Material Site					
		Beaumont	El Paso	Houston SH-99	Houston LP-610	Paris	Wichita Falls
Density, pcf	3%	137	138	138	140	129	128
	4.5%	140	143	144	141	129	132
	6%	141	145	141	140	130	136
UCS, psi	3%	161	192	206	76	214	227
	4.5%	126	169	95	160	141	354
	6%	119	55	69	85	128	236
Modulus, ksi	3%	609	526	865	560	650	653
	4.5%	558	871	690	1145	687	1443
	6%	271	436	481	623	426	962

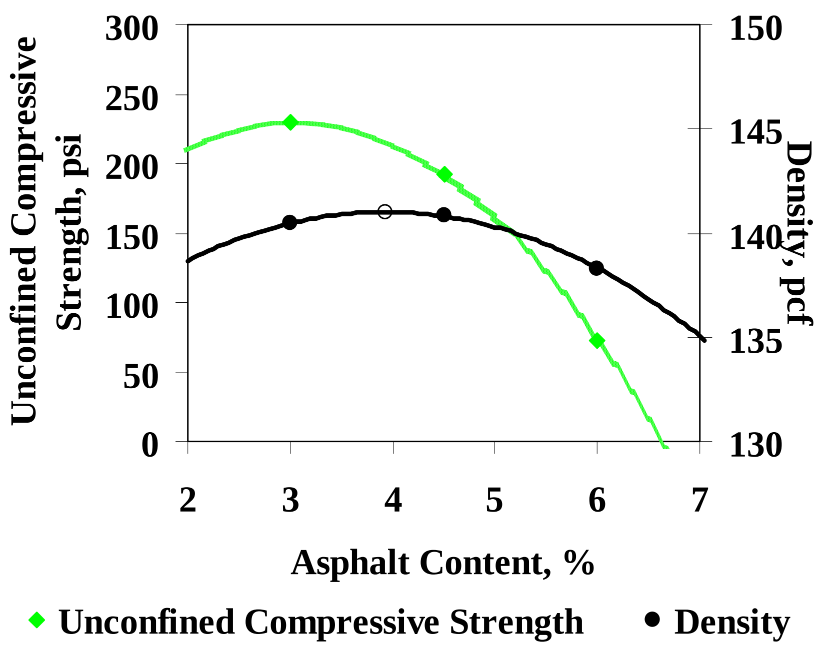
Table 4.5: Optimum Asphalt Content based on Density, Strength or Modulus as per Tex-126-E.

Optimum Asphalt Content Based on Maximum	Beaumont	El Paso	Houston SH-99	Houston LP-610	Paris	Wichita Falls
Density	6.0	5.6	4.7	4.5	4.3	6.0
UCS	6.0	3.0	3.0	4.5	3.0	4.5
Modulus	3.4	4.1	3.0	4.7	4.0	4.8

Tex-126-H. The specimens with 3% binder exhibited the highest modulus and strength. OAC from maximum density was obtained at 3.9% asphalt content. The density curve in Figure 4.4 is so flat that any asphalt content between 3% and 5% can easily be practically considered as the OAC from that curve given the uncertainties in obtaining the density of each specimen.



a) Asphalt Content–Density/Modulus Curves



b) Asphalt Content-Density/Strength Curves

Figure 4.4: Density/Modulus /Strength vs. Asphalt Content as per Tex-126-H.

The density, UCS and modulus of every sets of specimens tested are summarized in Table 4.6. The variations in the densities with AC contents were less than 5% for all materials. The densities are generally the greatest at binder contents of 4.5%. Ignoring Wichita Falls

Table 4.6: Density Strength and Modulus at Different Asphalt Contents as per Tex-126-H.

Parameter	Asphalt Content	Beaumont	El Paso	Houston SH-99	Houston LP-610	Paris	Wichita Falls
Density, pcf	**3%**	134	141	135	136	127	126
	4.5%	136	141	136	137	126	130
	6%	135	138	134	134	130	133
UCS, psi	**3%**	159	230	84	167	N/A*	207
	4.5%	180	192	55	121	163	410
	6%	105	73	48	70	125	301
Modulus, ksi	**3%**	948	664	1022	1422	N/A*	498
	4.5%	960	537	915	909	527	1423
	6%	462	115	369	534	600	1524

*** unable to retrieve an intact specimen after compaction and as such could not be tested.**

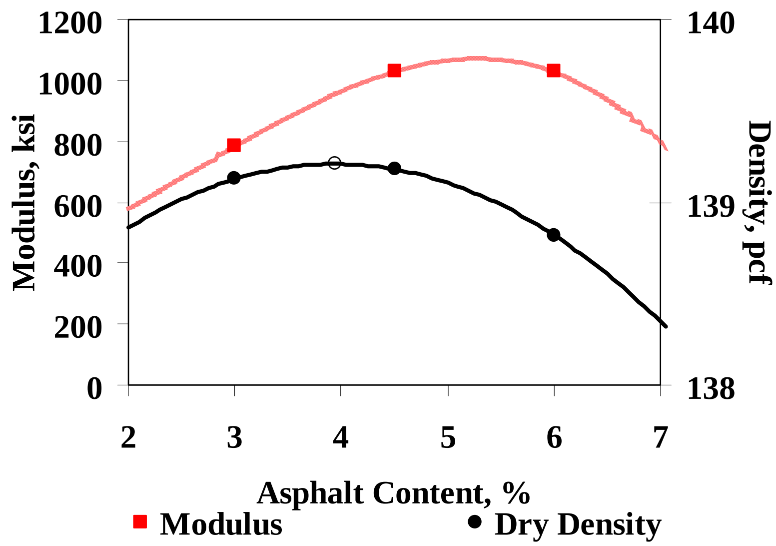
materials, the maximum strengths and moduli were obtained at 3% AC contents. Once again, all specimens marginally or significantly passed the strength requirement of 50 psi for ATB as per Item 292. As reflected in Table 4.7, once again the highest OACs are generally obtained based on density.

Table 4.7: Optimum Asphalt Contents Based on Density, Strength or Modulus as per Tex-126-H.

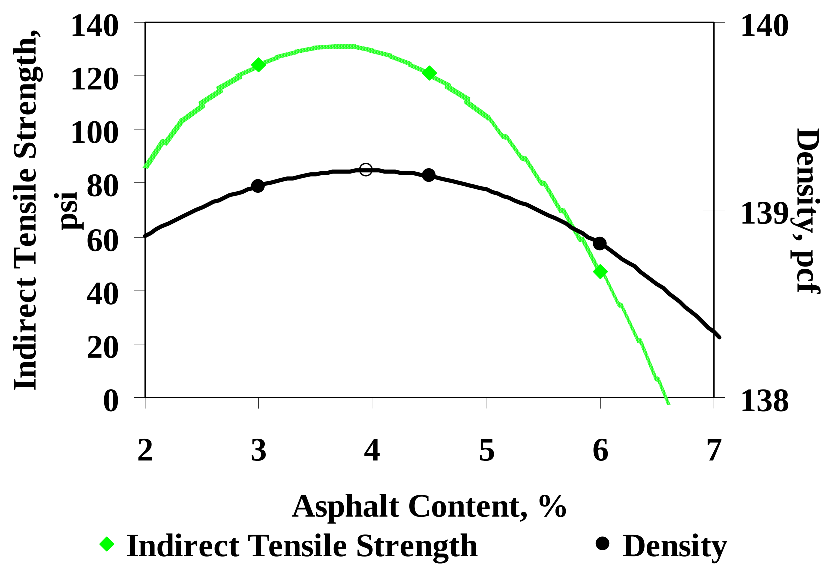
Optimum Asphalt Content Based on Maximum	Beaumont	El Paso	Houston SH-99	Houston LP-610	Paris	Wichita Falls
Density	4.7	3.9	4.2	4.1	6.0	6.0
UCS	4.1	3.0	3.0	4.2	5.0	4.7
Modulus	3.6	3.0	3.4	3.0	5.5	5.4

4.5.4 Results from Tex-204-H Protocol

The asphalt-density curve for Tex-204-H for El Paso material is illustrated in Figure 4.5. The OAC based on density is 3.9% at a density of about 139 pcf. It is hard to judge the accuracy of this OAC, since the density changes by less than 1% with the change in the asphalt content. The OACs based on modulus and IDT occur at about 5.3% and 3.7%, respectively. Since the specimens' heights were only 4.5 in., a v-meter was used to measure the moduli.



a) Asphalt Content-Density/Modulus Curves



b) Asphalt Content-Density/Strength Curves

Figure 4.5: Density/Modulus /Strength vs. Asphalt Content as per Tex-204-H.

The values obtained for density, indirect tensile strength (IDT) and modulus for all materials prepared under Tex-204-H are summarized in Table 4.8. For all materials the variations in the density with asphalt content are rather small. Most materials exhibit their highest IDT strengths between asphalt contents of 3 and 4.5%. Wichita Falls specimens with 3% asphalt content could not be tested because they broke after compaction but before testing. Moduli of the specimens tested do not vary much for most materials similar to the densities.

The OAC values based on maximum density, indirect tensile strength and modulus as per Tex-204-H for all materials are summarized in Table 4.9. As opposed to the Tex-126-E and Tex-126-H results, no clear pattern is apparent for this protocol.

Table 4.8: Density, Strength and Modulus at Different Asphalt Contents as per Tex-204-H.

Parameter	**Asphalt Content**	**Beaumont**	**El Paso**	**Houston SH-99**	**Houston LP-610**	**Paris**	**Wichita Falls**
Density, pcf	**3%**	133	139	138	137	126	129
	4.5%	136	139	138	138	130	132
	6%	135	139	135	134	130	134
IDT Strength, psi	**3%**	133	124	118	168	91	N/A
	4.5%	163	121	105	150	116	110
	6%	100	47	72	104	101	170
Modulus, ksi	**3%**	855	783	1007	538	617	N/A
	4.5%	862	1030	1103	553	622	790
	6%	880	1030	1055	547	688	857

Table 4.9: Optimum Asphalt Contents Based on Density, Strength or Modulus as per Tex-204-H.

Optimum Asphalt Content Based on Maximum	Beaumont	El Paso	Houston SH-99	Houston LP-610	Paris	Wichita Falls
Density	6.0	3.9	3.8	3.8	6.0	6.0
IDT Strength	4.2	3.7	4.7	3.0	6.0	6.0
Modulus	6.0	5.3	4.8	4.8	6.0	5.6

4.1.5 Tex-204-F

Figure 4.6 shows the gradation curves for the six materials and the acceptable limits for Item 344 type SP-A. El Paso, Houston LP-610 and Wichita Falls materials fit well between the gradation requirements for Item 344 Grade SP-A. Beaumont, Houston SH-99 and Paris materials do not fulfill those requirements, since they are coarser than Item 344 Grade SP-A requirements.

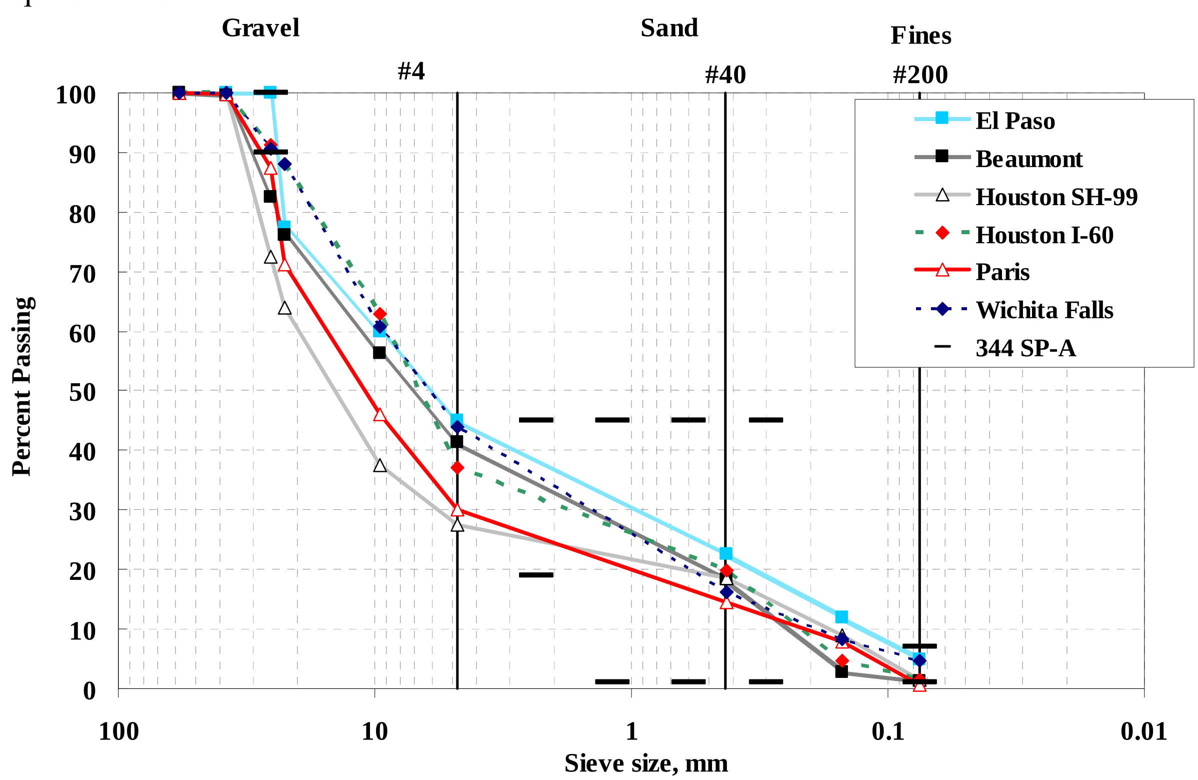


Figure 4.6: Gradation Curves from Different Materials Compared to Item 344 Grade SP-A.

To check whether any of the mixes would meet the volumetric properties as per Tex-204-F Part III, the theoretical maximum specific gravity (Gmm) in accordance to Tex-227-F were determined. Triplicate samples were tested to assess the repeatability of this test method. The Gmm values are reported in Table 4.10. All samples tested exhibited coefficients of variation (COV's) of 1.2% or less.

The bulk specific gravities (Gmb) for all specimens are presented in Table 4.11. The maximum Gmb values are obtained at binder contents between 4.5 and 6%.

Table 4.10: Maximum Theoretical Specific Gravities for Tex-204-H Specimens.

Asphalt Content	**Parameter**	**Beaumont**	**El Paso**	**Houston SH-99**	**Houston LP-610**	**Paris**	**Wichita Falls**
3.0%	**Average**	2.45	2.62	2.51	2.47	2.37	2.55
	COV*	0.3%	0.5%	0.3%	0.4%	0.1%	0.4%
4.5%	**Average**	2.45	2.55	2.43	2.41	2.46	2.48
	COV	0.2%	0.7%	0.1%	0.1%	0.3%	0.1%
6.0%	**Average**	2.39	2.46	2.38	2.36	2.4	2.41
	COV	0.3%	1.2%	0.4%	0.2%	0.7%	1.1%

*** COV = coefficient of variation**

Table 4.11: Bulk Specific Gravities for Tex-204-H Specimens.

Asphalt Content	**Beaumont**	**El Paso**	**Houston SH-99**	**Houston LP-610**	**Paris**	**Wichita Falls**
3.0%	2.33	2.41	2.39	2.30	2.25	2.28
4.5%	2.37	2.47	2.40	2.35	2.30	2.37
6.0%	2.41	2.40	2.41	2.30	2.34	2.41

Figure 4.7 shows an example of the volumetric properties for the El Paso material. For El Paso material, the OAC is 5.2% (see Figure 4.7a). Voids in mineral aggregates (VMA) for 5.2% asphalt content is about 15%, which is greater than a minimum value of 13 specified in Item 344 for SP-A mixes. The voids filled with asphalt (VFA) of about 74% at a binder content of 5.2% fell between the 65-75% as required by Item 344.

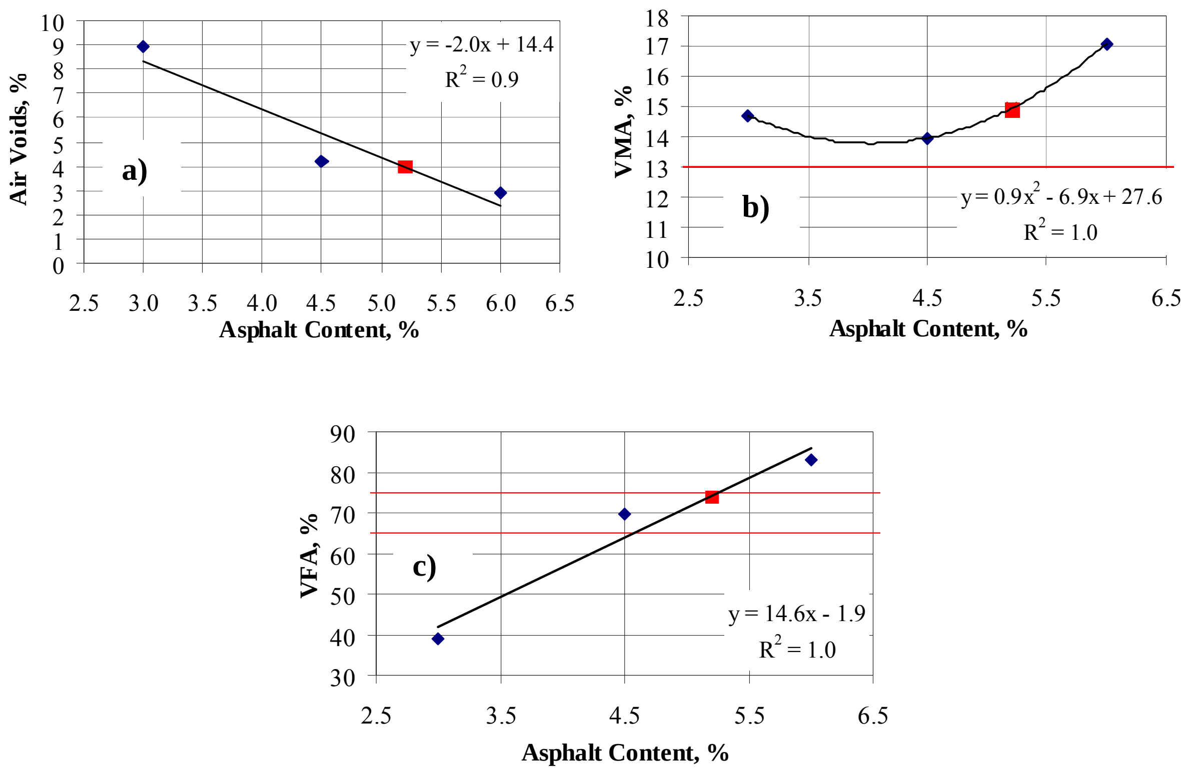


Figure 4.7: Volumetric Properties for El Paso Material.

Table 4.12 shows the volumetric properties for all specimens as well as the requirements to be met for Item 344 SP-A mixes. The majority of the materials successfully passed the requirements, except for the VFA value for Houston SH-99, Paris and Wichita Falls.

Table 4.12: Volumetric Properties for Tex-204-H Specimens.

Values at 4% Air Voids	Beaumont	El Paso	Houston SH-99	Houston LP-610	Paris	Wichita Falls	Item 344 Grade SP-A
AC, %	4.0	5.2	3.7	4.3	5.8	5.0	-
VMA, %	14.3	14.9	20.2	14.8	18.4	15.6	Min. 13
VFA, %	71.5	74.1	80.4	72.4	78	75.5	65-75

4.5.6 Moisture Susceptibility

One of the test methods used to quantify moisture susceptibility is Tex-242-F, Hamburg Wheel-Tracking Test (HWTD). This test was performed on specimens prepared at OAC's determined from Tex-126-H and Tex-204-H protocols based on density. The results for the HWTD are summarized in Table 4.13.

Table 4.13: Hamburg Wheel Test Results for All Materials.

Test Method	Number of Passes	Rut Depth, in					
		Beaumont	El Paso	Houston SH-99	Houston LP-610	Paris	Wichita Falls
Tex-126-H	OAC, %	4.7	3.9	4.2	4.1	6.0	6.0
	10,000	**0.089**	**0.17**	**0.207**	**0.219**	**0.088**	**0.108**
	15,000	**0.096**	**0.211**	**0.312**	**0.45**	**0.092**	**0.115**
	20,000	**0.124**	**0.235**	**0.463**	**0.478**	**0.111**	**0.135**
Tex-204-H	OAC, %	6.0	3.9	3.8	3.8	6.0	6.0
	10,000	**0.009**	**0.151**	**0.102**	**0.263**	**0.088**	**0.108**
	15,000	**0.016**	**0.174**	**0.108**	**0.482**	**0.092**	**0.115**
	20,000	**0.023**	**0.185**	**0.14**	*****	**0.111**	**0.135**

*** Exceeded 0.5 in. rut after 15,300 passes.**

were continued up to 20,000 passes. All specimens tested exhibited acceptable rut depths at 10,000 cycles. All specimens were in the acceptable limits at 15,000 and 20,000 cycles except for Houston LP-610 which reached the 0.5 in. deformation at 15,300 cycles.

The second test used to quantify moisture susceptibility is Tex-144-E, the Tube Suction Test (TST). This test was also performed using the OAC based on density obtained in this study for Tex-126-H and Tex-204-H. Table 4.14 shows the strengths and moduli before and after the TST tests. The difference between before and after the TST is the asphalt curing time. Most of the materials demonstrated an increase in strength and moduli after the TST. Houston LP-610 was the only material that did not gain strength and modulus with curing.

4.5.7 Analysis of Results

The OAC values based on the three different criteria (density, strength and modulus) used for all materials are summarized in Figure 4.8. The variations in OACs based on density for Tex-126-H, Tex-204-H and Tex-204-F are compared with the OACs based on density for Tex-126-E in Figure 4.9. The alternative mix design methods typically yield lower OACs as compared to Tex-126-E. Paris material was the only one that had higher OAC's with Tex-126-H and Tex-204-H than OAC for Tex-126-E. This inconsistency is caused by the flatness of the density curves as discussed above. In general, the density is not very sensitive to asphalt content as discussed above. As such, the OAC obtained based on density should be reviewed to decide on the OAC of a mix.

Similarly, the OACs based on strength are compared in Figure 4.10. Most of the OACs are greater than those of the Tex-126-E OAC values based on strength. Two of the materials, Beaumont and Houston LP-610 were outliers; they had a lower OAC than Tex-126-E in all three different mix design methods.

The OAC values based on modulus are compared in Figure 4.11. Similarly most of the OACs are greater than that of Tex-126-E values based on modulus with three outliers Houston LP-610 (Tex-126-H and Tex-204-F) and El Paso (Tex-126-H).

Table 4.14: Impact of Tube Suction Tests on UCS and Seismic Modulus.

Test Method	Parameter	Beaumont		El Paso		Houston SH-99		Houston LP-610		Paris		Wichita Falls	
		Before	After	Before	After	Before	After	Before	After	Before	After	Before	After
Tex-126-H	OAC, %	4.7		3.9		4.2		4.1		6.0		6.0	
	UCS, psi	405	509	485	653	373	498	583	539	337	446	392	426
	Modulus, ksi	2411	2242	1739	3028	1279	2738	2494	2462	1720	2482	1940	2054
Tex-204-H	OAC, %	6.0		3.9		3.8		3.8		6.0		6.0	
	IDTS, psi	92	107	123	190	63	148	203	153	98	134	143	212
	Modulus, ksi	833	938	825	901	835	1123	910	834	872	919	882	834

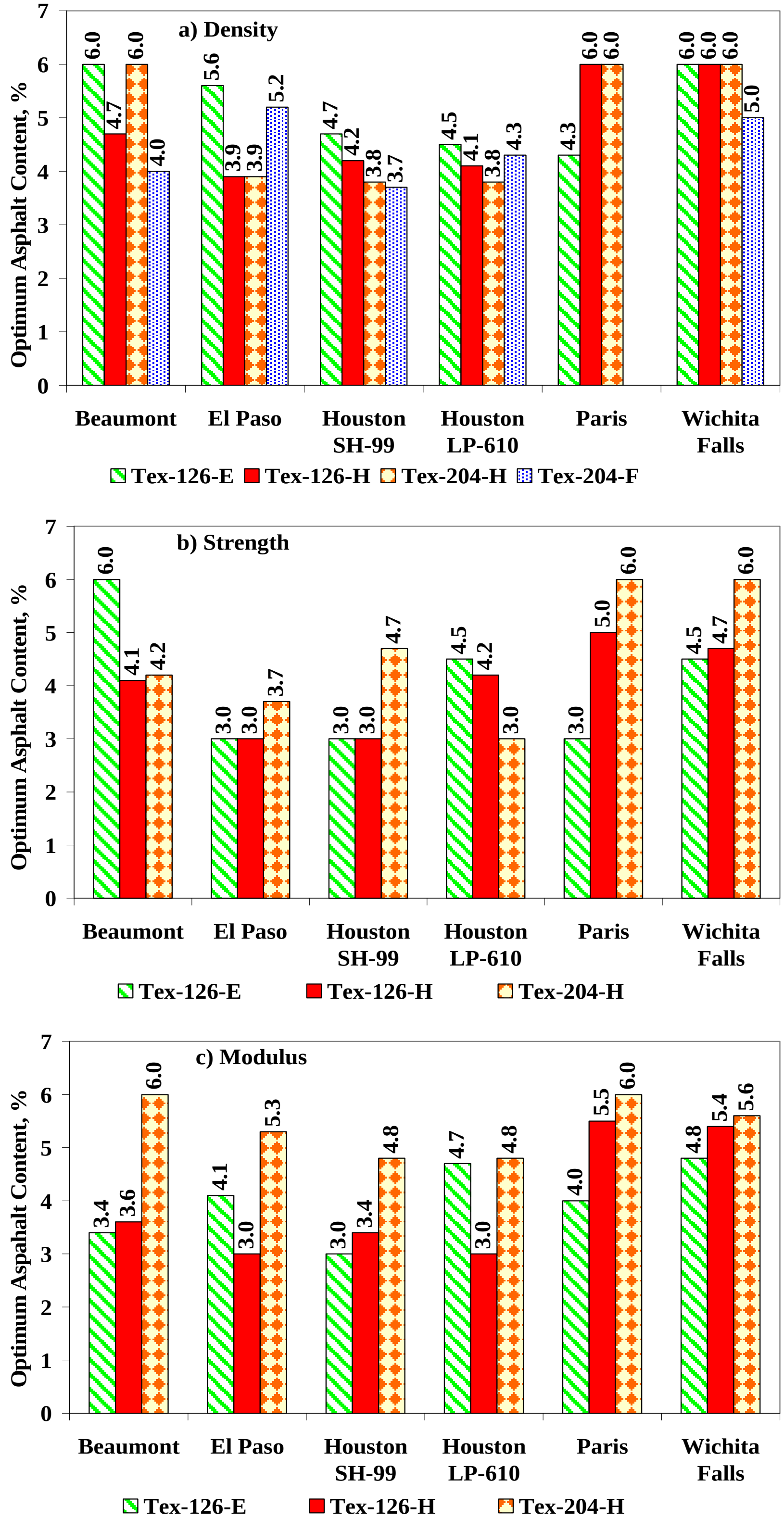


Figure 4.8: Comparison of Optimum Asphalt Contents Based on Density, Strength, and Modulus.

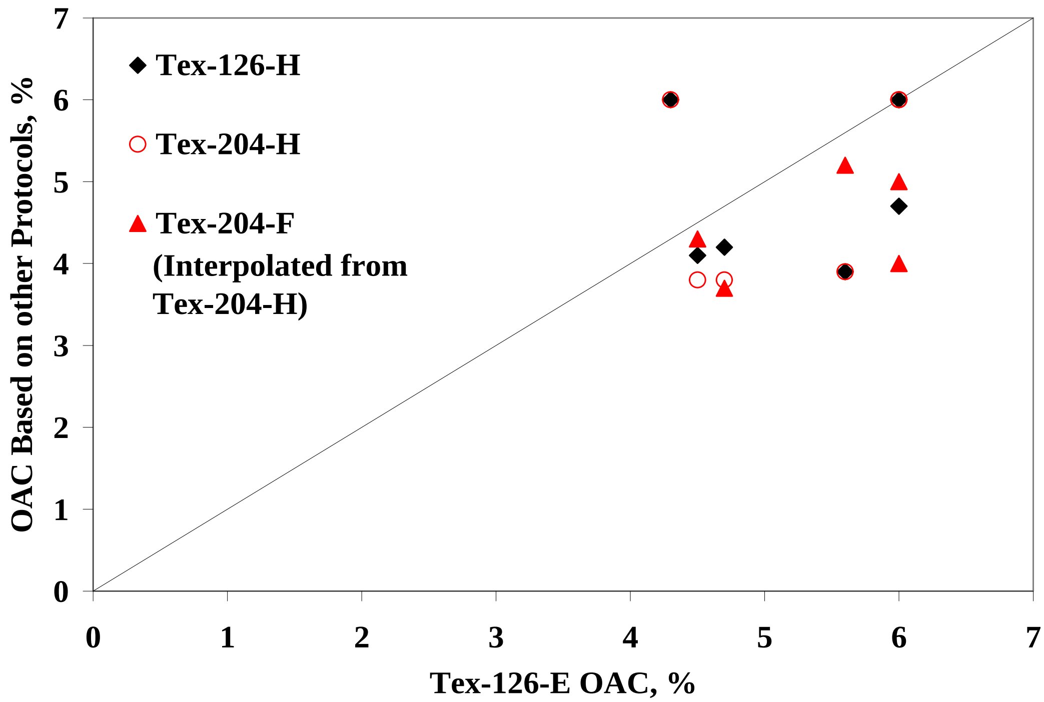


Figure 4.9: Comparison of Tex-126-E OAC with other Methods Based on Density.

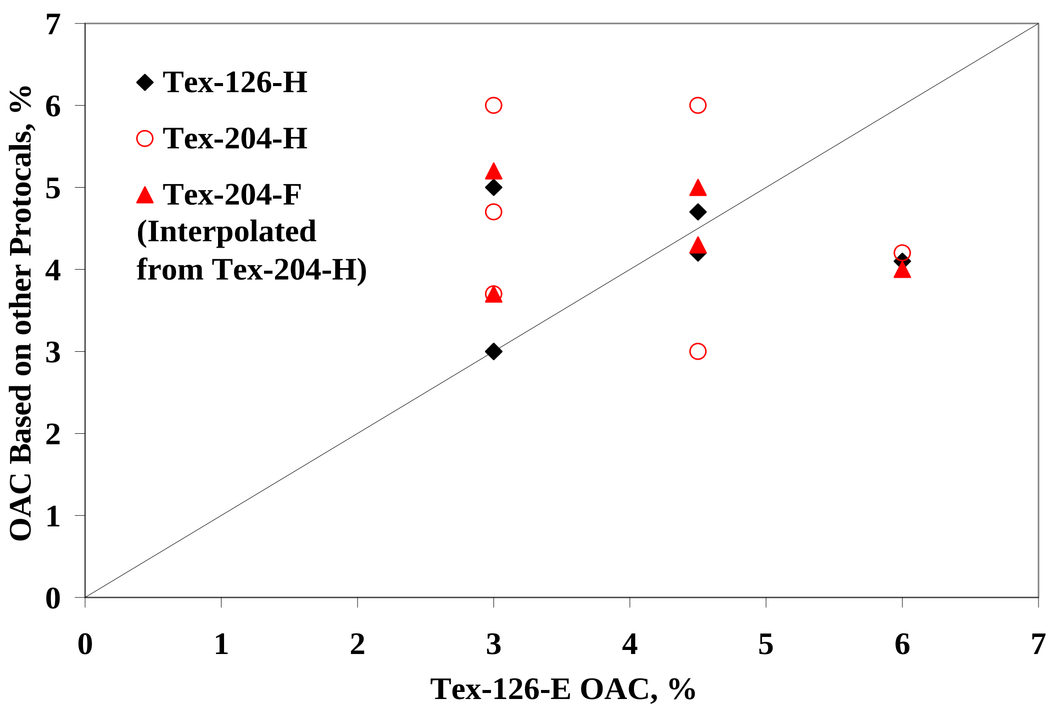


Figure 4.10: Comparison of Tex-126-E OAC with other Methods Based on Strength.

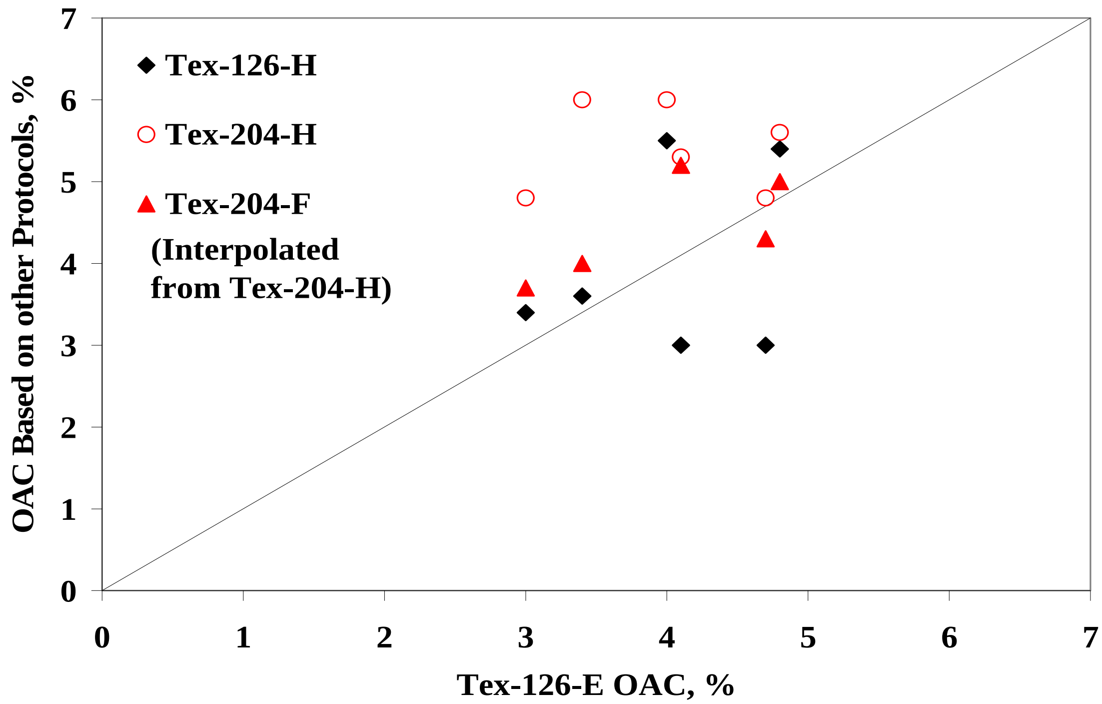


Figure 4.11: Comparison of Tex-126-E OAC with other Methods Based on Modulus.

Figure 4.12 shows the maximum densities for all materials and for all mix design methods. For a given mix, the densities are fairly close. The maximum density for most materials is generally reached when Tex-126-E is followed. The lowest densities are typically obtained from Tex-204-F by waiving some of the volumetric requirements.

For comparison purposes, UCS and IDT specimens were prepared based on OAC of each method and were tested for strength and modulus. The variations in the UCS values are shown in Figure 4.13. All materials reached the 50 psi strength requirement for ATB as per Item 292. The strongest material is Wichita Falls while the weakest material analyzed was Houston SH-99. The specimens prepared with the Superpave Gyratory Compactor usually exhibit similar or higher UCS as compared to those prepared with the Texas Gyratory Compactor.

The IDT strengths obtained from 6 x 4.5 in. specimens are shown in Figure 4.14. No obvious pattern could be found in the data. Given the uncertainty in the IDT tests, in most cases the strengths are similar from different methods of estimating OAC.

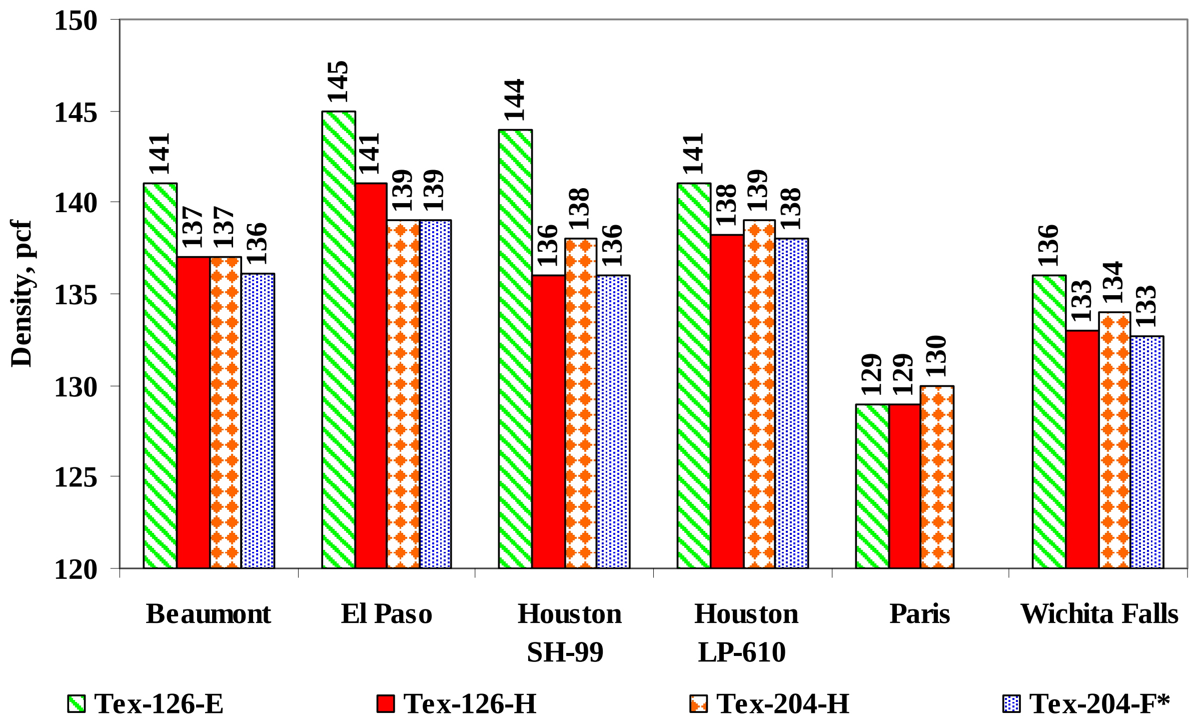


Figure 4.12: Densities at OACs for Each Test Method.

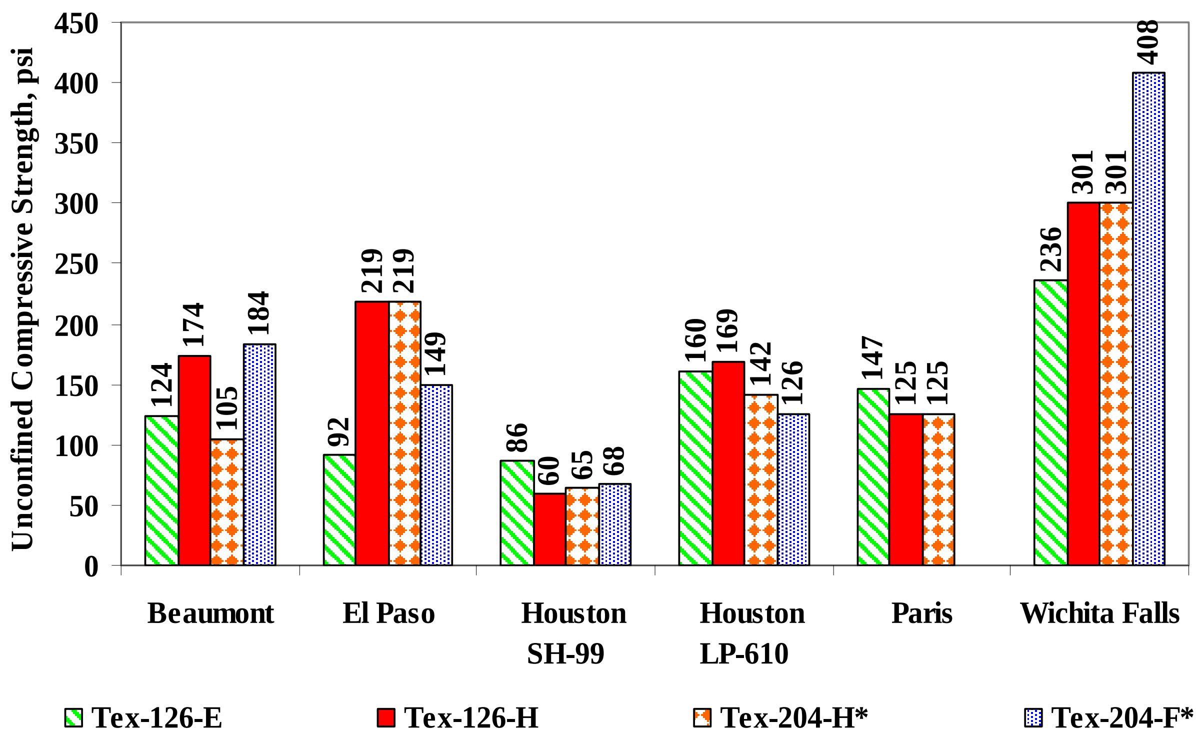


Figure 4.13: UCS at OACs for Each Test Method.

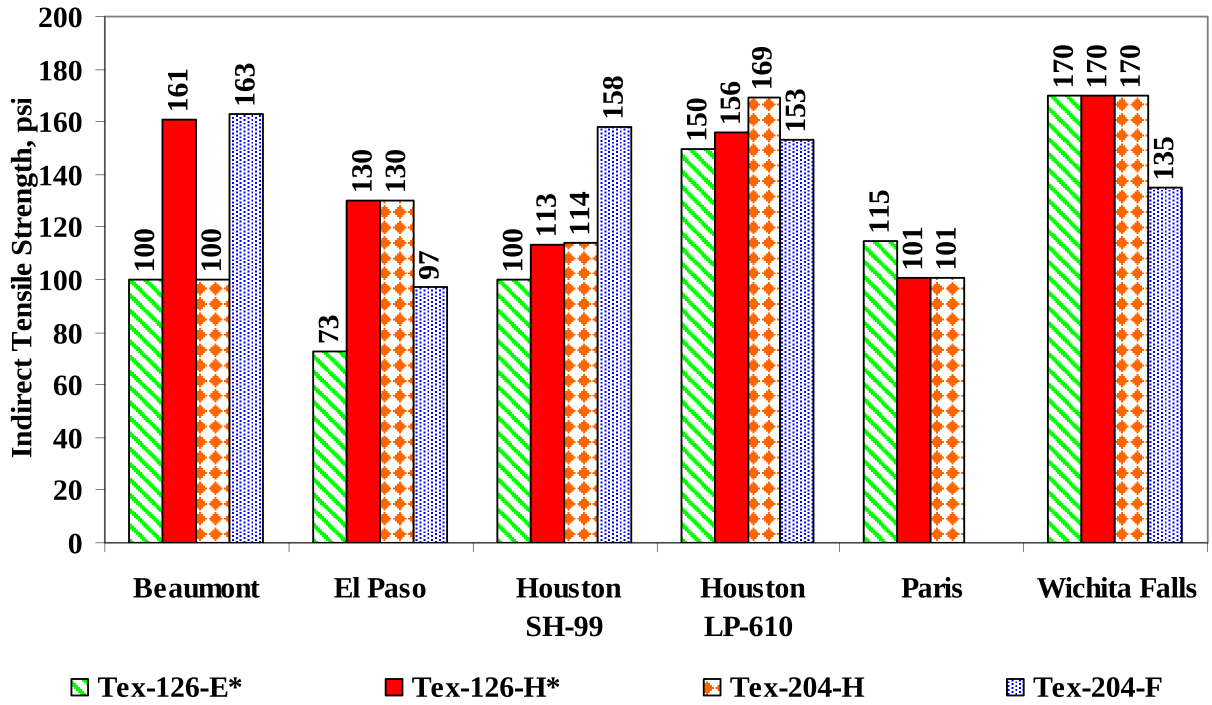


Figure 4.14: IDT at OACs for Each Test Method.

Finally, the moduli obtained from these specimens are shown in Figures 4.15a and 4.15b. The variations in moduli are in line with the corresponding strengths shown in Figures 4.13 and 4.14. So far this study shows that mix design using the SGC is feasible. Based on the results reported in this chapter, the adoption of either Tex 126-H or Tex 204-H will be discussed.

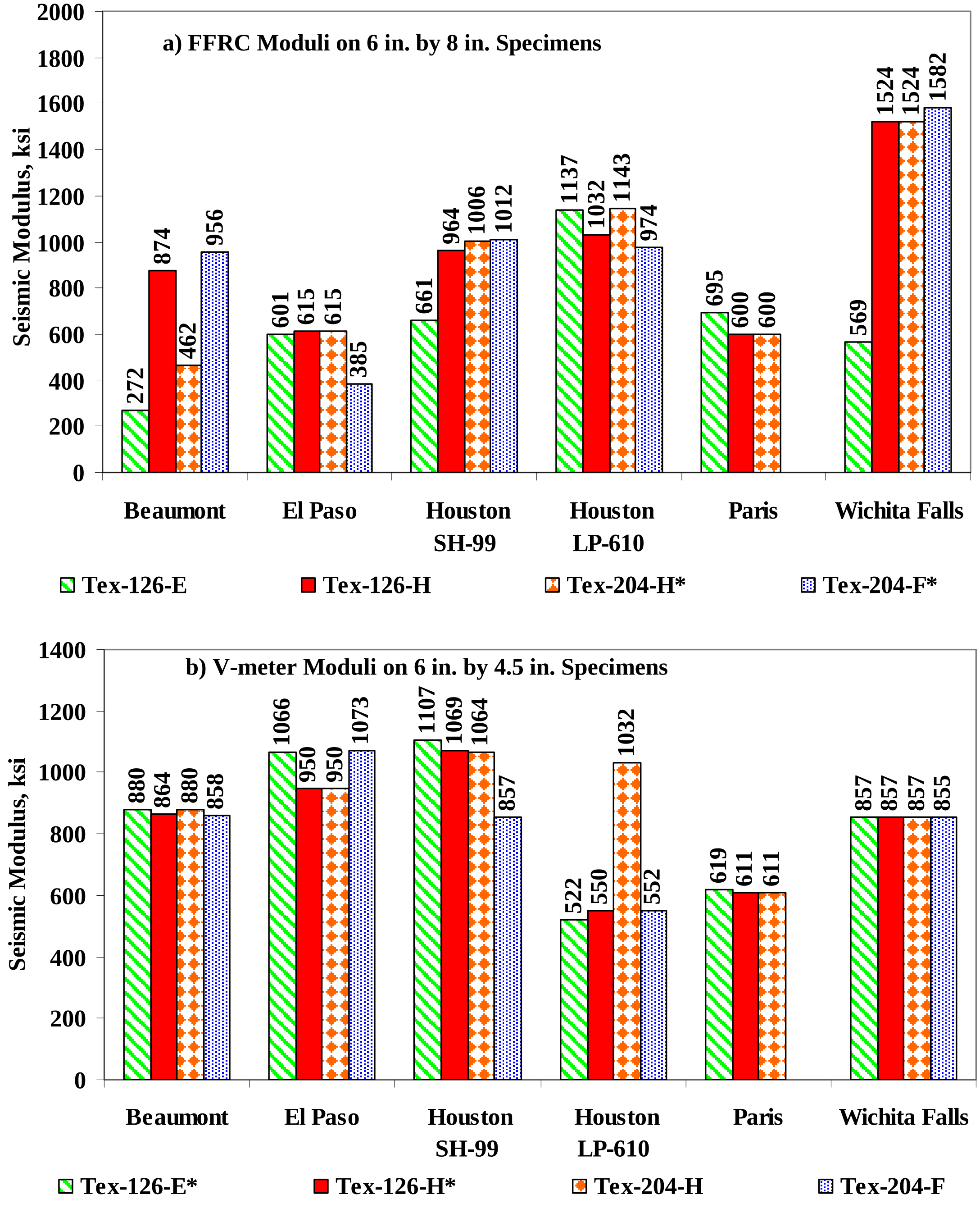


Figure 4.15: Seismic Modulus at OACs for Each Test Method.

Chapter Five: Evaluation of Parameters that Impact Performance

The two alternative protocols deemed reasonable for fulfilling the objectives of this project are Tex-126-H and Tex-204-H. Many specimens were mixed, compacted and tested at different numbers of gyrations using the SGC to evaluate the impact of the number of gyrations on the properties of the mixes. Specimens were also mixed and compacted using the TGC to try to match the densities of the SGC specimens with the ones compacted using the TGC. The impact of curing temperature is also presented in this chapter in which specimens were tested after 24 or 48 hrs of curing at 77°F or 140°F. Two different grades of asphalt (PG 76-22 and PG 64-22) were evaluated as well. The differences in optimum asphalt content, density, strength and modulus between the two binders are presented. The impact of the change in gradation on optimum asphalt content, density, strength and modulus is summarized at the end of this chapter.

5.1 IMPACT OF NUMBER OF GYRATIONS

5.1.1 Tex-126-H Protocol

The impact of the number of gyrations on the optimum asphalt content (OAC) based on density as per Tex-126-H is summarized in Figure 5.1. For Paris and Wichita Falls, the OAC's seem to be 6% (upper limit imposed on the asphalt content) or greater. Except for the Houston SH-99, the increase in the number of gyrations results in typically less than 1% change in the OAC. The OAC's as per Tex-126-E are also included in Figure 5.1. The OAC's for 60 to 80 gyrations are typically closer to the OAC's from Tex-126-E.

The densities at corresponding OAC's for different numbers of gyrations of SGC are compared with those from Tex-126-E in Figure 5.2. The impact of the number of gyrations of SGC on density is rather small (typically less than 3 pcf). In almost all cases, the highest densities are associated with Tex-126-E protocol.

The impact of the number of gyrations on UCS and modulus are shown in Figures 5.3 and 5.4, respectively. All specimens were prepared to the nominal dimensions of 6 in. by 8 in.

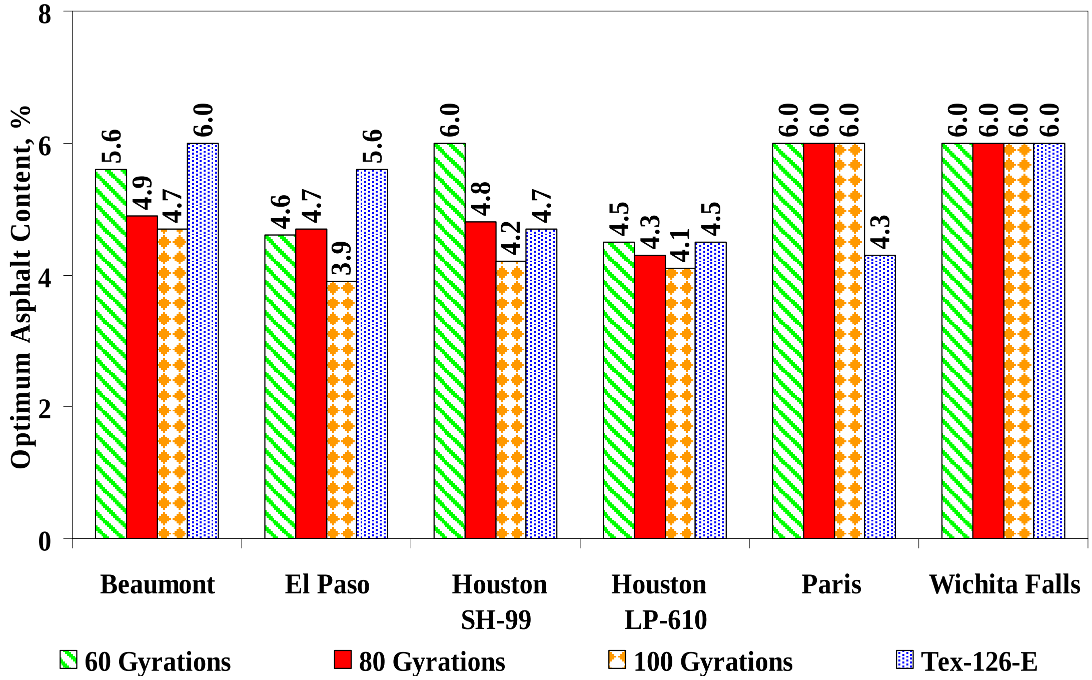


Figure 5.1: Impact of Number of Gyrations on OAC Based on Density for Tex-126 –H Protocol.

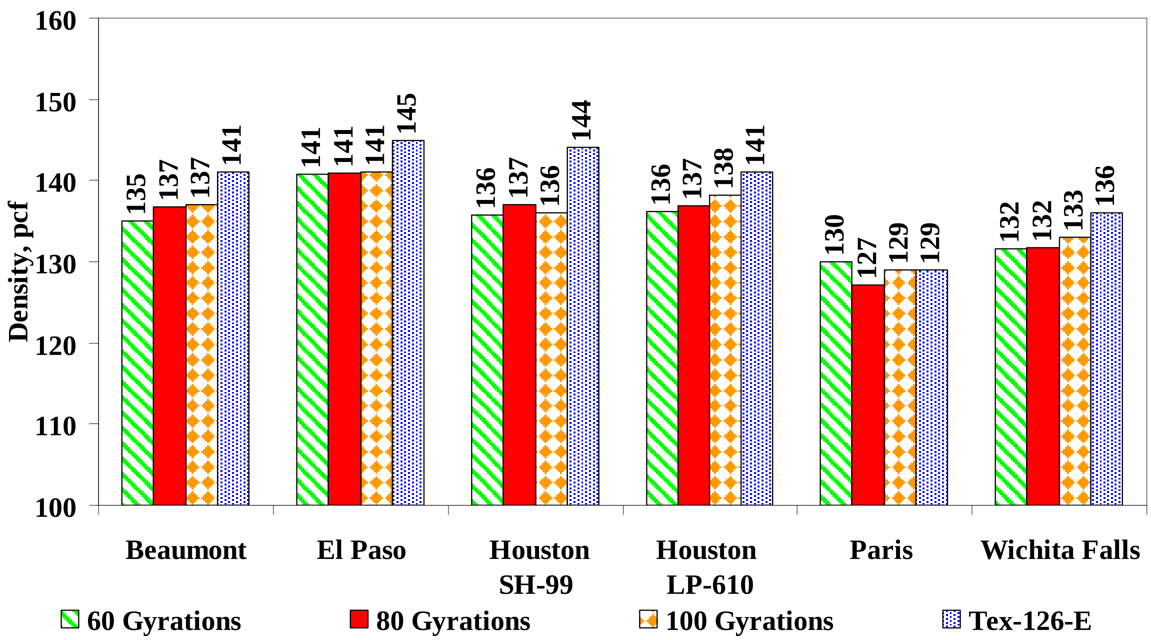


Figure 5.2: Impact of Number of Gyrations on Density for Tex-126-H Protocol.

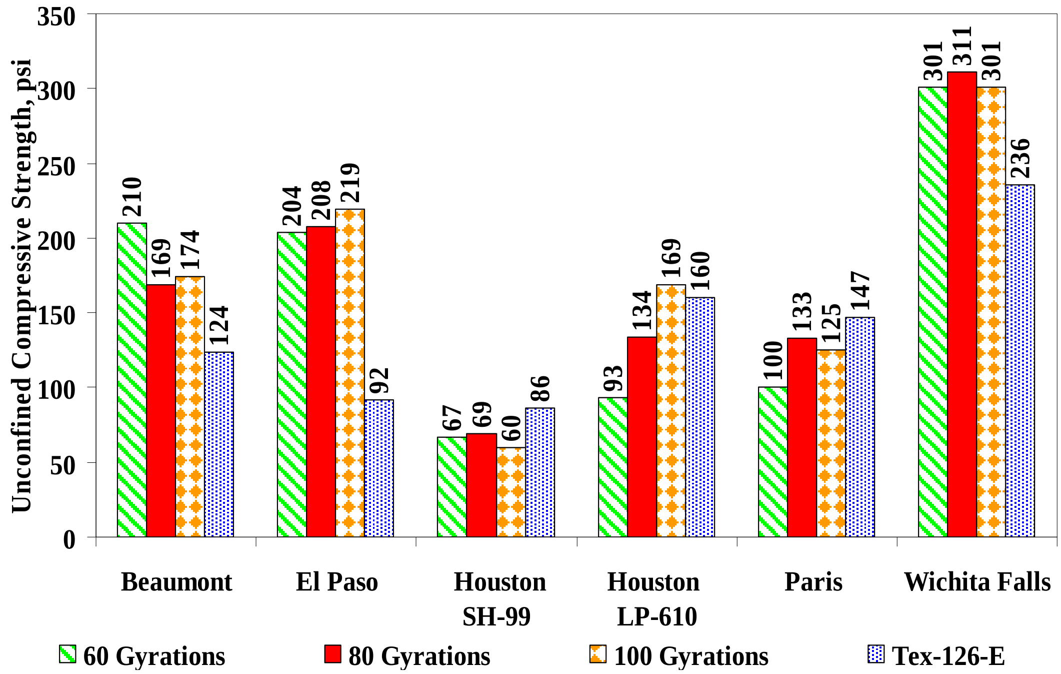


Figure 5.3: Impact of Number of Gyrations on UCS for Tex-126-H Protocol.

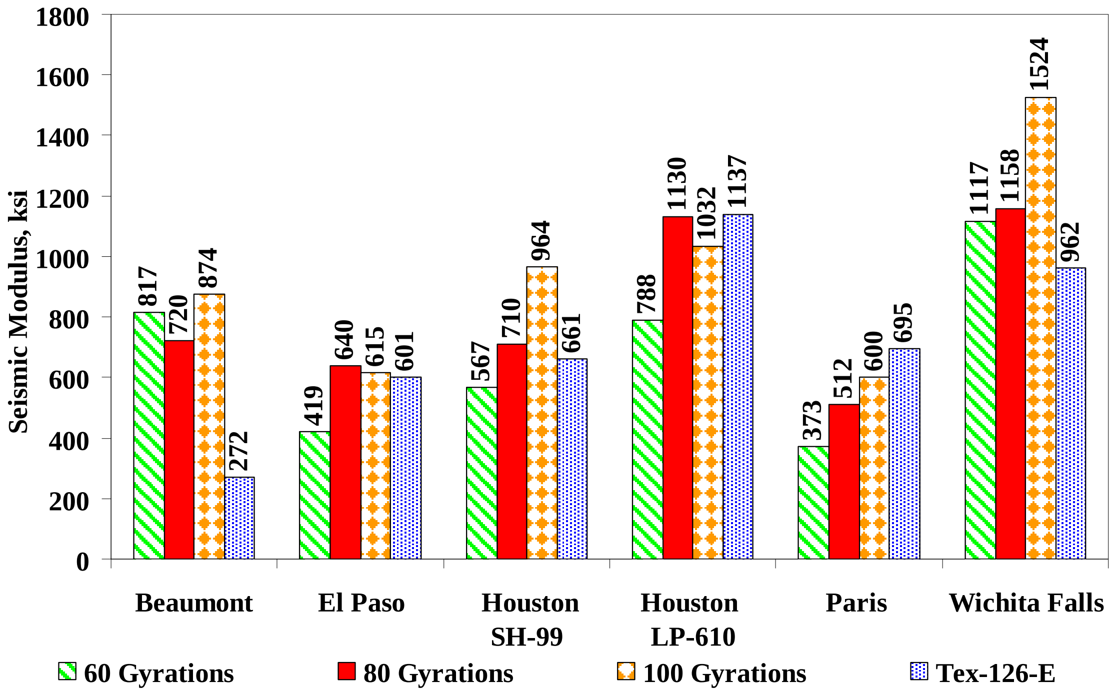


Figure 5.4: Impact of Number of Gyrations on Modulus for Tex-126-H Protocol.

The UCS and modulus values mostly follow the same pattern. As the UCS increases the modulus also increases. A clear pattern is not evident in these results primarily because of the complex interaction of the compaction energy with the asphalt content. The other complicating factor is the so-called locking point during the compaction with the SGC. Locking point is defined as the first gyration in the first occurrence of three gyrations of the same height (Varvik and Carpenter, 1998). At the locking point aggregates lock together and additional gyrations may degrade the aggregates. The locking points for all mixes are presented in Figure 5.5. Except for Paris materials, the locking points are less than 100 gyrations. This indicates that the use of 100 gyrations may not be desirable for the ATB mixes.

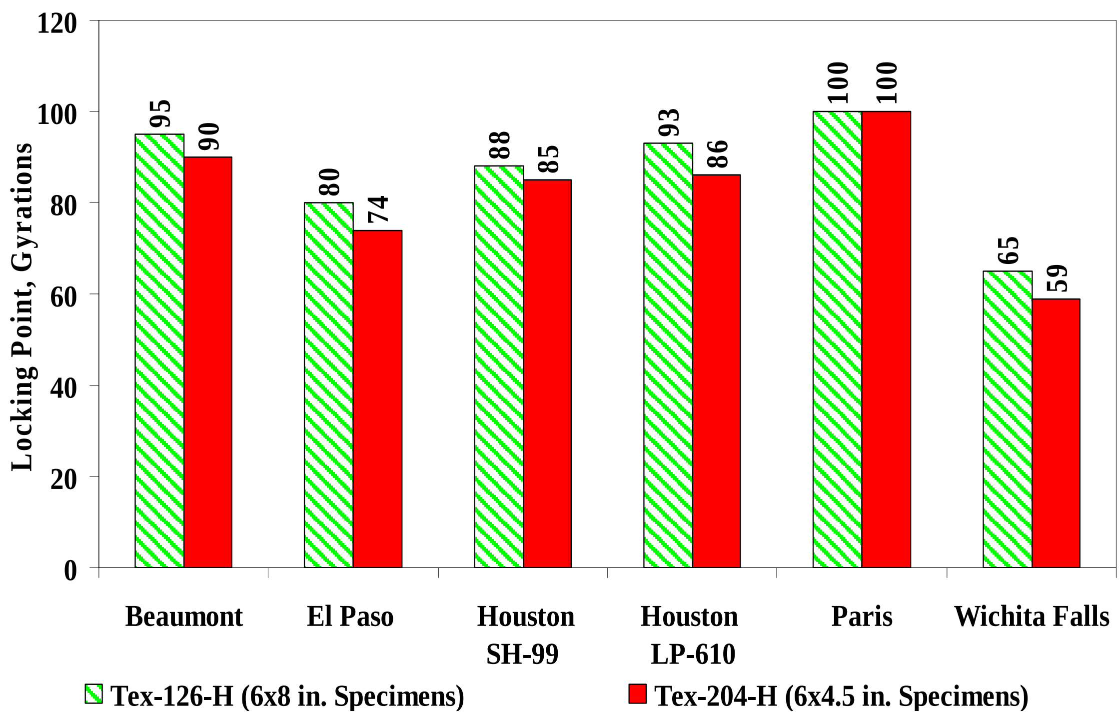


Figure 5.5: Locking Points for Tex-126-H and Tex-126-E Specimens.

A more systematic way of evaluating the impact of the number of gyrations on density is shown in Figure 5.6a. The densities from the SGC at different numbers of gyrations are compared with those from the TGC for all asphalt contents tested (i.e., 3%, 4.5% or 6%). Overall, the densities obtained with the SGC for the three numbers of gyrations are 2 to 3% less

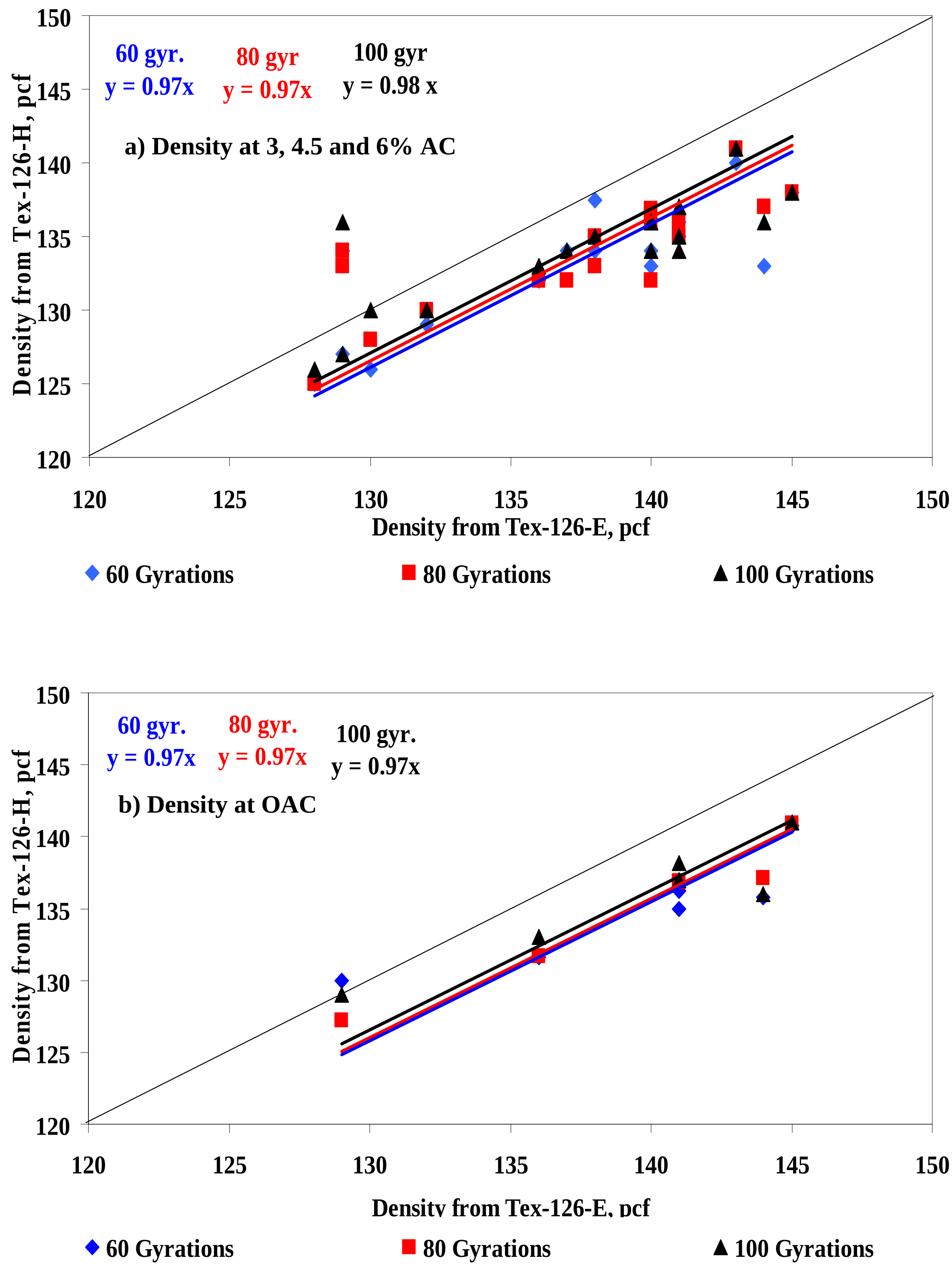


Figure 5.6: Comparison of Density from Tex-126-E and Tex-126-H Protocols.

than those from the TGC. Naturally, the same trends hold for the maximum density as shown in Figure 5.6b. The numbers of gyrations passed 60 do not seem to impact the density of the mix much.

As demonstrated in Chapter 4, the change in density with asphalt content is rather small for the ATB mixes, which may lead to uncertainty in determining the OAC. Another angle in recommending the number of gyrations is the sensitivity of the asphalt-content-density relationships as shown in Table 5.1. The sensitivity is defined as the difference between the maximum and minimum densities at 3%, 4.5% and 6% asphalt contents (used in defining the asphalt content density relationships) for a given compaction protocol, divided by the average density of the three measurements. The greater this value is, the better the asphalt content density curve is defined. In no case the sensitivity is greater than 7%, indicating that perhaps judgment should be used in estimating the OAC in all cases. However, the preliminary data from 80 gyrations seem to be the most promising.

Table 5.1: Sensitivity of Density to Asphalt Content for Tex-126-H Protocol.

Compaction Method	**Sensitivity, (Max. Density – Min Density)/Avg. Density**					
	Beaumont	**El Paso**	**Houston SH-99**	**Houston LP-610**	**Paris**	**Wichita Falls**
SGC 60 Gyrations	1%	2%	3%	2%	6%	5%
SGC 80 Gyrations	3%	4%	3%	4%	5%	5%
SGC 100 Gyrations	1%	2%	1%	2%	7%	5%
TGC	3%	5%	4%	1%	1%	6%

5.1.2 Tex-204-H Protocol

The impact of the number of gyrations on OAC, density, indirect tensile strength and modulus are summarized in Figure 5.7 to Figure 5.10 for Tex-204-H protocol. As reflected in Figure 5.5, the locking points for this protocol are slightly lower than those for Tex-126-H, perhaps because of the smaller specimen heights 4.5 in. as opposed to 8 in. for Tex-126-H). The trends for the OAC's (Figure 5.7) are similar to those presented for Tex-126-H in Figure 5.1 except for the Beaumont material. The patterns for the densities (Figure 5.8) at the OAC's are also similar to those from Tex-126-H.

The number of gyrations does not seem to significantly impact the IDT strengths (Figure 5.9) and moduli (Figure 5.10), and the IDT strengths from the SGC and TGC specimens are closer to one another than the corresponding UCS values from Tex-126-H and Tex-126-E.

The impact of the number of gyrations on density at respective OAC's of the mixes is shown in Figure 5.11. The densities obtained from SGC for the three numbers of gyrations are 2 to 3% less than those from the TGC. In general, the numbers of gyrations above 60 do not significantly or systematically impact the density. As reflected in Table 5.2, the densities of the specimens prepared at 60 or 80 gyrations exhibit higher sensitivity to the asphalt content. Based on this exercise, 60 to 80 gyrations may be more appropriate for preparing specimens.

5.2 IMPACT OF CURING TEMPERATURE

5.2.1 Tex-126-H Protocol

The impacts of curing temperature on the UCS and modulus for specimens prepared as per Tex-126-H are shown in Figures 5.12 and 5.13, respectively. All specimens were prepared at their corresponding OAC's at 100 gyrations. The first series of specimens was cured for 24 hours at room temperature and tested also at room temperature. The second series of specimens was cured in an oven at 140°F for 48 hours but tested at room temperature, while the third series of tests was carried out on specimens cured for 48 hours at 140°F and tested at that temperature.

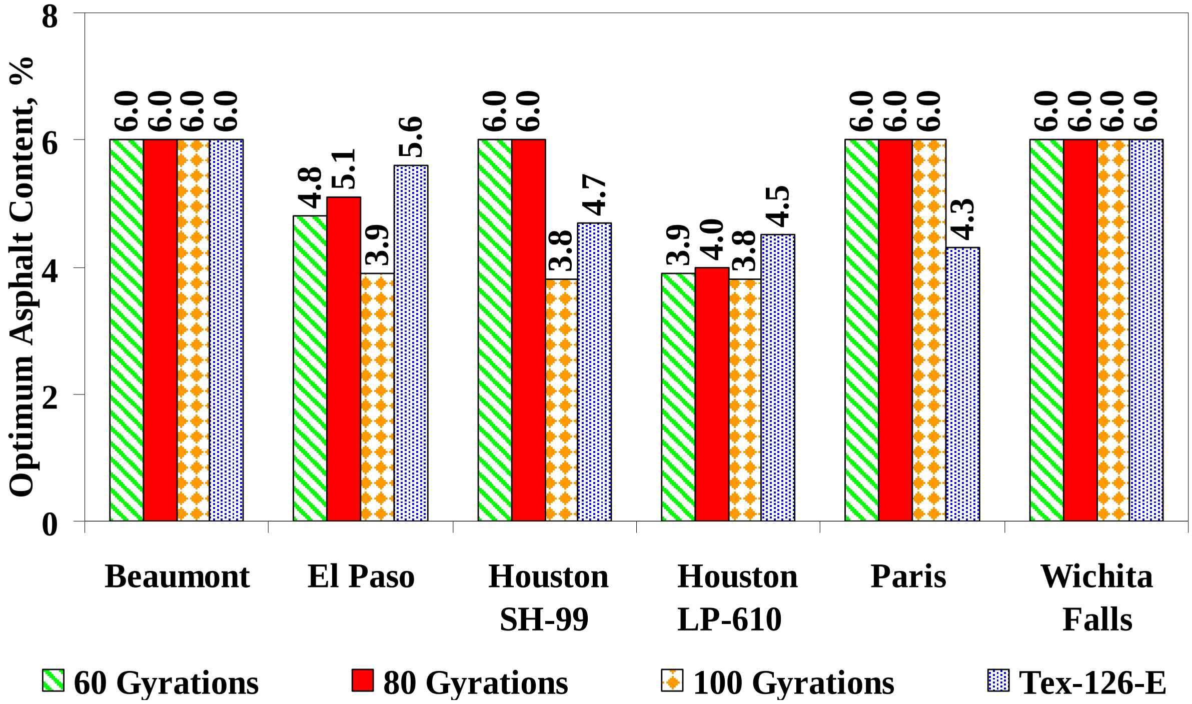


Figure 5.7: Impact of Number of Gyrations on OAC for Tex-204-H Protocol.

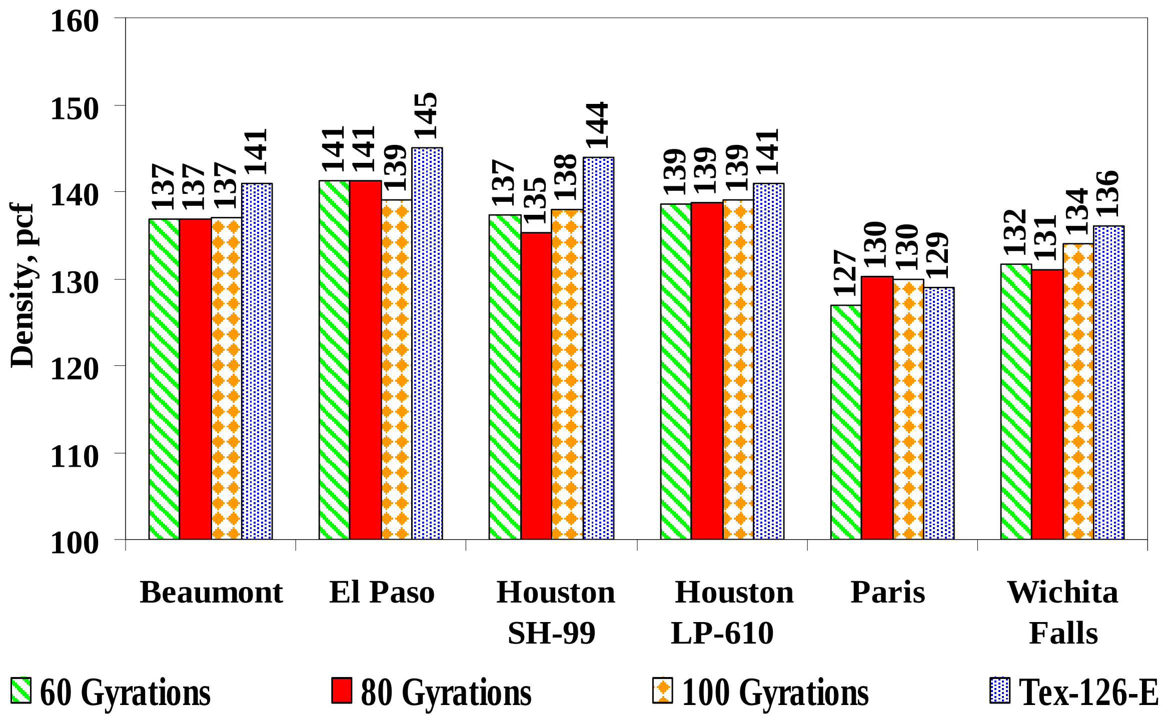


Figure 5.8: Impact of Number of Gyrations on Density for Tex-204-H Protocol.

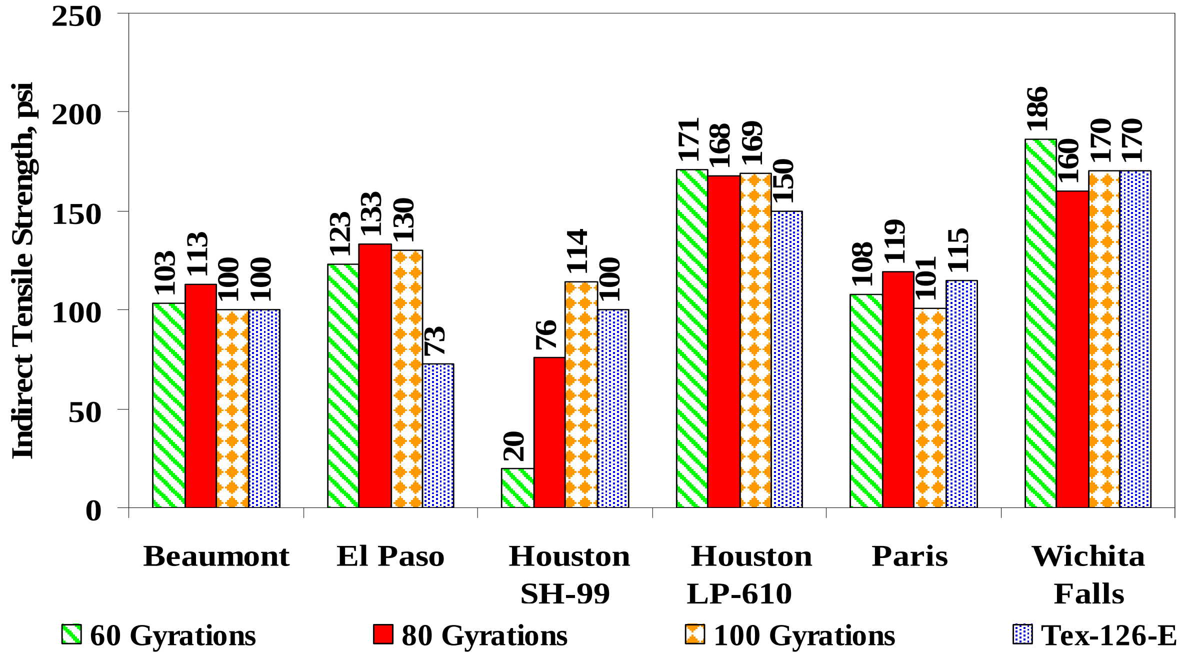


Figure 5.9: Impact of Number of Gyrations on IDT Strength for Tex-204-H Protocol.

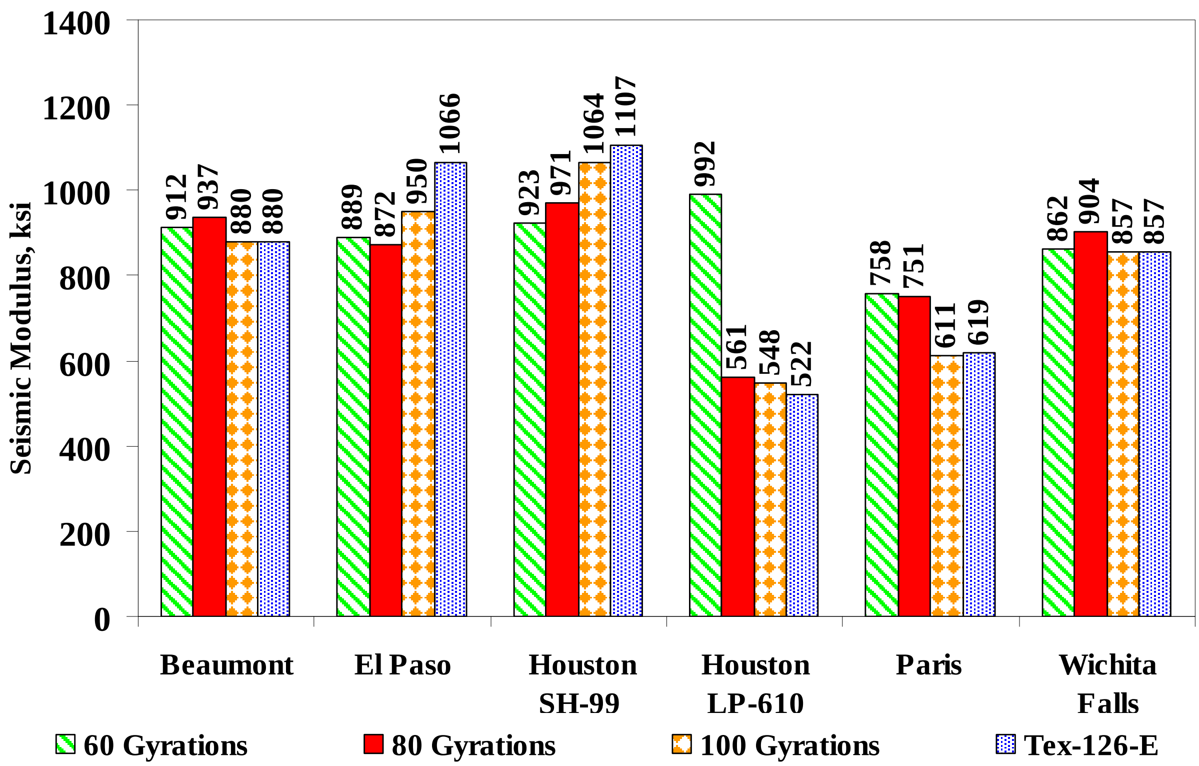


Figure 5.10: Impact of Number of Gyrations on Modulus for Tex-204-H Protocol.

Table 5.2: Sensitivity of Density to Asphalt Content for Tex-204-H Protocol.

Compaction Method	**Sensitivity, (Max. Density – Min Density)/Avg. Density**					
	Beaumont	**El Paso**	**Houston SH-99**	**Houston LP-610**	**Paris**	**Wichita Falls**
SGC 60 Gyrations	6%	5%	1%	3%	4%	5%
SGC 80 Gyrations	3%	6%	1%	3%	9%	3%
SGC 100 Gyrations	3%	3%	2%	3%	6%	4%
TGC	3%	5%	4%	1%	1%	6%

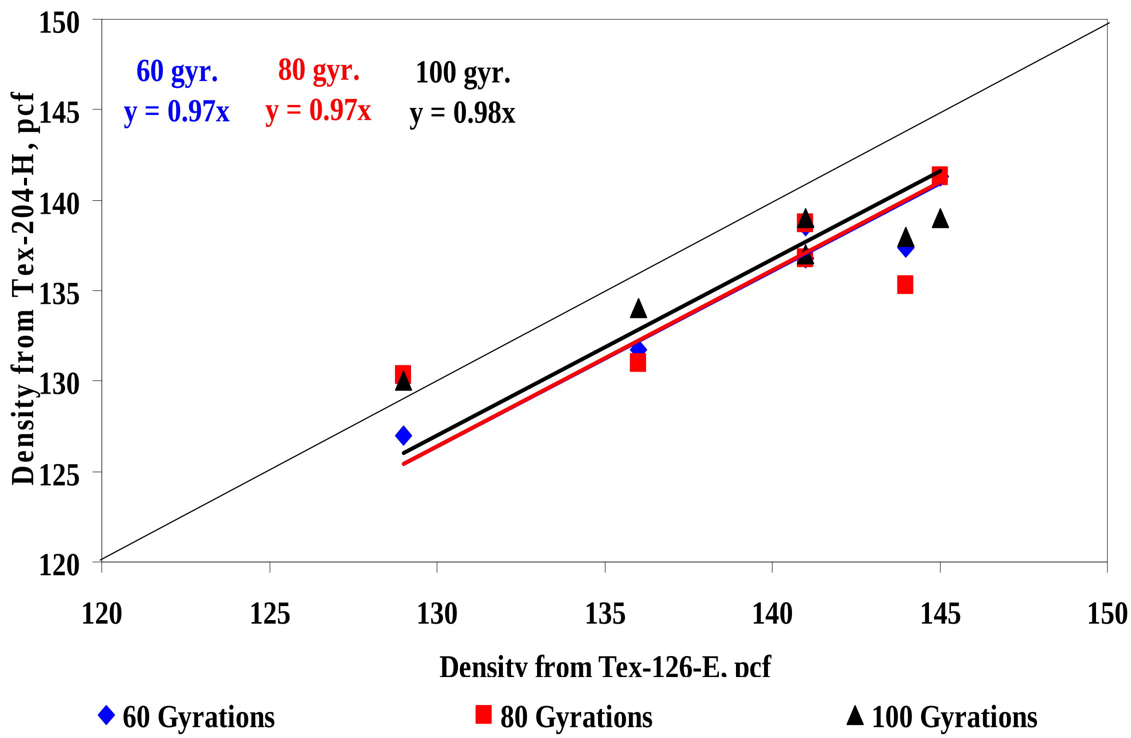


Figure 5.11: Comparison of Density from Tex-126-E and Tex-204-H Protocols.

From the first two sets of data, the 48 hours of curing in the oven does not seem to significantly impact the strength or modulus of the mixes. As such, instead of curing the specimens for 48 hrs, one can simply cure them for 24 hrs at room temperature to save time and complications with handling high-temperature specimens.

As expected, the impact of the temperature at the time of testing is more pronounced comparing the second and third sets of data. Perhaps the UCS and modulus testing can also be done at the room temperature to mainstream the testing and to improve the reliability of the results since the operator is not rushed to test the specimen in order to maintain the temperature of the specimen.

The strengths from the two alternative curing processes are compared with the standard ones (i.e. 48 hrs of curing at 140°F and testing at 140°F) in Figure 5.14. The strengths from the Tex-126-H specimens after subjecting them to standard curing were similar to those of Tex-126-E, indicating that the compactor used for sample preparation has little impact on the UCS strength. However, the Tex-126-H specimens tested at 77oF exhibit strengths that are about 3.4 times greater than those tested under Tex-126-E protocol, independent of the number of days (1 or 2 days) that the Tex-126-H specimens were cured.

To further verify this concept, the specimens prepared using Tex-126-E protocol were subjected to three curing regimes and tested. As shown in Figure 5.15, the trends are similar to those discussed above. There is more scatter in the data that can perhaps be attributed to the better quality of the specimens prepared by the SGC. The impact of the temperature is also less evident, perhaps due to the fact that the TGC specimens are denser requiring more than 4 hrs to cool to room temperature.

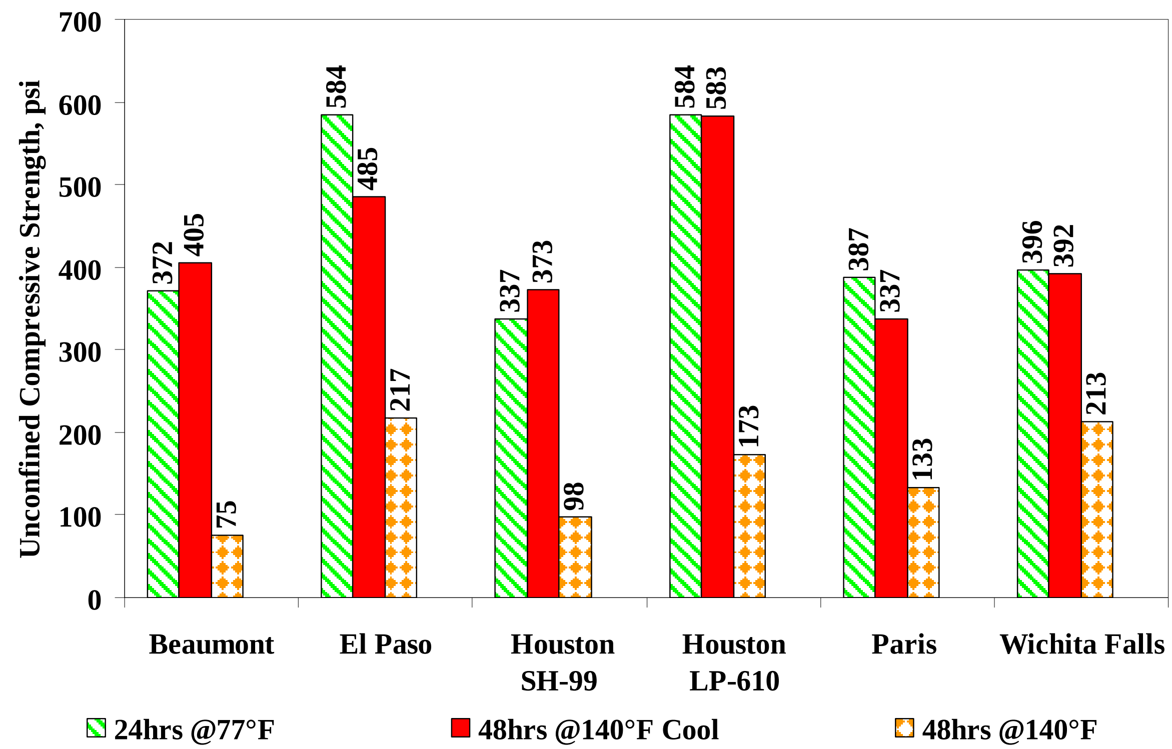


Figure 5.12: Impact of Curing Temperature on UCS for Tex-126-H Protocol.

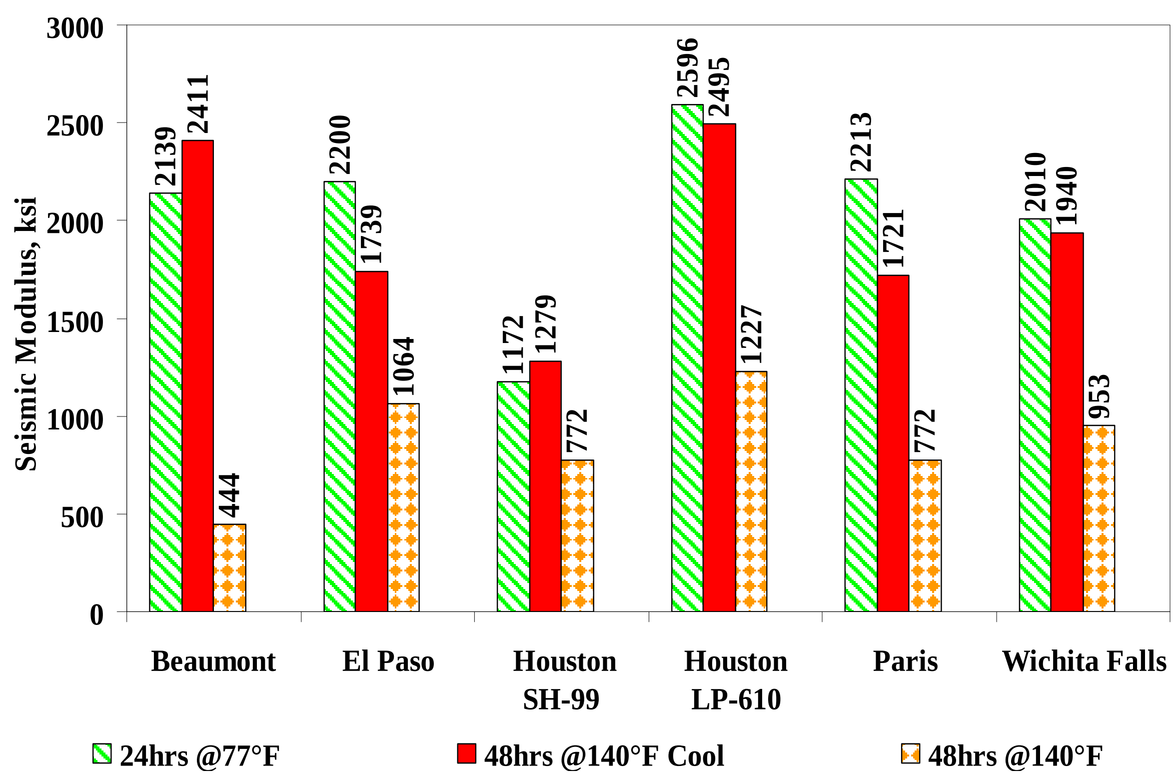


Figure 5.13: Impact of Curing Temperature on Modulus for Tex-126-H Protocol.

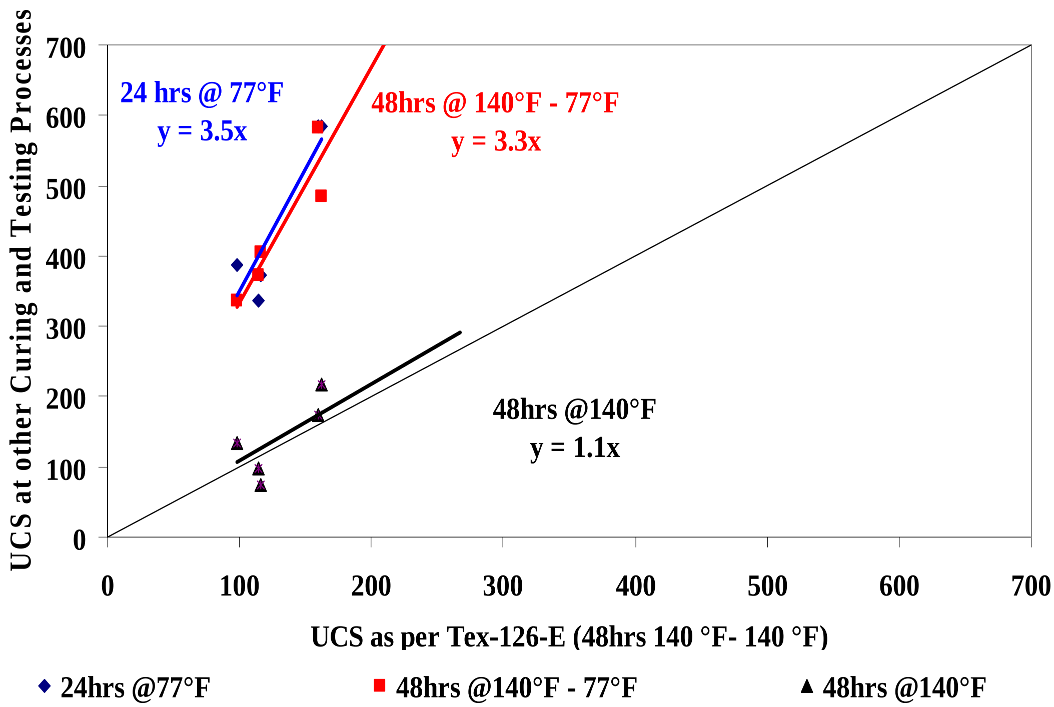


Figure 5.14: Comparison of Strengths from Different Curing Regimes using Tex-126-H Specimens with Those from Tex-126-E.

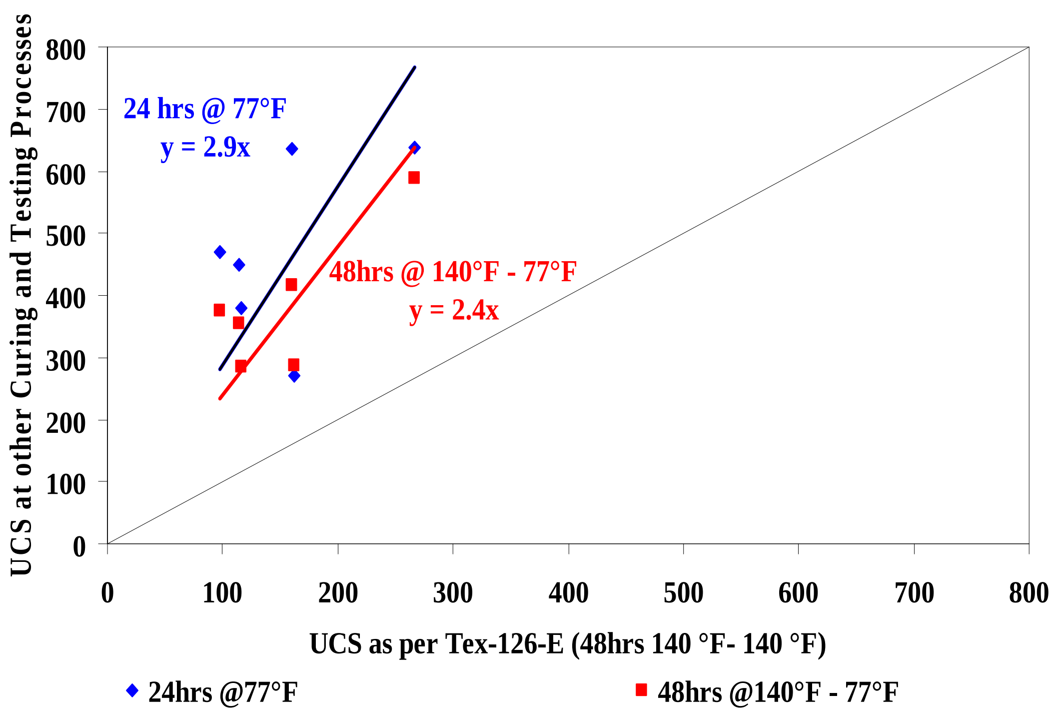


Figure 5.15: Comparison of Strengths from Different Curing Regimes using Tex-126-E Specimens.

5.2.2 Tex-204-H Protocol

The impacts of curing temperature on IDT and modulus for specimens prepared as per Tex-204-H are shown in Figures 5.16 and 5.17, respectively. Once again, the trends are similar to those from Tex-126-H. As reflected in Figure 5.18, specimens cured either one or two days at 140oF and tested at 77oF yielded similar results. However, when the specimens were tested at 140oF, the strengths were about 4 times less than those tested at 77°F.

5.3 IMPACT OF BINDER GRADE

5.3.1 Tex-126-H Protocol

The OAC's for the specimens prepared as per Tex-126-H using PG 64-22 and PG 76-22 binders for all materials are shown in Figure 5.19. The OAC's are less for the PG 64-22 binder except for the Wichita Falls material. The Paris mixes were not tested because of the lack of raw materials.

The impact of the binder grade on the densities of the materials is rather small (less than 2 pcf) for most materials as depicted in Figure 5.20. Wichita Falls is the only material that is experiencing a significant increase in the density with the PG 76-22 binder. The reason for this pattern is unknown.

The UCS values for the Tex-126-H specimens mixed with different binder grades at their respective OAC's are shown in Figure 5.21. Because of the complex interaction of the AC content, a clear pattern cannot be observed. The specimens with the 76-22 binder seem to be stronger in compression for the Houston SH-99, Houston LP-610 and Wichita Falls materials. The Beaumont material showed no impact of the binder grade, and the El Paso material is the only material in which the PG 64-22 specimen showed a higher UCS value.

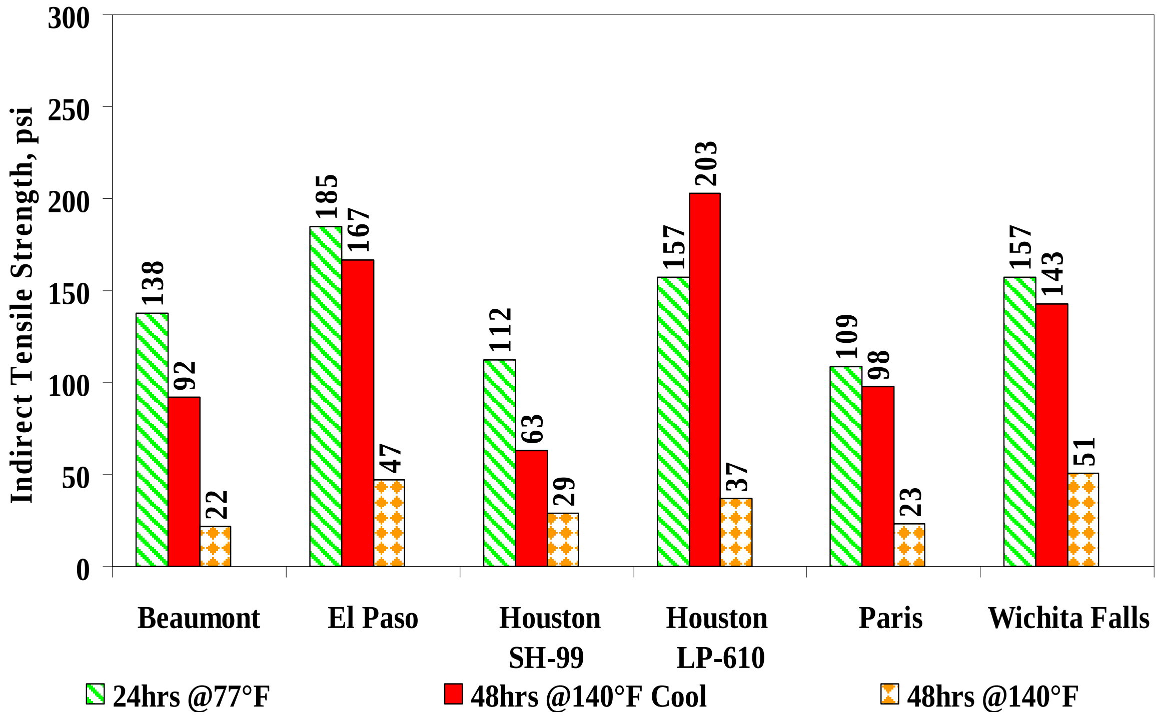


Figure 5.16: Impact of Curing Temperature on IDT Strength for Tex-204-H Protocol.

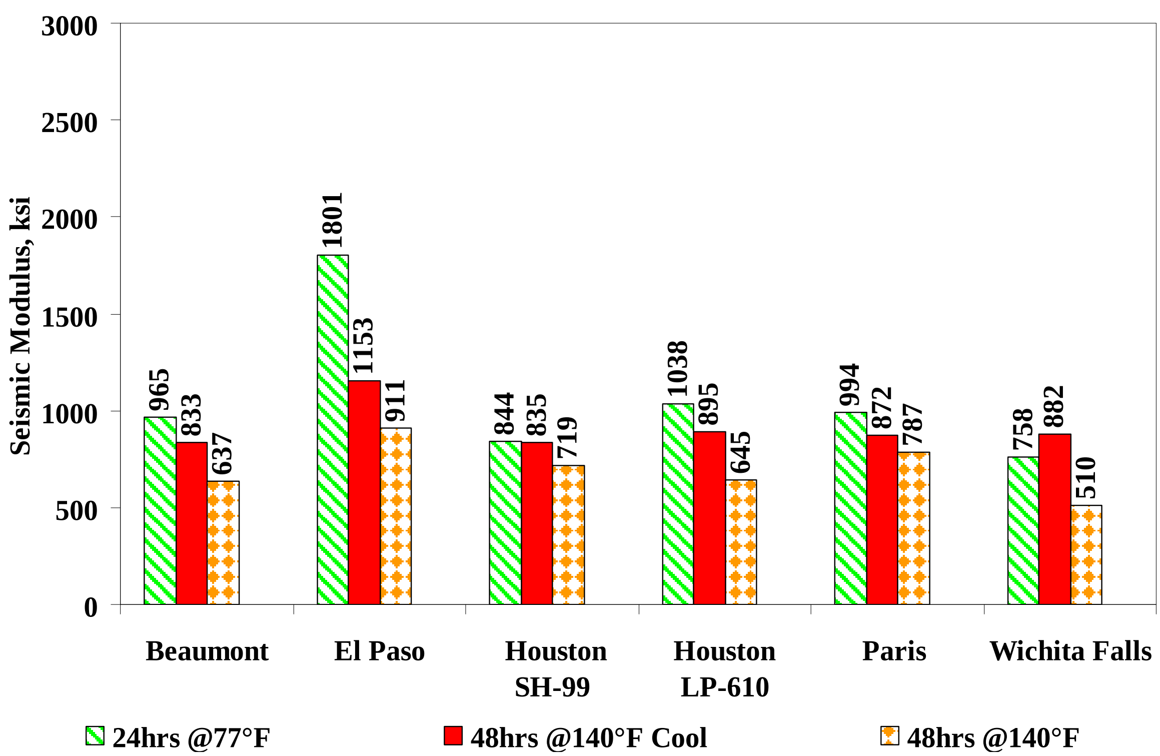


Figure 5.17: Impact of Curing Temperature on Modulus for Tex-204-H Protocol.

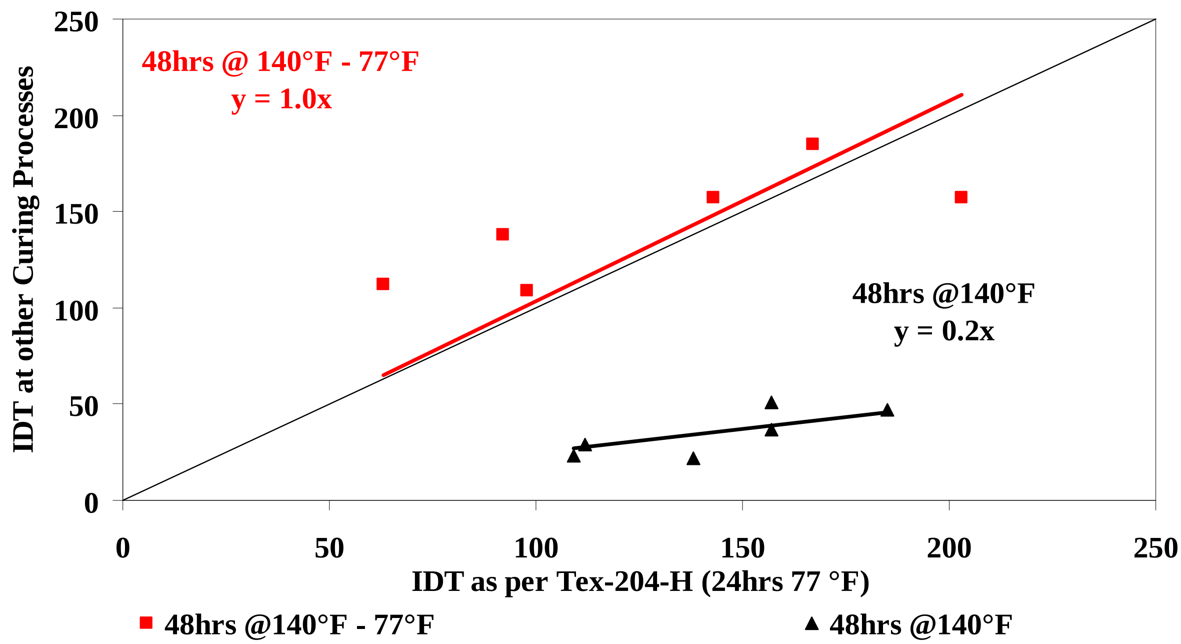


Figure 5.18: Comparison of Strengths from Different Curing Regimes using Tex-204-H Specimens.

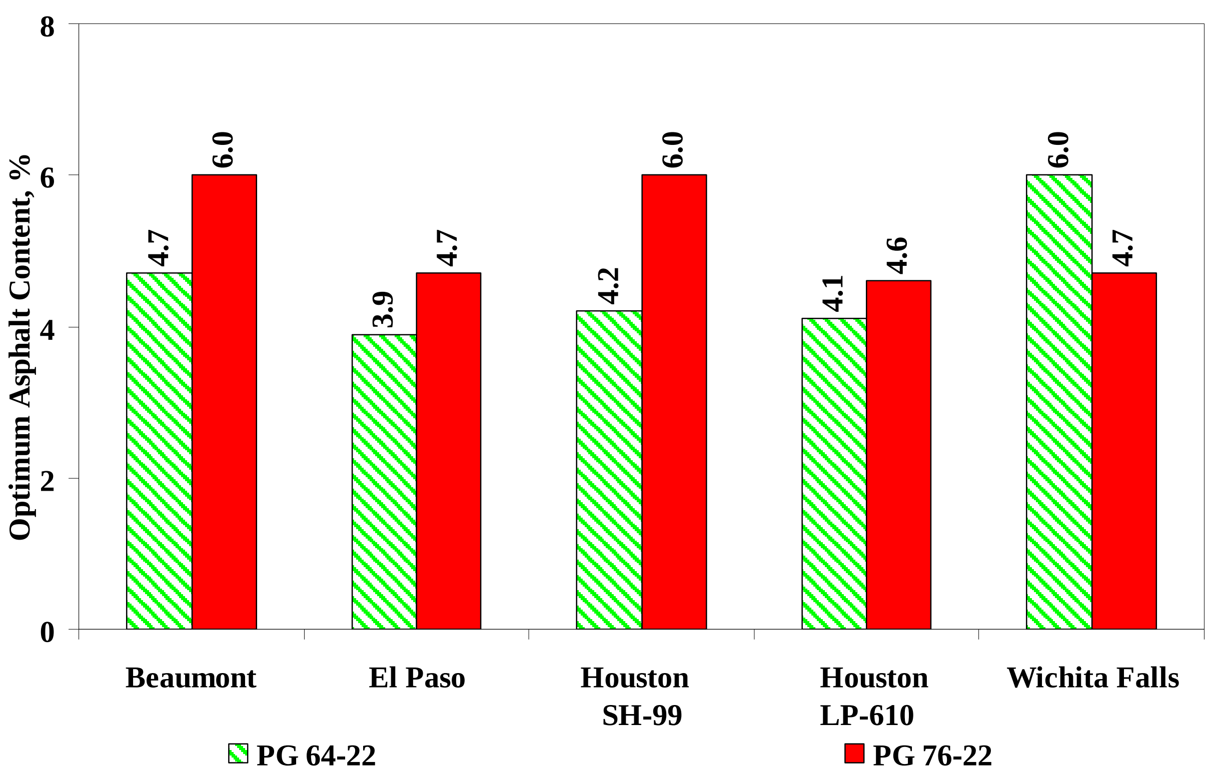


Figure 5.19: Impact of Binder Grade on OAC based on Density for Tex-126-H Protocol.

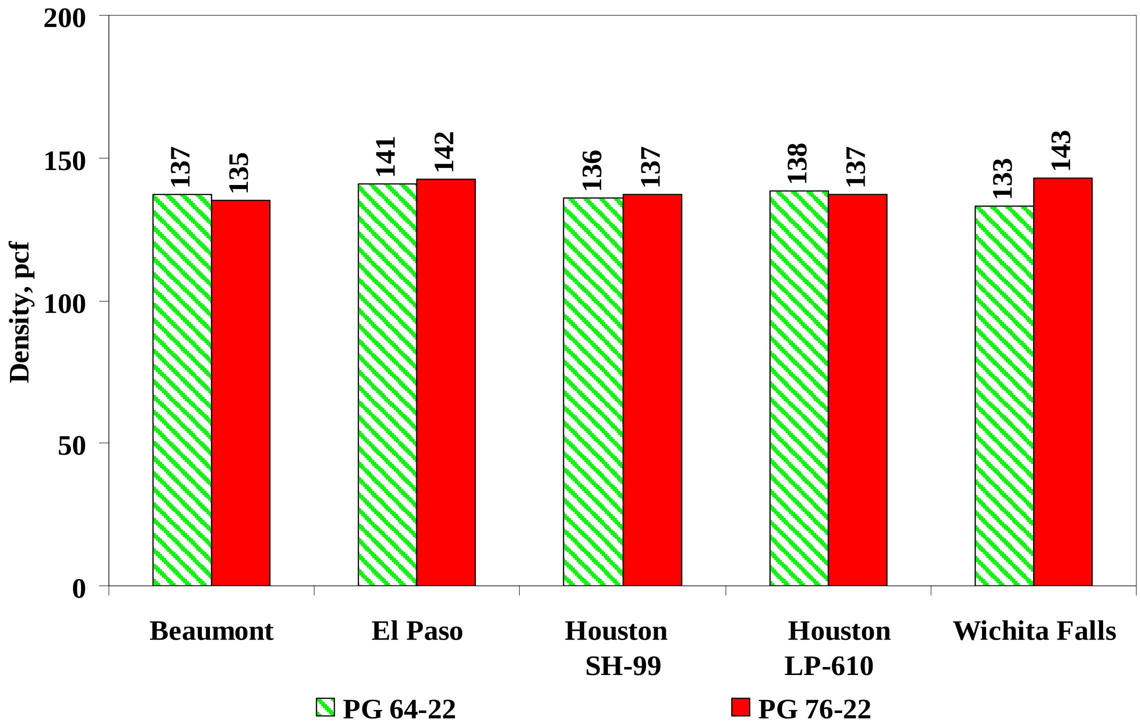


Figure 5.20: Impact of Binder Grade on Density for Tex-126-H Protocol.

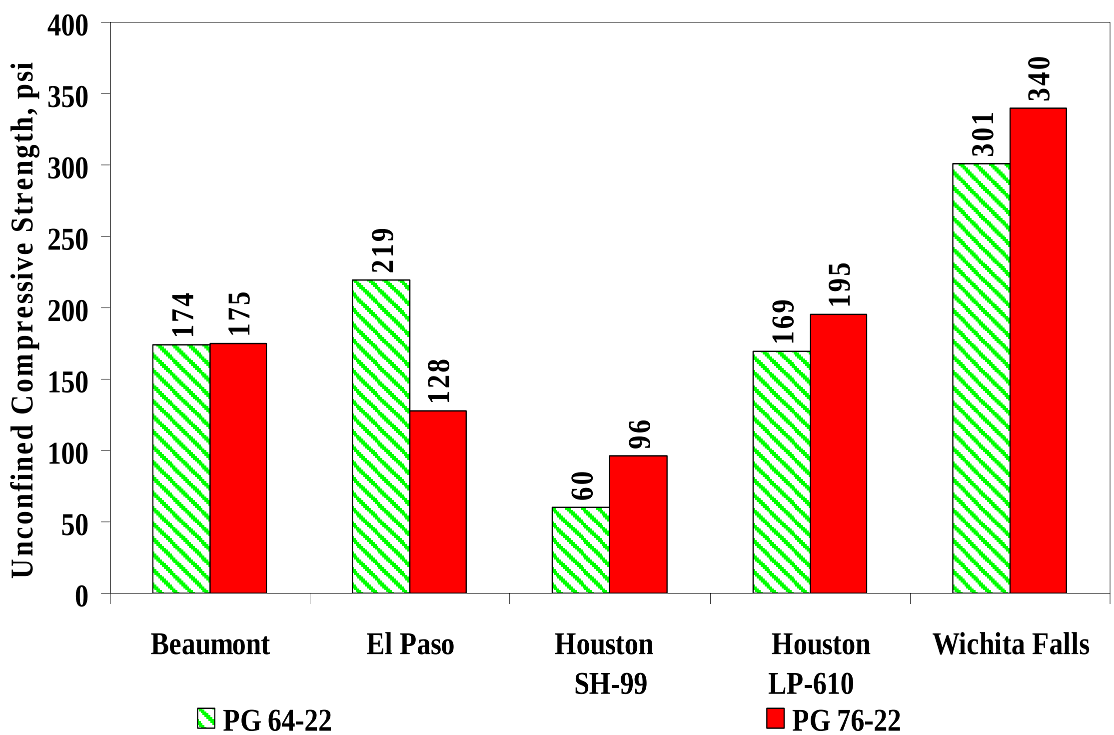


Figure 5.21: Impact of Binder Grade on UCS for Tex-126-H Protocol.

5.3.2 Tex- 204-H Protocol

The OAC's based on density obtained from the two binders from the Tex-204-H protocol are compared in Figure 5.22. In this case, the OAC's from the two binders are closer to one another. As shown in Figure 5.23, the densities were not impacted significantly by the binder grade. They are generally within 3 pcf of one another.

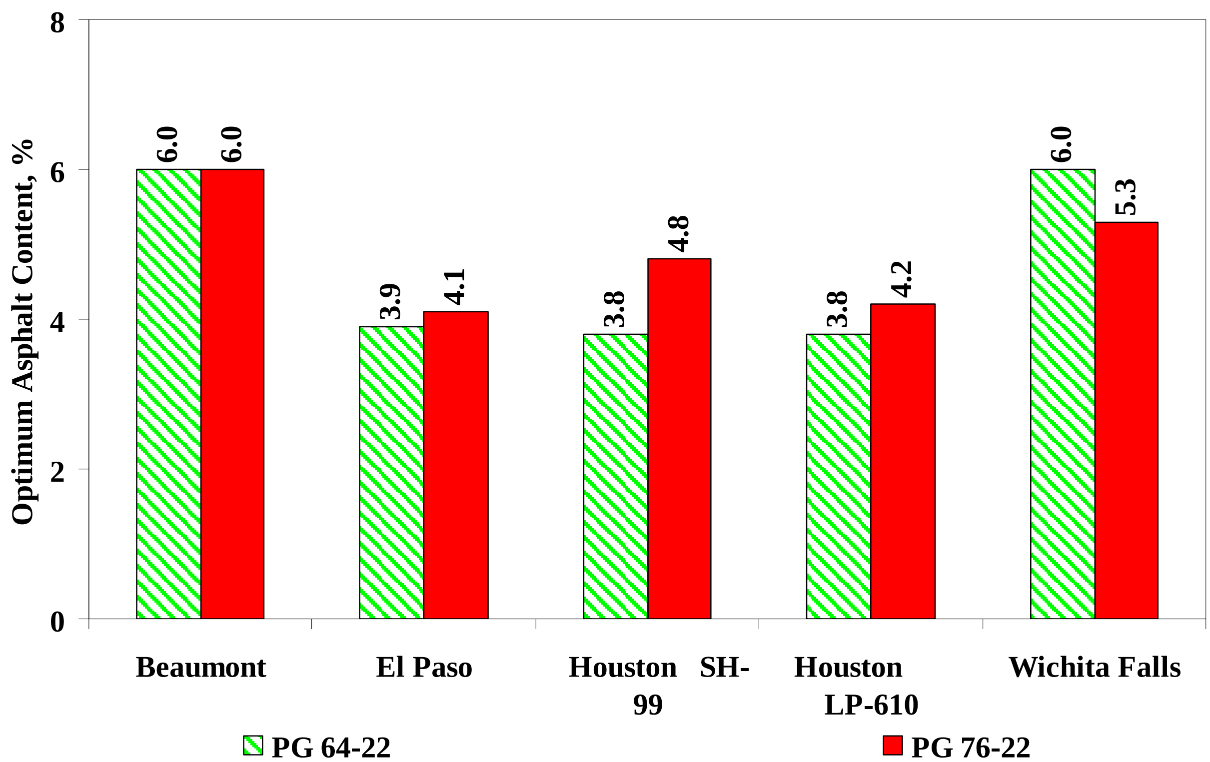


Figure 5.22: Impact of Binder Grade on OAC for Tex-204-H Protocol.

The impact of the binder grade on the IDT strengths is shown in Figure 5.24. Unlike the UCS trends in Figure 5.21, the PG 76-22 specimens seem to be stronger under tension as compared to the PG 64-22 specimens for almost all materials. The trends for seismic moduli, as shown in Figure 5.25, are different. The reason for this contradiction is that the modulus is more indicative of the compressive strength than the tensile strength.

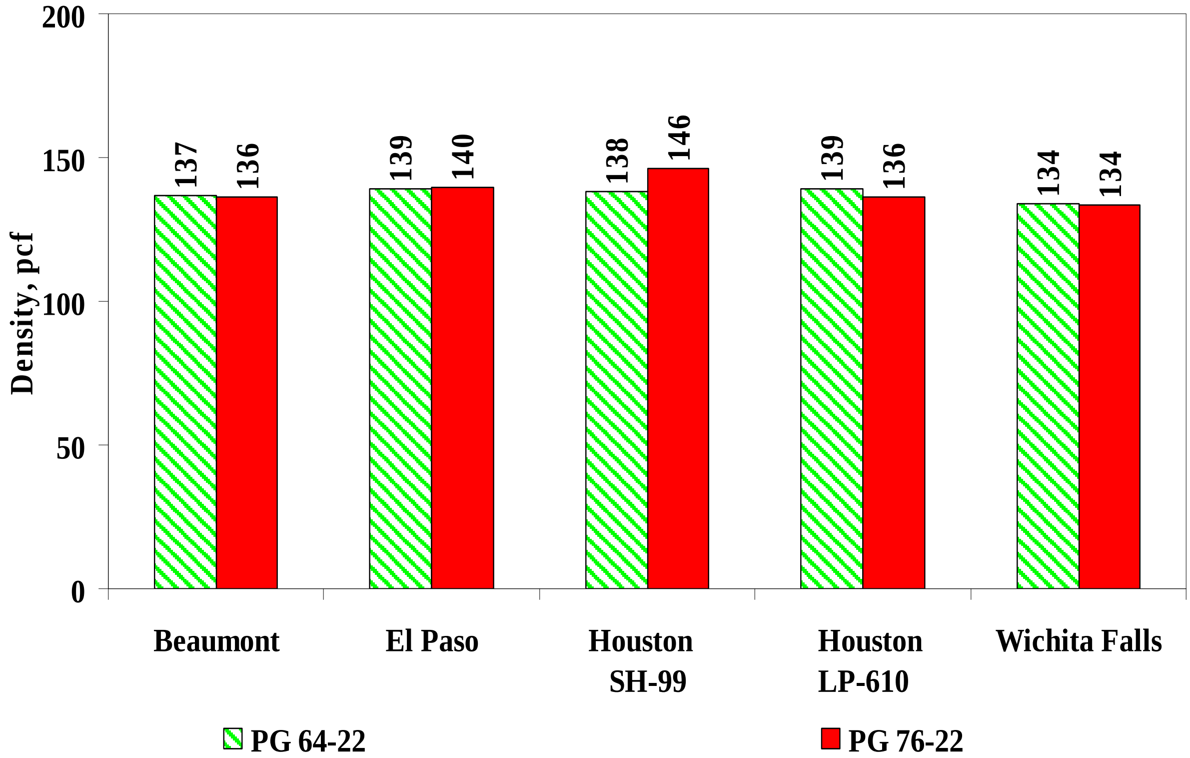


Figure 5.23: Impact of Binder Grade on Density for Tex-204-H Protocol.

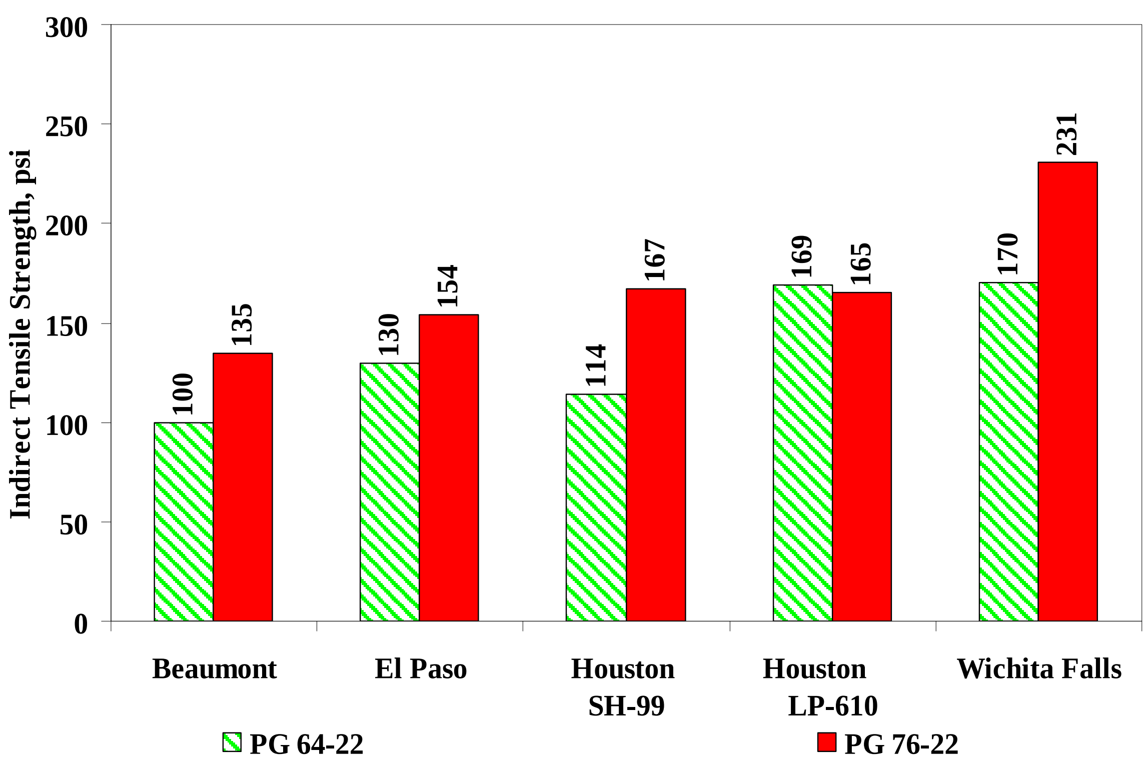


Figure 5.24: Impact of Binder Grade on IDT Strength for Tex-204-H Protocol.

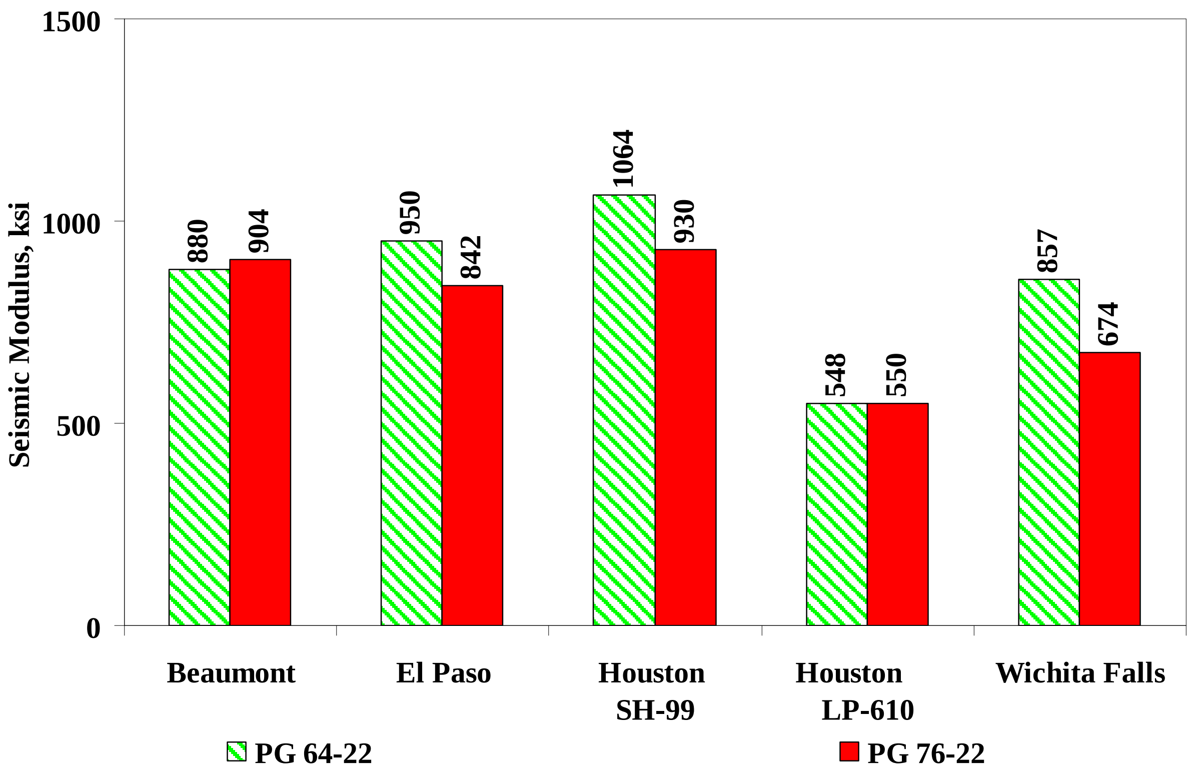


Figure 5.25: Impact of Binder Grade on Modulus for Tex-204-H Protocol.

5.4 Impact of AC Content

Each material was tested at its OAC and ±1 of its OAC to observe how the change in binder content will impact the behavior of the mix. The specimens were cured for 24 hrs and tested at 77°F. The variations in the UCS with asphalt content for Tex-126-H specimens are shown in Figure 5.26. Most materials are stronger either at the OAC or 1% less than OAC. A clear pattern is not obvious because as indicated in Chapter 4 the selection of OAC based on the density is rather uncertain. The modulus pattern is not that different as shown in Figure 5.27.

The IDT strengths and seismic moduli for the Tex-204-H specimens are shown in Figures 5.28 and 5.29, respectively. In most cases, the change in AC content does not seem to impact the IDT strength and modulus much.

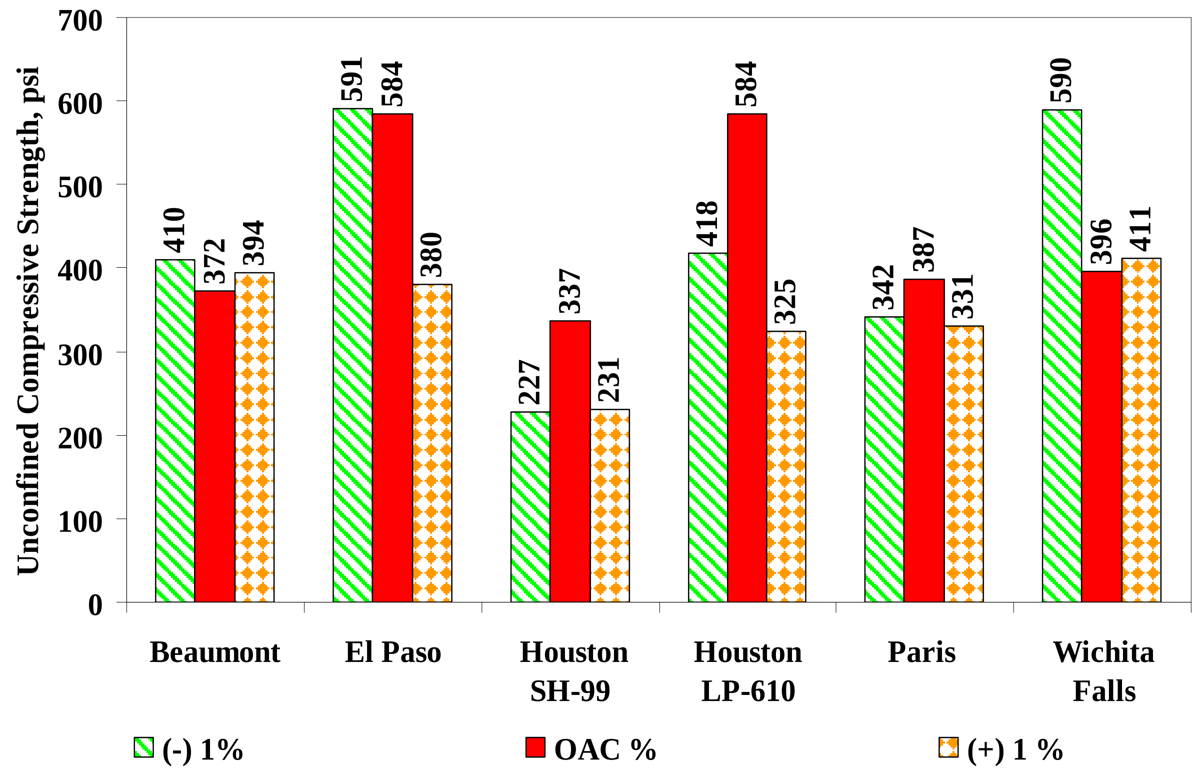


Figure 5.26: Impact of Asphalt Content Variation on UCS for Tex-126-H Protocol.

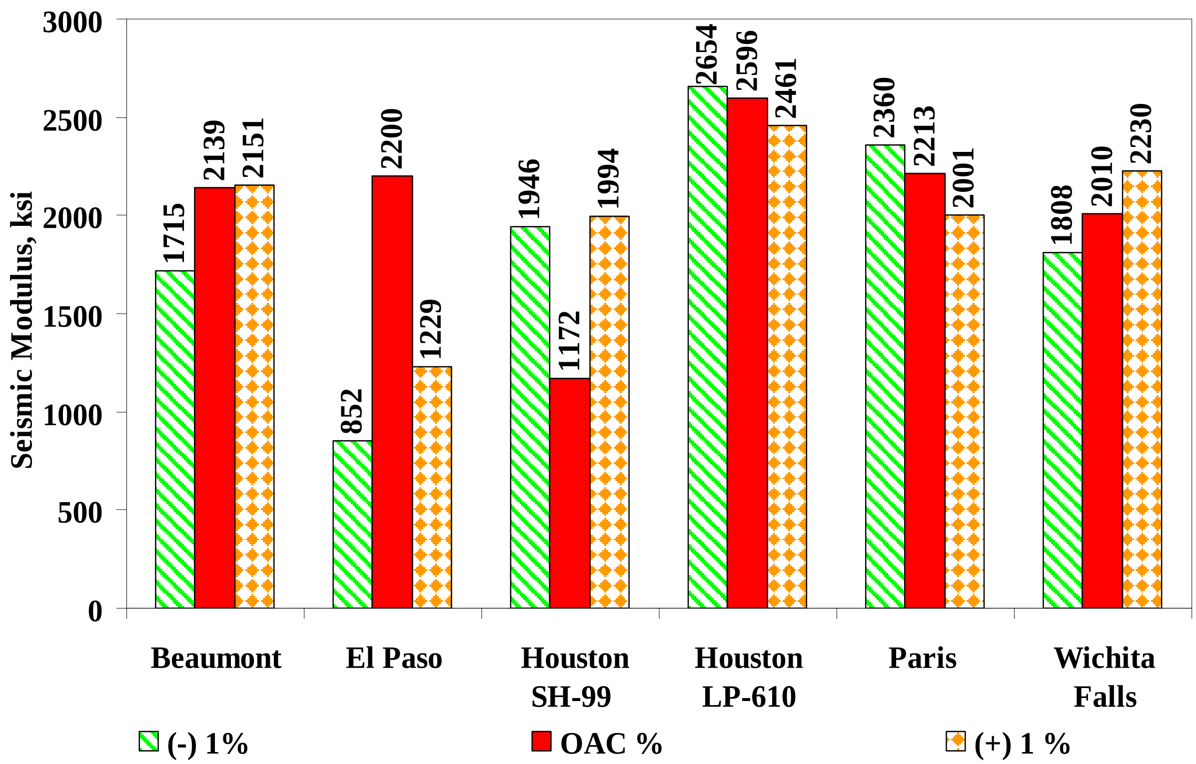


Figure 5.27: Impact of Asphalt Content Variation on Modulus for Tex-126-H Protocol.

5.5 IMPACT OF GRADATION

Up to this point, all the specimens were molded using the in place gradation of the materials as delivered to UTEP. As indicated in Chapter 4, those gradations met the Item 344 gradations. A second gradation that was compatible with Item 292 was developed for each material by increasing the fine contents in order to quantify the importance of the gradation in a mix. As reflected in Table 5.3, the modified gradations (values in parenthesis) were obtained by modifying only the materials finer than No. 40 Sieve so that 15% of the materials passed the number 200 sieve (while the original gradation contained less than 5%). This experiment was carried out on four materials.

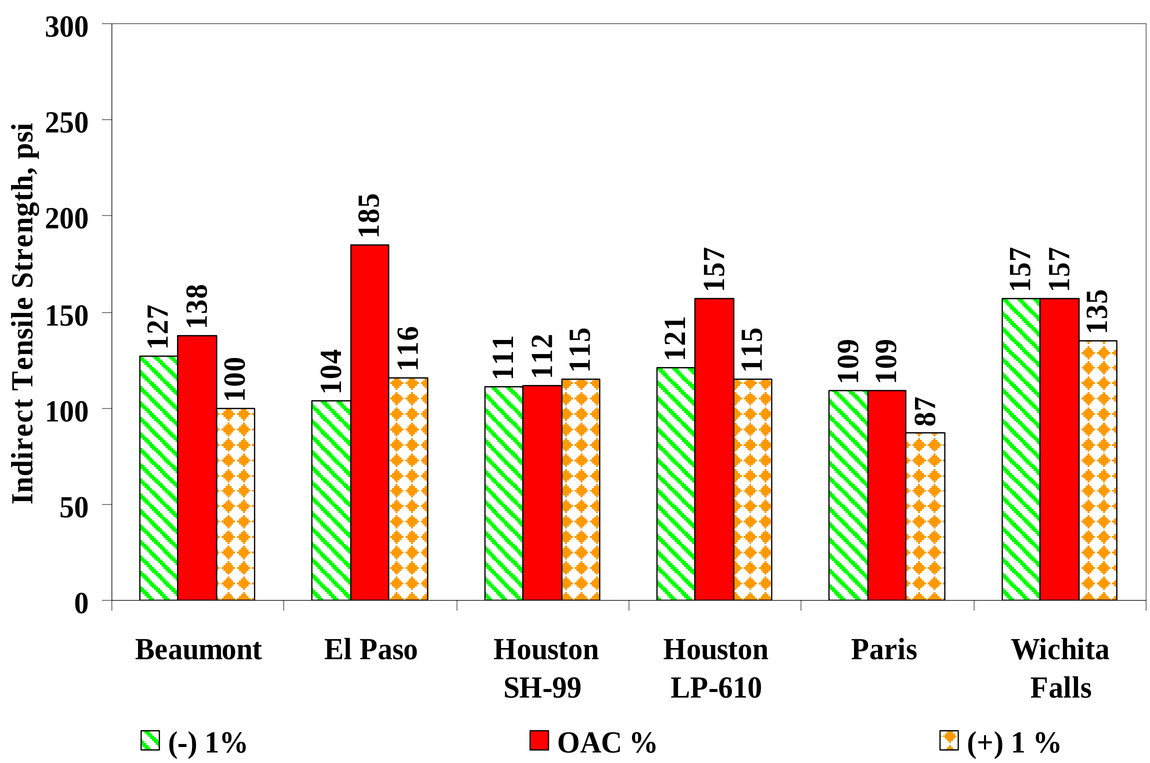


Figure 5.28: Impact of Asphalt Content Variation on IDT for Tex-204-H Protocol.

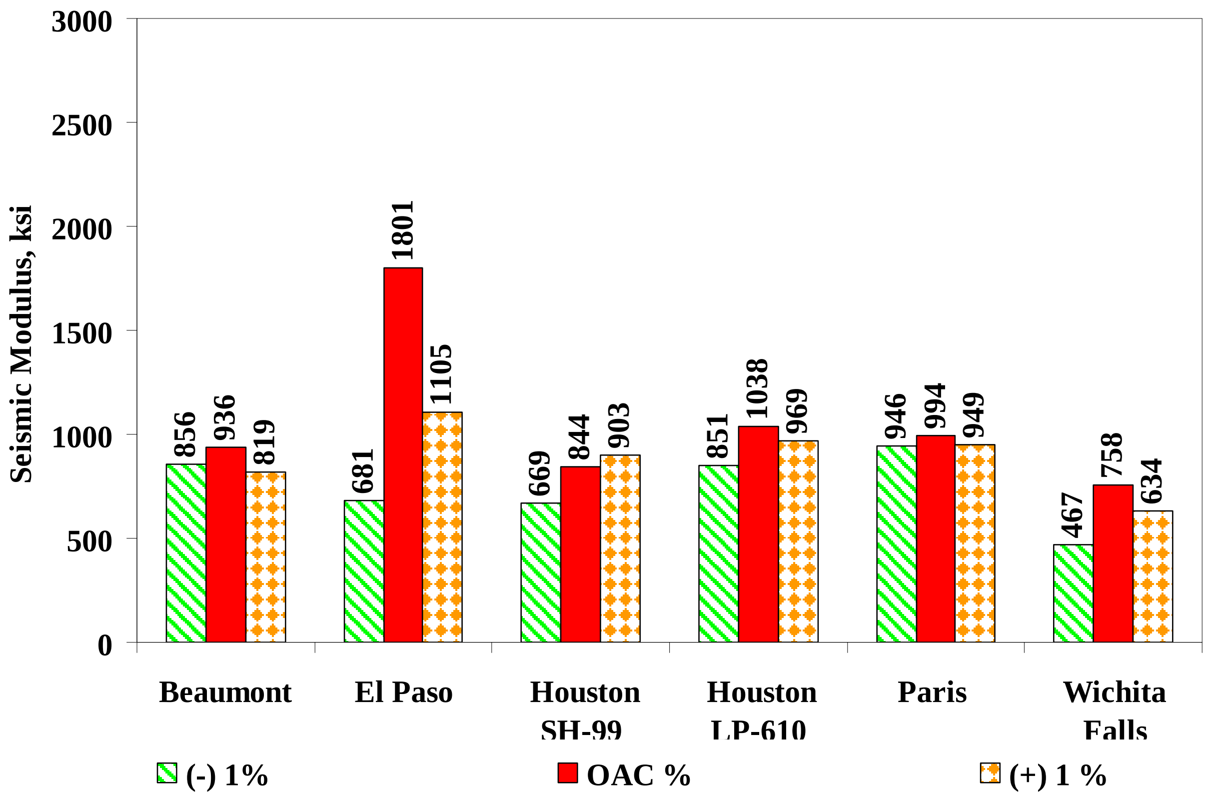


Figure 5.29: Impact of Asphalt Content Variation on Modulus for Tex-204-H Protocol.

5.5.1 Tex-126-H Protocol

Figure 5.30 compares the OACs for the Tex-126-H protocol from the two different gradations used. The quantities of the asphalt needed to achieve the maximum density for the Item 292 gradations are higher or close to those from the Item 344 gradations. The changes in maximum densities are not significant as the fines content increases, but the high fines content specimens' densities were found equal or slightly higher than the densities of the original gradations (see Figure 5.31).

As reflected in Figures 5.32 and 5.33, the addition of fines negatively or positively impacts the UCS and modulus depending on the original gradations of the materials as retrieved from the field.

Table 5.3: Original and Modified Gradation.

Sieve Size	**Particle Diameter (mm)**	**Beaumont**	**El Paso**	**Houston SH-99**	**Houston LP-610**	**Paris**	**Wichita Falls**
2”	**58**	100	100	100	100	100	100
1.5”	**38.1**	100	100	100	100	100	100
1"	**25.4**	83	100	73	91	87	91
7/8"	**22.4**	76	78	64	88	71	88
3/8"	**9.52**	56	60	38	63	46	61
#4	**4.75**	41	45	28	37	30	44
#40	**0.425**	18 (26)*	23 (26)	19 (22)	20 (26)	14 (22)	16 (25)
#100	**0.15**	3 (19)	12 (18)	9 (19)	5 (19)	8 (18)	8 (18)
#200	**0.075**	1 (15)	5 (15)	1 (15)	2 (15)	1 (15)	5 (15)

* Numbers in parentheses corresponds to modified gradations that would meet Item 292 requirements.

5.5.2 Tex-204-H Protocol

The OACs for Item 292 and Item 344 gradations for Tex-204-H specimens are compared in Figure 5.34. The changes in the OACs as a result of changes in gradations seem to be more pronounced for this protocol as opposed to Tex-126-H. The changes in densities, however, are again rather small except for the Wichita Falls material (see Figure 5.35).

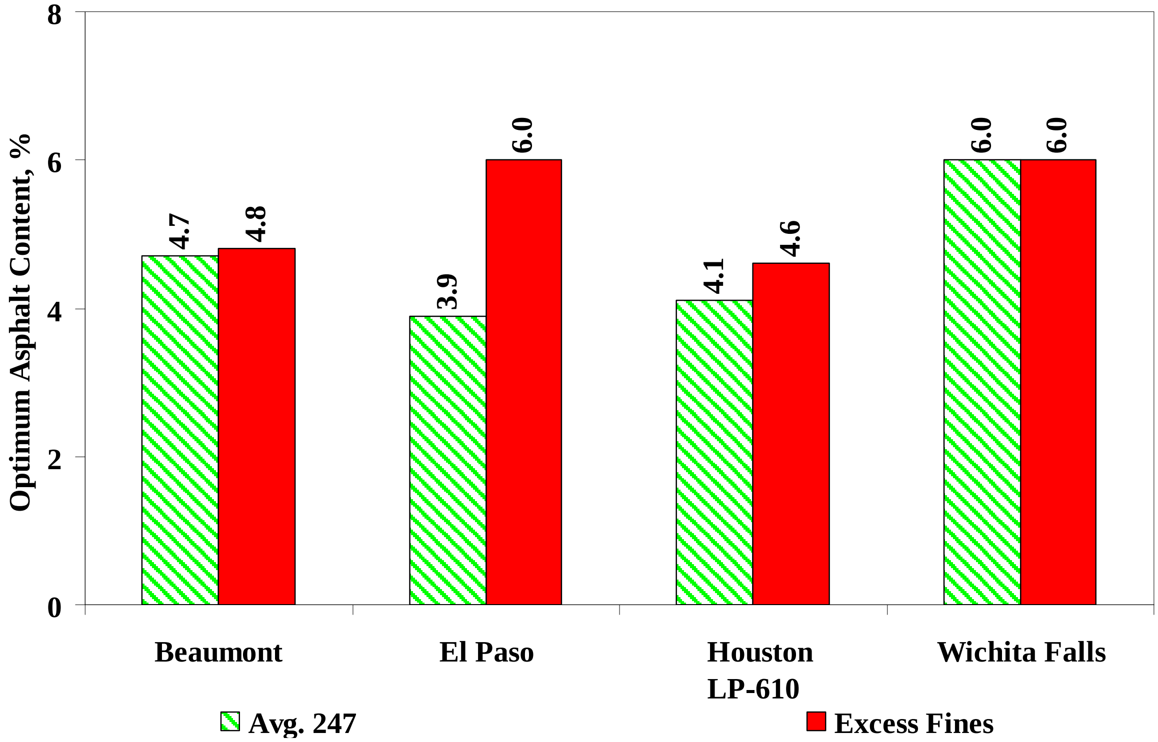


Figure 5.30: Impact of Fines Content Variation on OAC Tex-204-H Protocol.

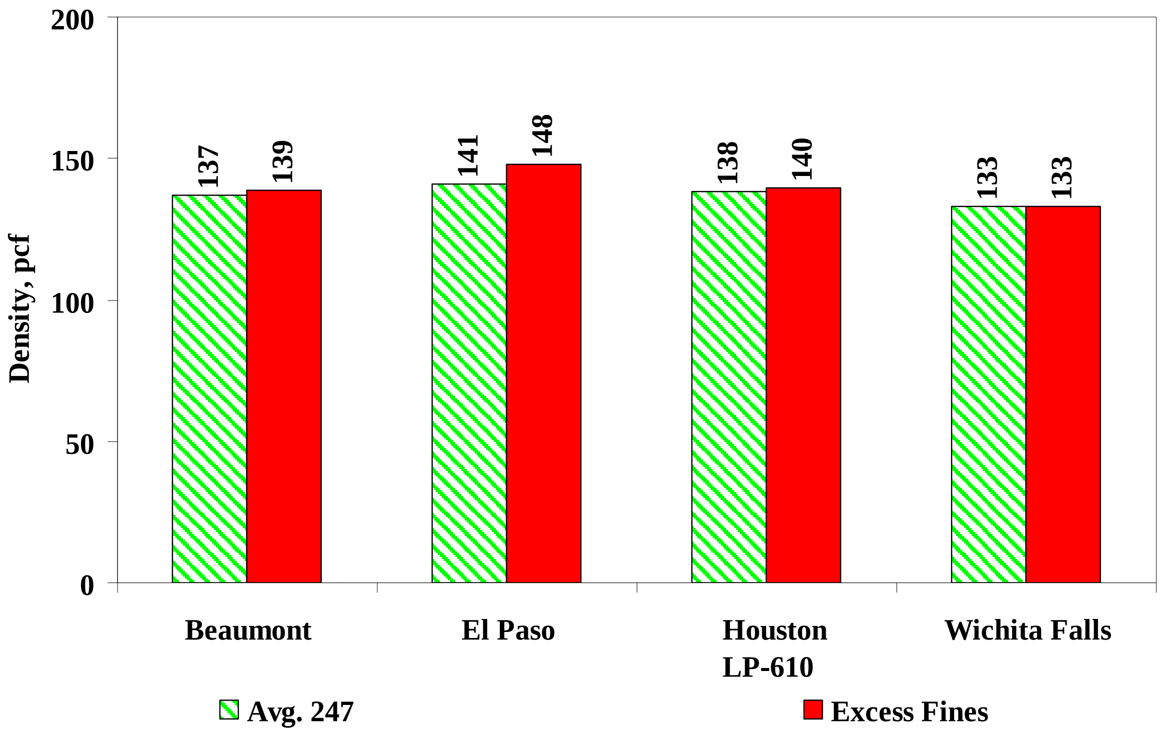


Figure 5.31: Impact of Fines Content Variation on Density Tex-204-H Protocol.

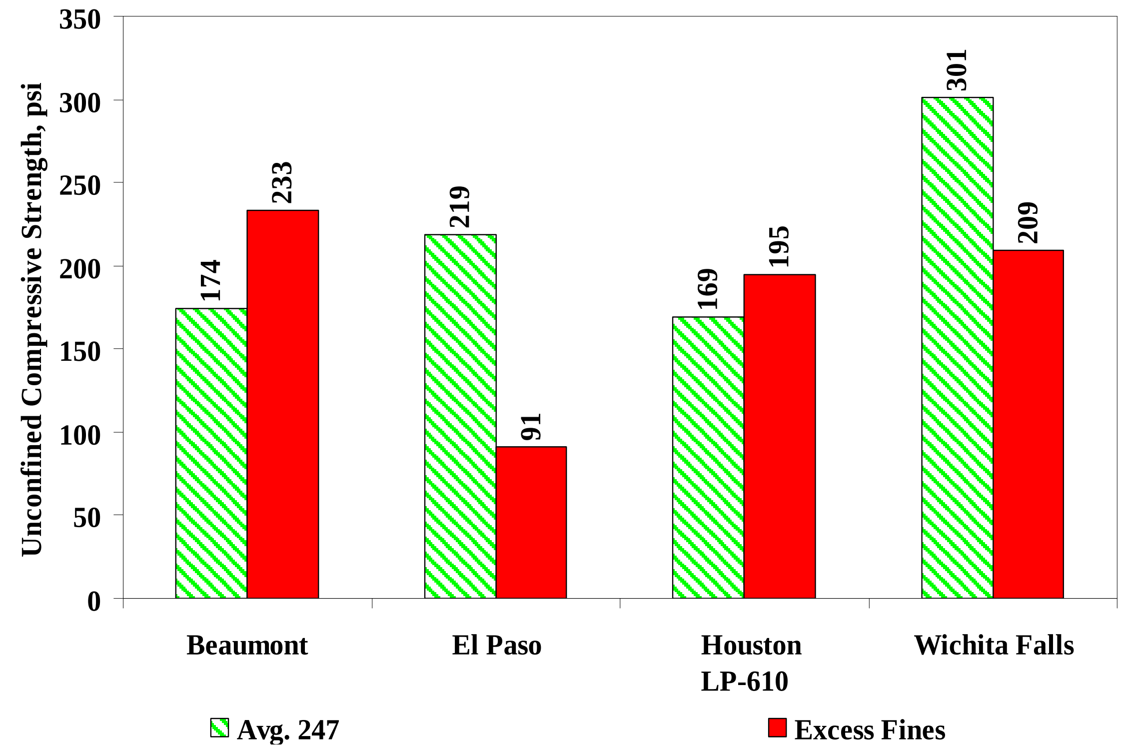


Figure 5.32: Impact of Fines Content Variation on UCS Tex-126-H Protocol.

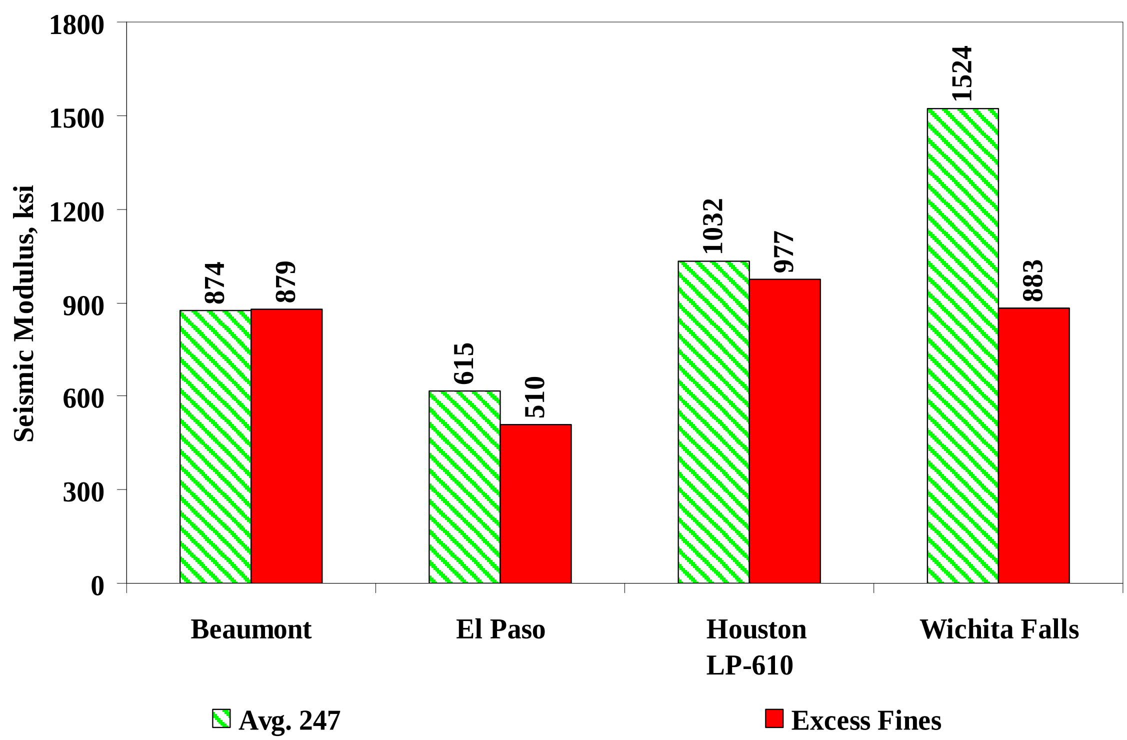


Figure 5.33: Impact of Fines Content Variation on Modulus for Tex-126-H Protocol.

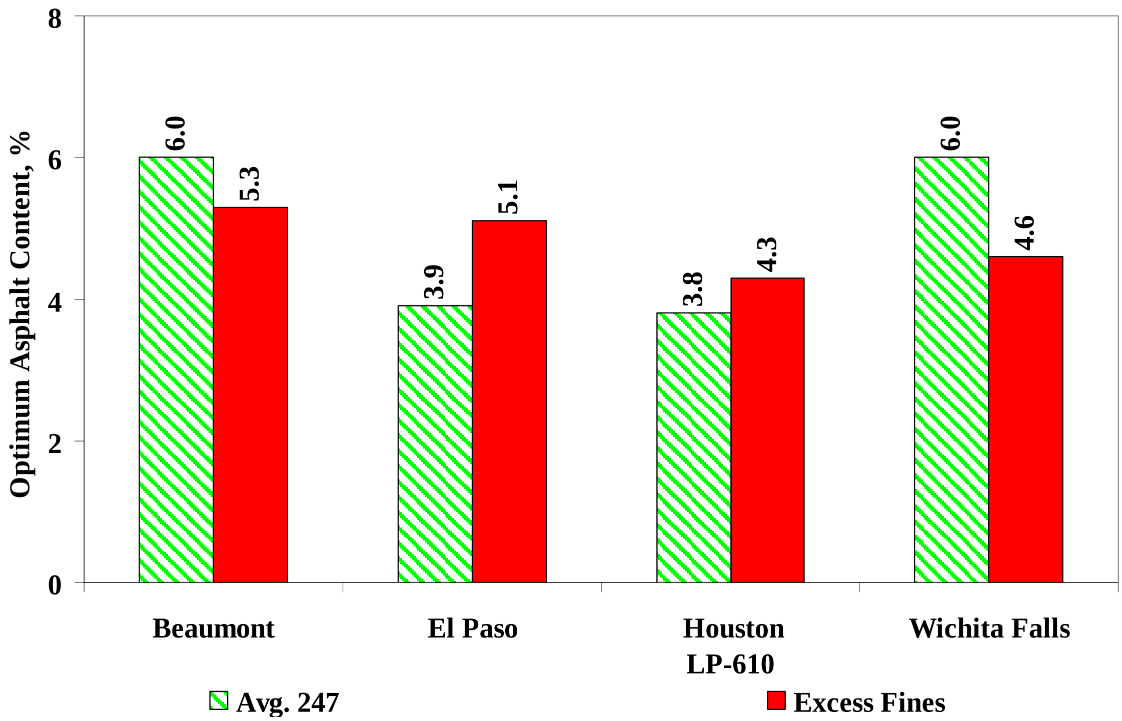


Figure 5.34: Impact of Fines Content Variation on OAC for Tex – 204 –H.

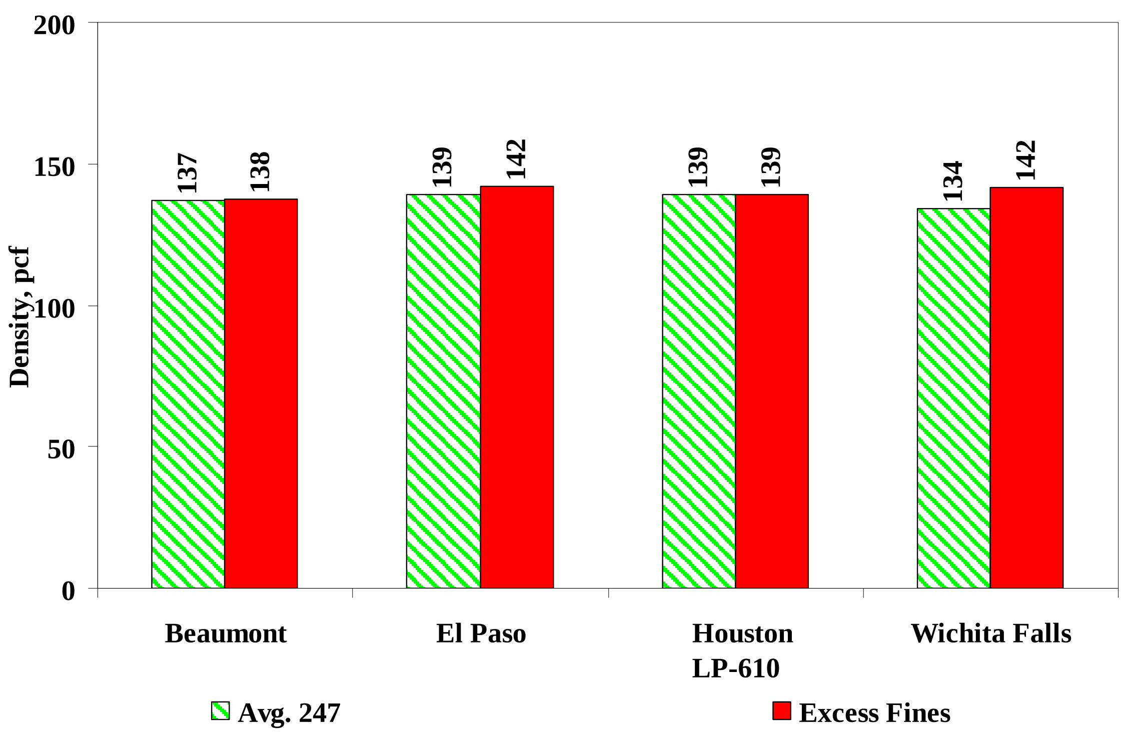


Figure 5.35: Impact of Fines Content Variation on Density for Tex – 204 –H.

As reflected in Figure 5.36, the IDT strengths from the finer gradations are typically similar or higher than those from the original, coarser gradations. In Figure 5.37 the moduli show somewhat different trend. As indicated before, the modulus is more an indication of the compressive behavior of a mix than tensile behavior.

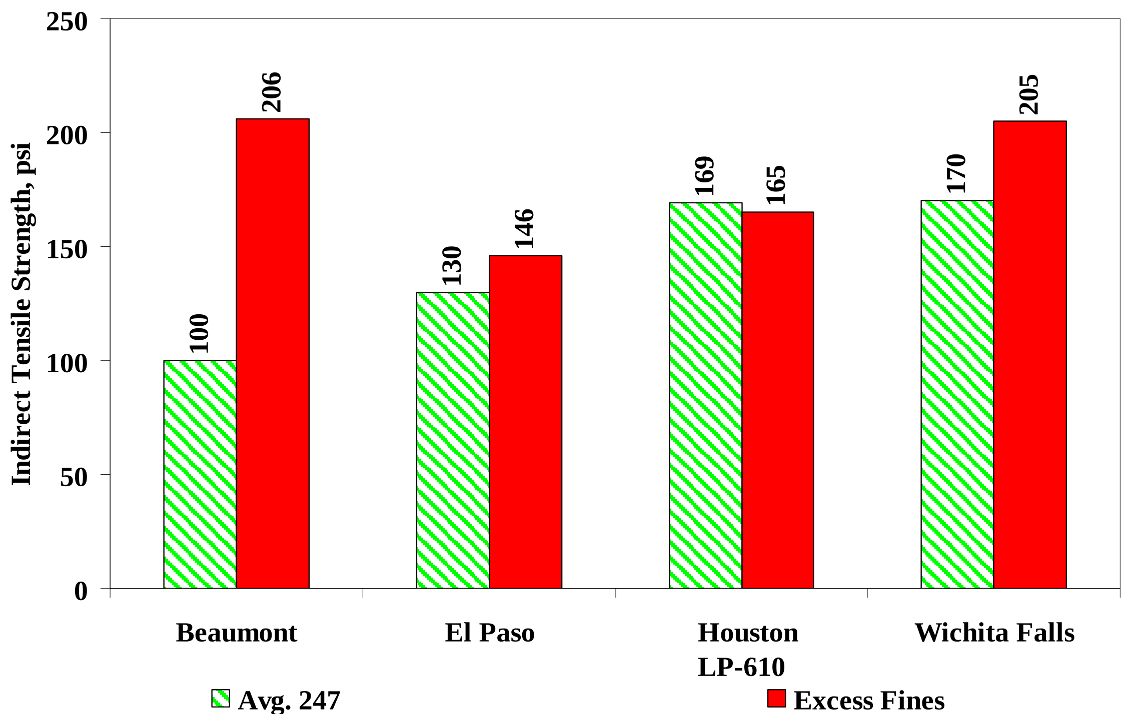


Figure 5.36: Impact of Fines Content Variation on IDT for Tex – 204 –H.

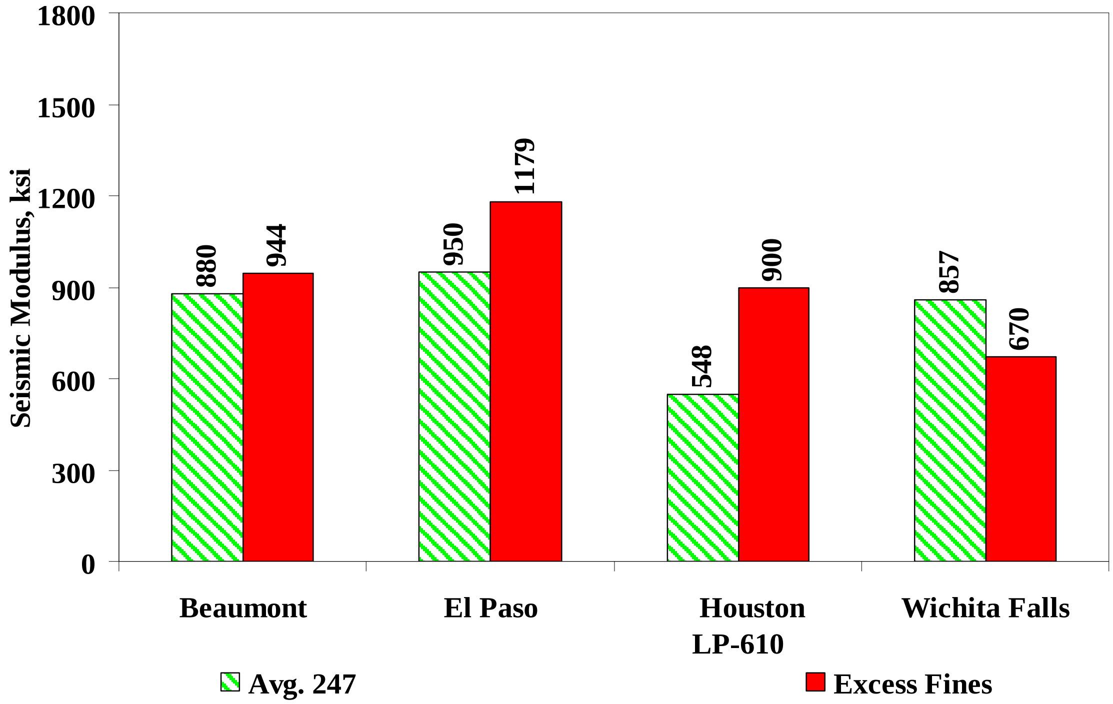


Figure 5.37: Impact of Fines Content Variation on Modulus for Tex – 204 –H.

Chapter Six: Proposed Mix Design Protocol

Along this chapter the proposed mix designed protocol is determined. Many factors were taken into account to select one of the two proposed methods. Since ATB propose to save money by reducing the amount of binder, decreasing that amount is primordial in this project, so the sensitivity of the parameters (density, strength, modulus) to asphalt content is an important factor to take into account. Other factors were as well considered for availability and ease in preparation such as: Compaction Device, Specimens Size, Strength Parameter, Curing and Testing Temperature.

As reflected in the previous chapters, two alternative mix design protocols were considered. The salient features of these two protocols are compared with those from the existing Tex-126-E and Tex-204-F in Table 6.1. Based on the evaluation process and considering many practical aspects, the protocol called Tex-204-H is recommended for the mix design of the ATB. The preliminary protocol for implementing Tex-204-H is included in Appendix A.

Table 6.1: Summary of Mix Design Methods Studied.

Mix Design Protocol	**Tex-126-E**	**Tex-204-F**	**Tex-126-H**	**Tex-204-H**
Compactor	TGC	SGC	SGC	SGC
Specimen Size, in.	6x8	6x4.5	6x8	6x4.5
OAC based on	Asphalt Density Curve	Volumetric Properties	Asphalt Density Curve	Asphalt Density Curve
Curing Duration and Temperature	48 hrs @140°F	48 hrs @77°F	24 hrs @140°F	24 hrs @ 77°F
Curing/Testing Temperatures, °F	140	77	77	77
Strength Test	UCS	IDT	UCS	IDT

6.1 BEST MIX DESIGN SELECTION

A large number of technical and operational factors were considered in this selection. These factors are discussed below.

- Compaction Device: Because of the availability of the Superpave Gyratory Compactors (SGC) in all districts and scarcity of the Texas Gyratory Compactors (TGC), the SGC was selected. This will save funds to refurbish or acquire TGC devices and minimizes the training of the district technicians.
- Specimen Size: Standard 6 in. x 4.5 in. specimens are recommended because of ease in preparation, requiring less material for mix design and more uniformity in specimens.
- Density Calculation: The density is calculated by dividing the weight of the molded specimen over its volume as done in Item 247, "Flexible Base".
- Number of Gyrations: Tentatively, a number of gyrations of 75 is proposed. As shown in Table 6.2, density is not very sensitive to the changes in the asphalt content for the ATB mixes, which may lead to uncertainty in determining the OAC. In Table 6.2, sensitivity is defined as the difference between the maximum and minimum densities at 3%, 4.5% and 6% asphalt contents (used in defining the asphalt content density relationships) for a given compaction protocol, divided by the average density of the three measurements. The greater this value is, the better the asphalt content density curve is defined. The highest sensitivity is obtained for the number of gyrations of about 80. Since the 75 gyrations are already included in Tex-204-E, that number was selected. Another angle in recommending the number of gyrations is the sensitivity of the asphalt content to strength and modulus. Table 6.2 shows the average sensitivity of these parameters for Tex-126-H and Tex-204-H protocols for the six different types of materials used in this study. Again, the highest sensitivities for strength and modulus are obtained at around 80 gyrations.

Table 6.2: MD Curve Sensitivity Results.

Gyrations	Density		Strength		Modulus	
	UCS	IDT	UCS	IDT	UCS	IDT
60 Gyrations	2%	4%	43%	61%	50%	33%
80 Gyrations	4%	5%	64%	61%	75%	47%
100 Gyrations	2%	3%	48%	45%	55%	22%
TGC	3%	NF	68%	NF	80%	NF*

*** NF – Not Feasible to Compact**

- Strength Parameter: Indirect tensile strength tests (IDT) was selected as the surrogate strength test because of the size of the specimen prepared and sensitivity of IDT to asphalt content.

- Curing and Testing temperature. Curing temperature of 77oF for 24 hrs and testing at 77oF are recommended. This will reduce the mix design time and eliminates the need for additional equipment for high-temperature curing of specimens (see Chapter 5 for more detail).
- Strength Requirement: A minimum strength of 85 psi as per TxDOT Item 344 is recommended.
- Moisture Susceptibility: Even though there are some concerns about the moisture susceptibility of ATB. None of the specimens tested as part of this research exhibited any signs of stripping or moisture susceptibility. As such, such a requirement has not been added to the specification.

6.2 DETERMINATION OF OAC

In order to determine the OAC the following steps are recommended:

1. Develop IDT-density-asphalt content curves such as the one shown in Figure 6.1.
2. Estimate the maximum density.
3. Convert the density to relative density by dividing the measured densities by the maximum density as shown in Figure 6.2.
4. Determine the maximum and minimum asphalt contents where the relative densities are equal or above 97% relative density. An example of this is shown in Figure 6.3.
5. Determine the maximum and minimum asphalt contents where the IDT strengths are greater than 85 psi (see Figure 6.3).
6. Determine a range of feasible binder contents as shown in Figure 6.4. The range of binder content where the relative density is greater than 97% and IDT strength is greater than 85 psi is recommended. For example, in Figure 6.4, the range is from 3.3 to 5.4%.
7. Based on the experience of the district and the constructability of the mix, select the design asphalt content. A value between the traditional OAC and upper limit range of feasible asphalt contents is recommended (e.g., between 4.4% and 5.4% in Figure 6.4).

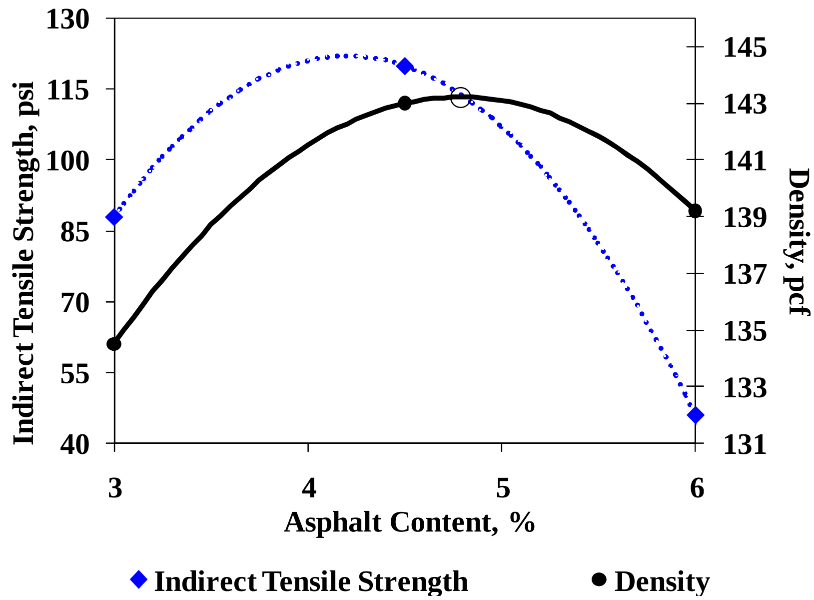


Figure 6.1: Density/IDT Asphalt Content Combined Curves.

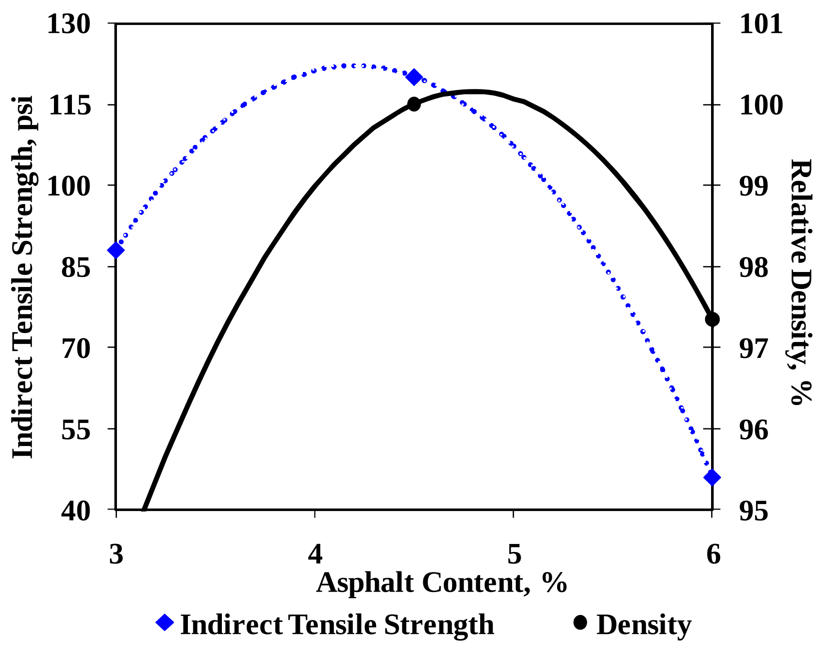


Figure 6.2: Relative Density/IDT Asphalt Content Combined Curves.

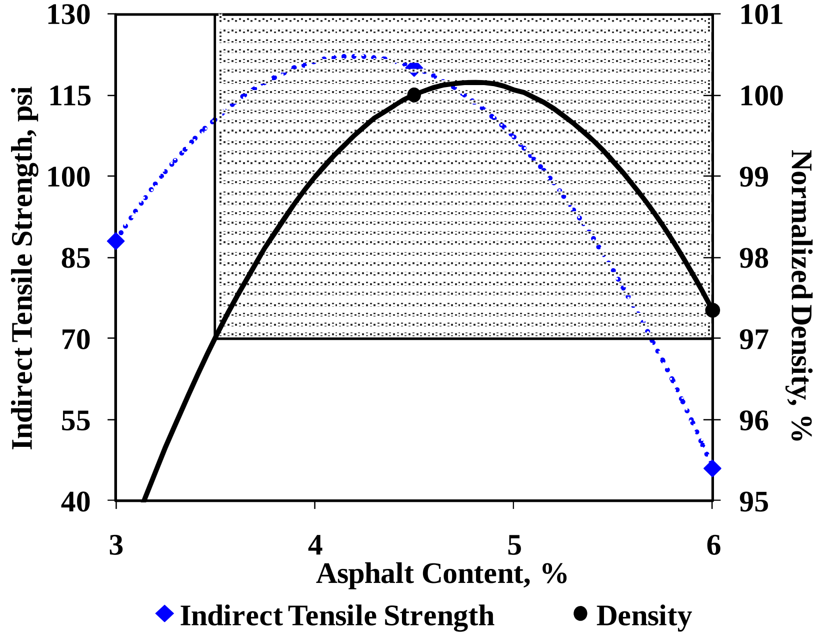


Figure 6.3: Relative Density/IDT Asphalt Content Combined Curves showing Allowable Asphalt Content Zone based on Density.

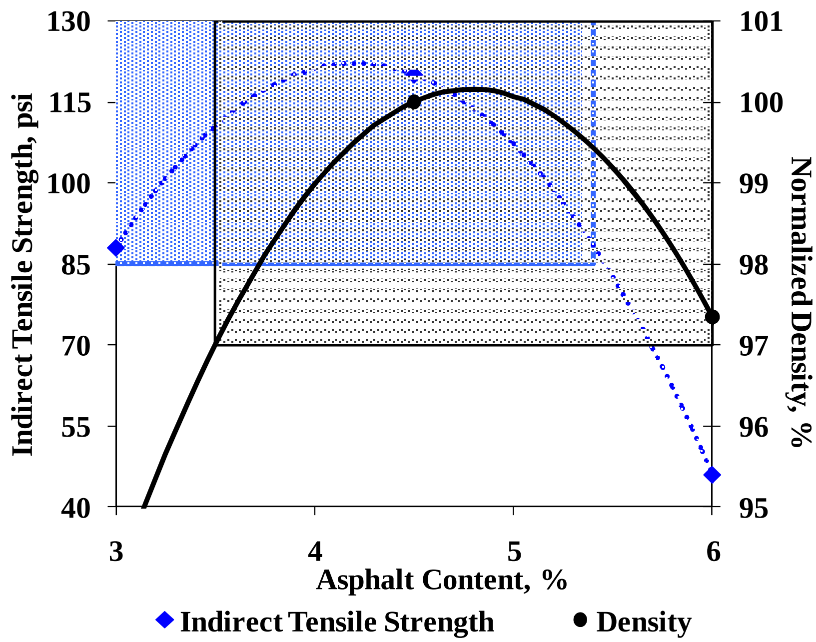


Figure 6.4: Relative Density/IDT Asphalt Content Combined Curves showing Allowable Asphalt Content Zones based on both Density and IDT.

Chapter Seven: Field Performance Monitoring

In this chapter the information obtained from field investigation conducted at several sites in the El Paso, Houston and Beaumont Districts is presented. Figure 7.1 shows the locations of the sites. Five sites were monitored before, during and after construction. These sites were located in Alpine in El Paso District, Beltway 8 and FM 526 (Federal Rd) in Houston District, and SH 943 and FM 2798 in Beaumont District.

The pavement profile of each site is shown in Figure 7.2. The ATB layers varied in thickness between 8 in. and 10 in. All sites were covered with surface treatments, expect for Alpine that the ATB was covered with a concrete slab, and Houston Beltway 8 that the ATB was not covered at all.

Raw materials were collected from the plants for laboratory tests. During construction, plant-mixed materials were also collected and taken to the laboratory for index and strength tests. A Portable Seismic Property Analyzer (PSPA) was used about 24 hrs and 6 months after field compaction at selected stations. Falling Weight Deflectometer (FWD) tests were performed 6 months after construction also at the selected stations. Cores were extracted from most of the sites at selected stations. With the material and information gathered at every site, the laboratory and field properties were compared. The results and conclusions of this activity are presented in this chapter.

7.1 LABORATORY RESULTS

7.1.1 Raw Materials

The gradation curves of raw materials collected at the quarries are shown in Figure7.3. Since the same quarry materials were used for both projects in Houston and Beaumont, only one mix design was carried out for each district. To prepare for mix design from each source, the entire stock was sieved first to develop a global gradation curve. That gradation curve was used throughout the study. The acceptable range as per Item 292 Grade 1 is also shown in the figure. All sites gradation curves met the requirements of Item 292.

As shown in Table 7.1, the Alpine materials had a Plasticity Index (PI) of 4 while Beaumont and Houston materials were non-plastic. The sand equivalency and aggregate hardness for each material are also presented in Table 7.1. The Houston and Beaumont materials met the sand equivalency requirements while the Alpine material did not. The Wet Ball Mill test results are within acceptable range for all sites.

Chapter 6 contains a detailed explanation of the proposed protocol for the determination of optimum asphalt content (OAC). Three sets of 6 in. x 4.5 in. specimens were prepared with 75 gyrations in a Superpave Gyratory Compactor (SGC) at nominal asphalt contents of 3%, 4.5% and 6% and tested as per that protocol. Figures 7.4 through 7.6 show the test results for the Alpine, Beaumont and Houston materials, respectively. Since the variations in the density with asphalt content are rather small and the IDT strengths in all cases are greater than 85 psi, OAC's between 3% and 6% can be used. Based on the current TxDOT practices, an OAC of about 4.5% seems reasonable for all materials. Moduli for each site at different asphalt contents (AC) are shown in Table 7.2. The greatest moduli were recorded for the Houston material and the smallest for the Beaumont material. The Alpine material with 3% AC exhibited a very low modulus because of difficulties in obtaining stable specimens perhaps because of the lack of fines.

Table 7.1: Sand Equivalency, Wet Ball Mill and Hardness for Materials Used.

Parameter	Alpine	Beaumont	Houston	Specification Requirement
Plasticity Index	4	Non Plastic	Non Plastic	10 max
Sand Equivalency, %	34	86	63	40 min
Wet Ball Mill, %	27	31	26	50% max

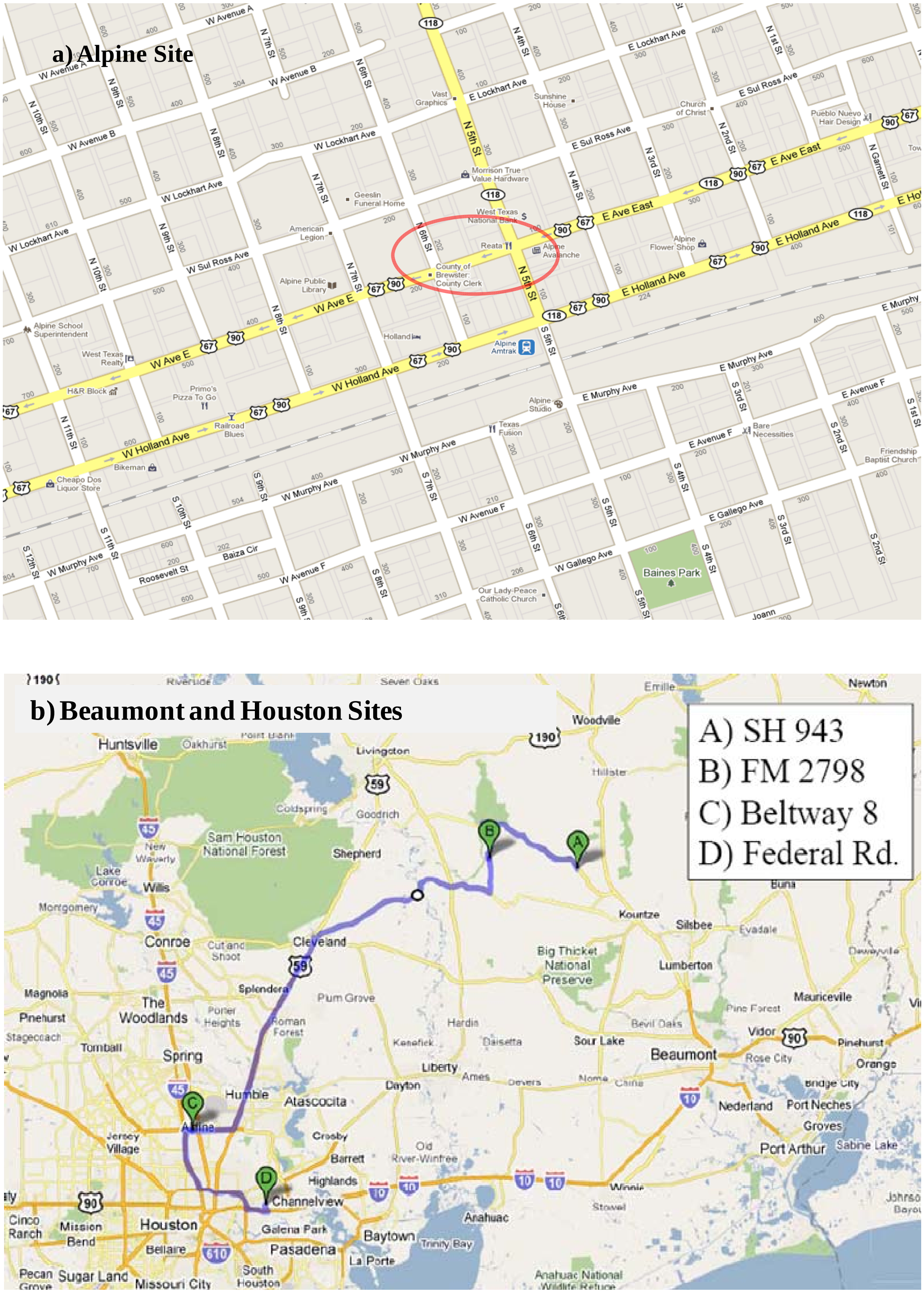


Figure 7.1: Locations of Sites Visited used in this Study.

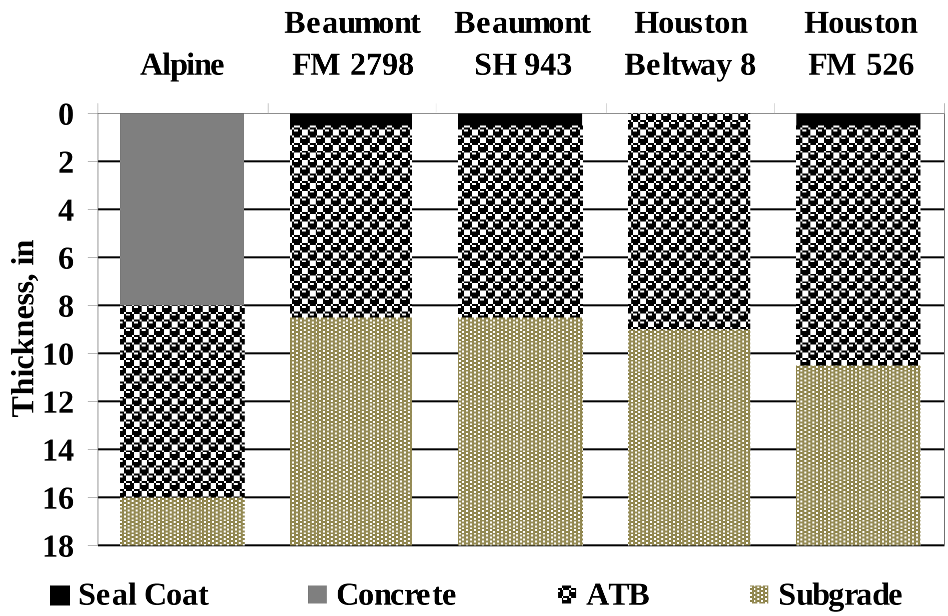


Figure 7.2: Pavement Profiles for Sites Visited in this Study.

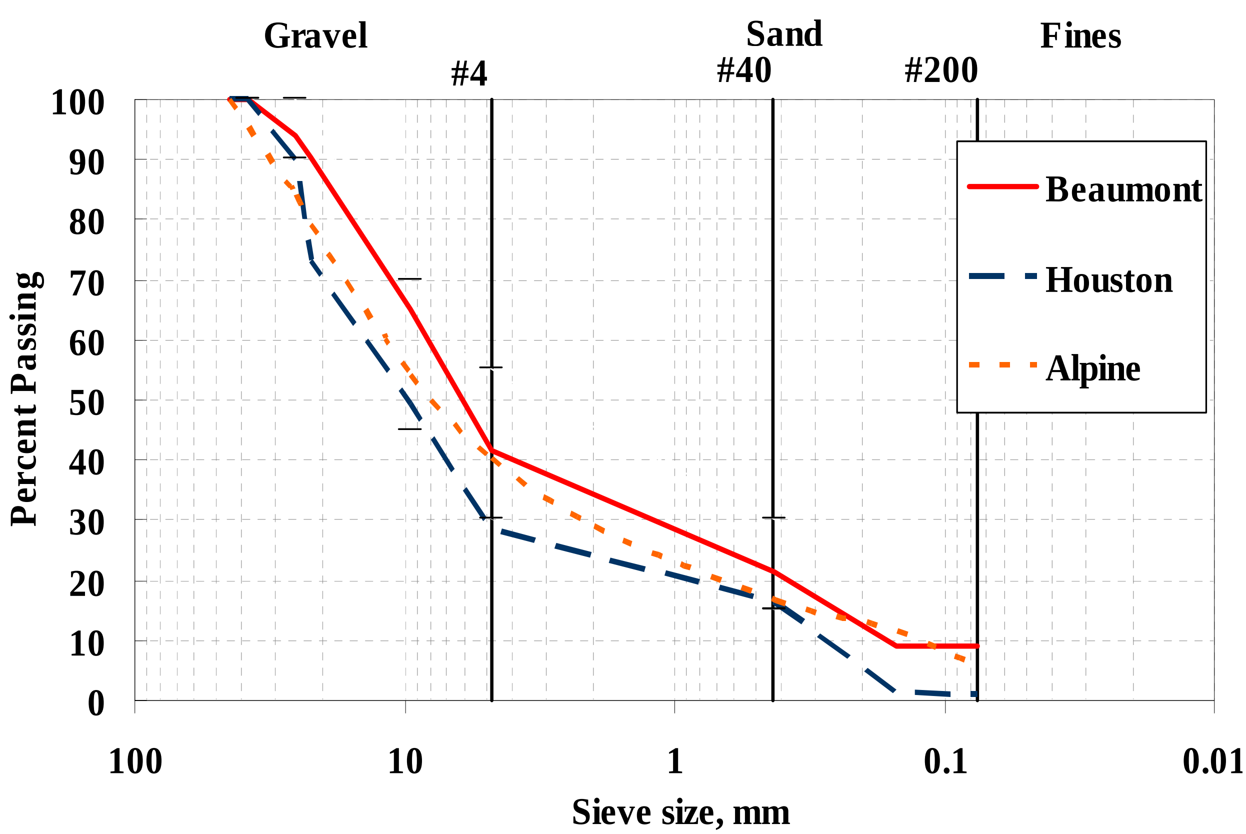


Figure 7.3: Laboratory Gradation Curves for Alpine, Beaumont and Houston Materials.

Table 7.2: Variations in Modulus with Asphalt Content for Alpine, Beaumont and Houston Materials.

Sites	**Alpine**	**Beaumont**	**Houston**
AC, %	**Modulus, ksi**		
3.0	399	964	1062
4.5	1103	852	1215
6.0	1203	814	1286

7.2 Plant-Mixed Materials

The binder content of plant-mixed material at each particular site and station was obtained using an ignition oven. The averages and coefficients of variation (COV) of binder content are reported in Table 7.3. The Alpine material had the highest average asphalt content (5.5%). For Houston and Beaumont materials, the average binder contents were close to 4.5%. The Beaumont SH 943 site exhibited an average AC content of 4.8% with the lowest COV (3.2%), while the Beaumont FM 2798 site had the lowest AC content with a COV of 5.9%. The design OAC of each site as provided by the District is also included in Table 7.3 as well. The asphalt content should not vary by more than 0.5% from the design target. All sites met this requirement except for the Beaumont SH 943.

Table 7.3: Asphalt Contents of Plant-Mixed Materials.

Site	**Binder Content**		
	Design, %	**Average, %**	**COV, %**
Alpine	6.0	5.5	4.3
Beaumont FM 2798	4.1	4.3	5.9
Beaumont SH 943	4.1	4.8	3.2
Houston Beltway 8	4.5	4.6	6.2
Houston FM-526	4.5	4.6	9.2

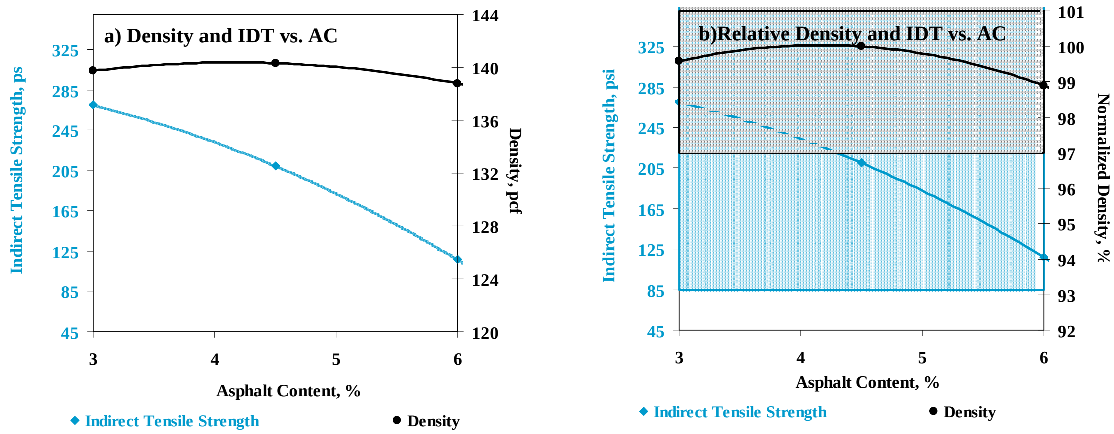


Figure 7.1: Density/ Strength vs. Asphalt Content for Alpine Material.

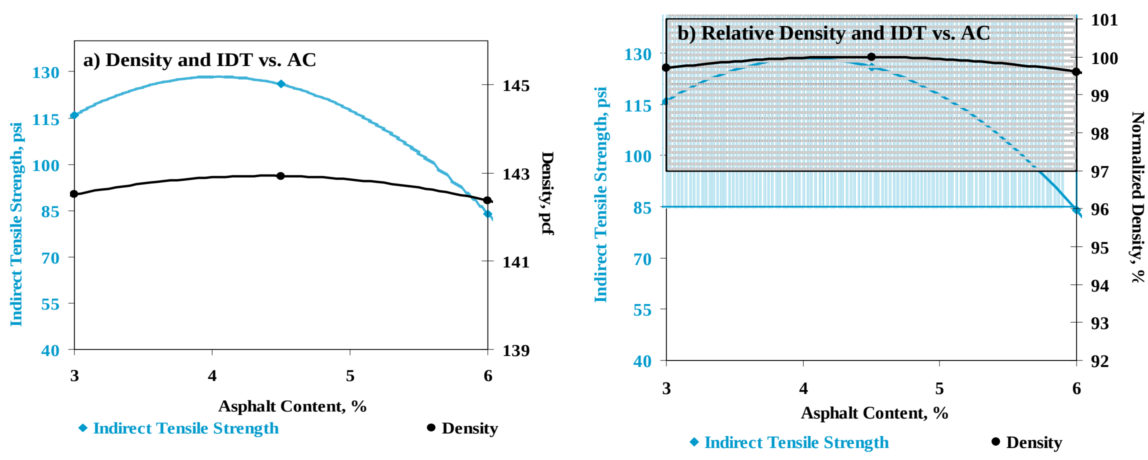


Figure 7.2: Density/ Strength vs. Asphalt Content for Houston Material.

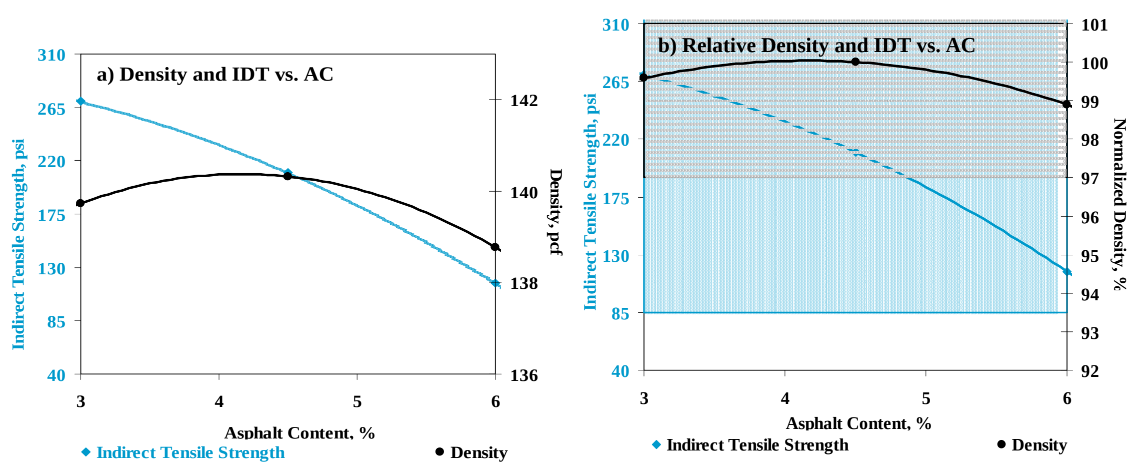


Figure 7.3: Density/ Strength vs. Asphalt Content for Beaumont Material.

Average gradation curves for materials subjected to the ignition oven are presented in Figure 7.7. Both Alpine gradations (original gradation and the gradation obtained from the plant mixed materials) met the Item 292 requirements. Even though the Beaumont SH 943 and Houston FM 526 material gradations obtained from the plant mixed material did not exactly follow the original design gradation, they still met the Item 292 requirements. The Houston Beltway 8 and Beaumont FM 2798 plant-mixed materials contained higher sand contents as compared to their respective original design gradations which mainly consisted of gravel. These two plant-mixed materials do not meet Item 292 gradation requirements.

Bulk and theoretical maximum specific gravities (Gmb and Gmm, respectively) were obtained in accordance with Tex-227-F and Tex-201-F for each plant-mixed material. The average and COV of each parameter, performed on ten specimens for Gmm and six specimens for Gmb, are shown in Table 7.4. The variability in the specific gravities as judged from COVs is rather small, and the air voids are all less than 4%.

Three plant-mixed specimens were compacted to nominal dimensions of 6 in. x 4.5 in. using 75 gyrations of the SGC. These specimens were tested in the same fashion as those prepared for the proposed mix design (cured at 77°F for 24 hrs). The properties of these plant mixed specimens are shown in Table 7.5.

Table 7.4: Bulk and Maximum Theoretical Specific Gravities for Plant-Mixed Materials Molded at 75 Gyrations.

Site	Gmm		Gmb		Average Air Voids
	Average	COV, %	Average	COV, %	
Alpine	2.432	0.1	2.427	0.5	1.0%
Beaumont FM 2798	2.441	1.3	2.353	0.3	3.6%
Beaumont SH 943	2.465	0.8	2.377	0.3	3.6%
Houston Beltway 8	2.436	1.3	2.365	0.2	2.9%
Houston FM 526	2.477	1.1	2.404	0.3	3.0%

In general, the densities from the five materials are similar with Alpine material being the densest. The Alpine material exhibited the lowest IDT strength and the highest modulus as expected from a material with low fines content. Even though, the Beaumont SH 943 and Houston Beltway 8 specimens exhibited the same density (142 pcf), the Beaumont specimens showed higher IDT and modulus values as compared to the Houston materials. This may be explained by the differences in gradations and sand equivalencies.

Table 7.5: Properties of Plant-Mixed Molded using 75 Gyrations of SGC.

Site	**Density**		**IDT**		**Seismic Modulus**	
	Average, pcf	**COV, %**	**Average, psi**	**COV, %**	**Average, ksi**	**COV, %**
Alpine	147	0.7	150	14.1	1189	1.1
Beaumont FM 2798	140	0.4	214	6.1	886	3.4
Beaumont SH 943	142	0.3	305	2.8	955	3.1
Houston Beltway 8	142	0.4	224	8.7	905	1.1
Houston FM 526	143	0.6	315	4.5	918	4.5

7.3 FIELD CORES

Six cores were extracted from the Alpine site, both Beaumont sites and Houston Beltway 8 site about 24 hrs after compaction. Coring at the Houston FM 526 was not permitted due to the criticality of the project. The average delivery and compaction temperatures of the mixes at the approximate locations of the cores are summarized in Table 7.6. The temperatures at delivery and compaction for the Alpine site were not recorded. According to Item 292 the delivery temperatures must not exceed 350°F and compaction must be completed before temperature drops below 175°F. In this case, all the temperatures readings met those requirements.

The densities based on the weight and dimensions and air voids based on the Gmb value of every core were determined in the laboratory as summarized in Table 7.7. Comparing the

results in Tables 7.4 and 7.5 with Table 7.6, the cores' air voids are significantly higher and their densities are significantly lower than those of the plant-mixed materials molded by the SGC at 75 gyrations.

Table 7.6: Delivery and Compaction Temperatures of Asphalt Material at the Sites.

Material		**Temperature**	
		Average, °F	**COV, %**
Beaumont FM 2798	**Delivery**	289	3.5
	Compaction	228	3.0
Beaumont SH 943	**Delivery**	300	5.0
	Compaction	217	7.4
Houston Beltway 8	**Delivery**	273	6.6
	Compaction	209	2.7
Houston FM 526	**Delivery**	220	1.4
	Compaction	202	2.1

Table 7.7: Volumetric Properties of Field Cores.

Sites	**Density, pcf**		**Gmb**		**Air Voids, %***
	Average	**COV, %**	**Average**	**COV, %**	
Alpine	145	3.7	2.377	2.3	2.3
Beaumont FM 2798	129	3.9	2.143	4.0	12.2
Beaumont SH 943	124	3.7	2.094	5.2	15.1
Houston Beltway 8	136	4.3	2.288	2.3	6.1

*** G_{mm}'s in Table 7.4 was used to calculate air voids**

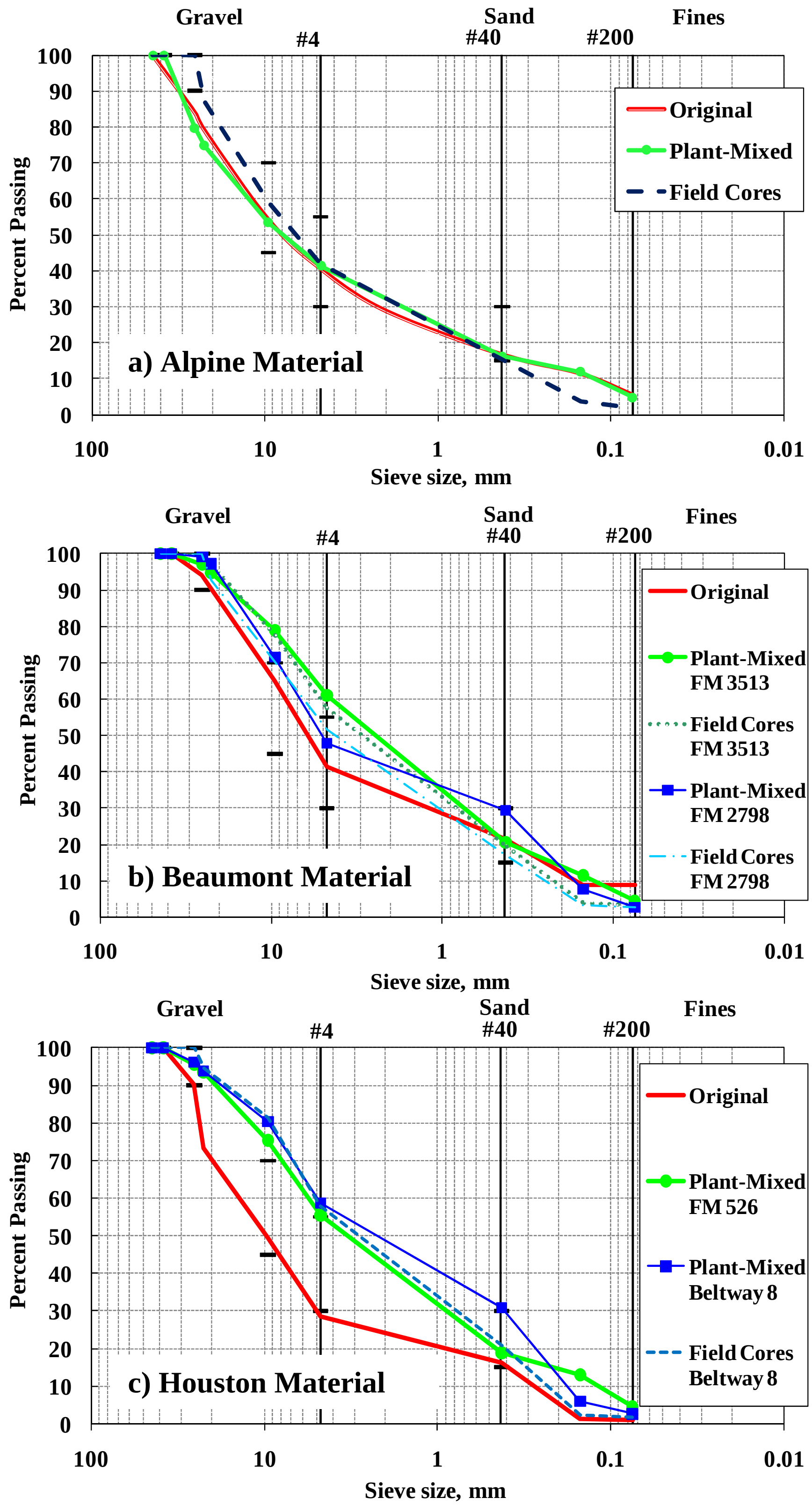


Figure 7.7: Gradation Curves from Plant-Mixed Materials as Delivered.

The average modulus and IDT strength of five random cores at each site are shown in Table 7.8. Alpine and Houston Beltway 8 field cores showed higher IDT strengths and moduli. Alpine field cores were found to be more uniform since the COVs for IDT and modulus are low.

Table 7.8: Strength and Modulus obtained from Field Cores.

Site	IDT Strength		Modulus	
	Average, psi	COV, %	Average, ksi	COV, %
Alpine	145	16.6	891	8.8
Beaumont FM 2798	91	58.0	463	35.9
Beaumont SH 943	86	13.7	380	20.3
Houston Beltway 8	159	29.4	770	18.8

7.4 FIELD RESULTS

The PSPA moduli obtained after 24 hrs and approximately 6 months after construction of the ATB at each site are shown in Figure 7.8. The PSPA tests could not be performed six months after construction at Alpine because it was covered with a concrete slab. PSPA moduli increased in the 6-month period for all sites tested except for Houston Beltway 8 where the ATB was exposed to the environmental elements and was not trafficked at all. The backcalculated FWD moduli for all sites are also shown in Figure 7.8. The Alpine FWD results are suspect because deciphering the moduli of the ATB under a concrete layer is rather difficult.

7.4.1 Comparison of Laboratory and Field Results

The air voids from the plant-mixed materials molded at 75 gyrations using the SGC are compared with the air voids of the field cores in Figure 7.9. The air voids of the field cores are considerably higher for all sites as compared to the lab-molded specimens. Similarly, the densities for the plant-mixed material molded at 75 gyrations are compared to the densities from

the field cores in Figure 7.10. Again, substantial differences are observed between the lab and field densities.

A study was carried out to observe the number of gyrations required to match the field densities in the lab. It was impossible to simulate the Beaumont SH 943 field density in the lab. The lowest density that could be obtained by the SGC was 128 pcf by just applying the weight of the ram. Beaumont FM 2798 and Houston Beltway 8 materials reached their corresponding field densities in the SGC after 3 and 15 gyrations, respectively. About 66 gyrations were needed to reach the average field density for the Alpine material.

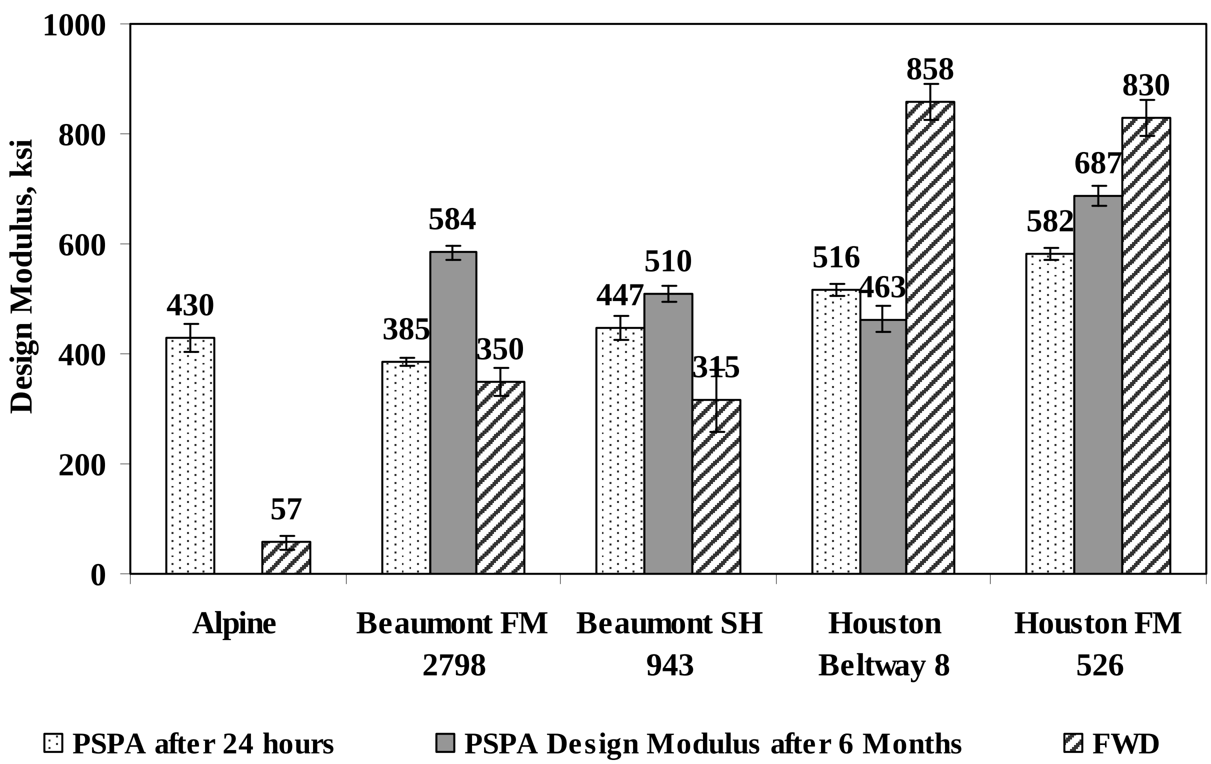


Figure 7.8: PSPA and FWD from ATB Sites.

As shown in Figure 7.11, the IDTs of specimens molded at 75 gyrations are naturally higher at all sites as compared to those from cores extracted from the field. In addition, the IDTs of the lab specimens compacted to field densities are also marginally to significantly higher than

the cores. The same trend is also observed for the seismic moduli of the specimens as shown in Figure 7.12.

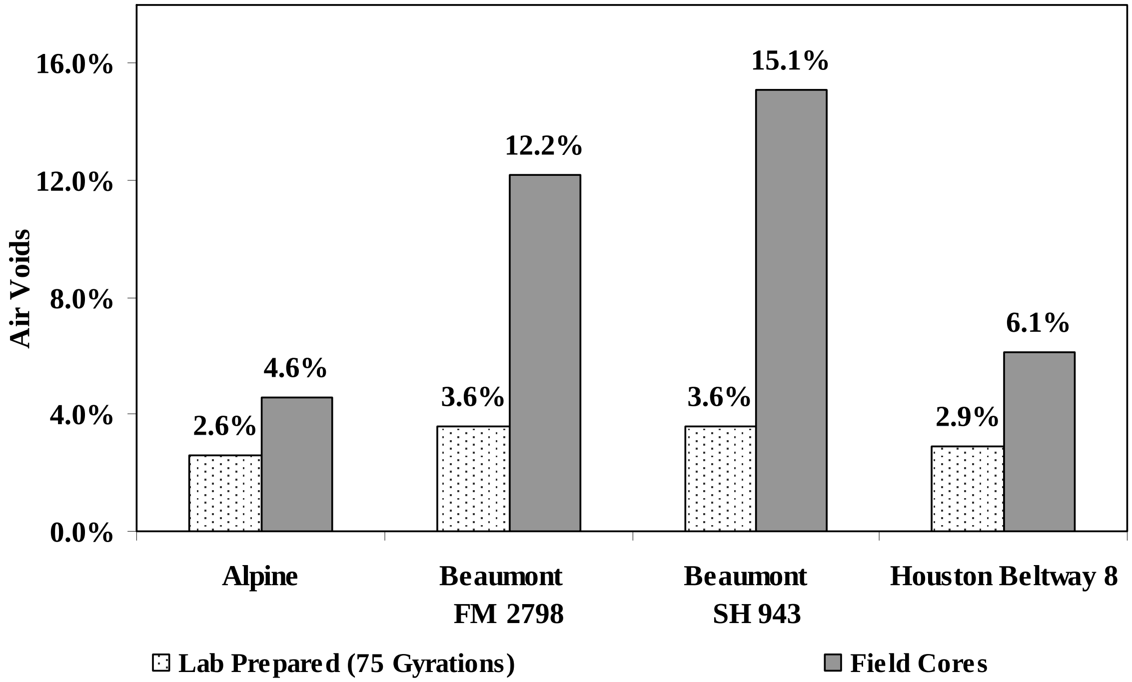


Figure 7.9: Comparison between Air Voids of Laboratory and Field Specimens.

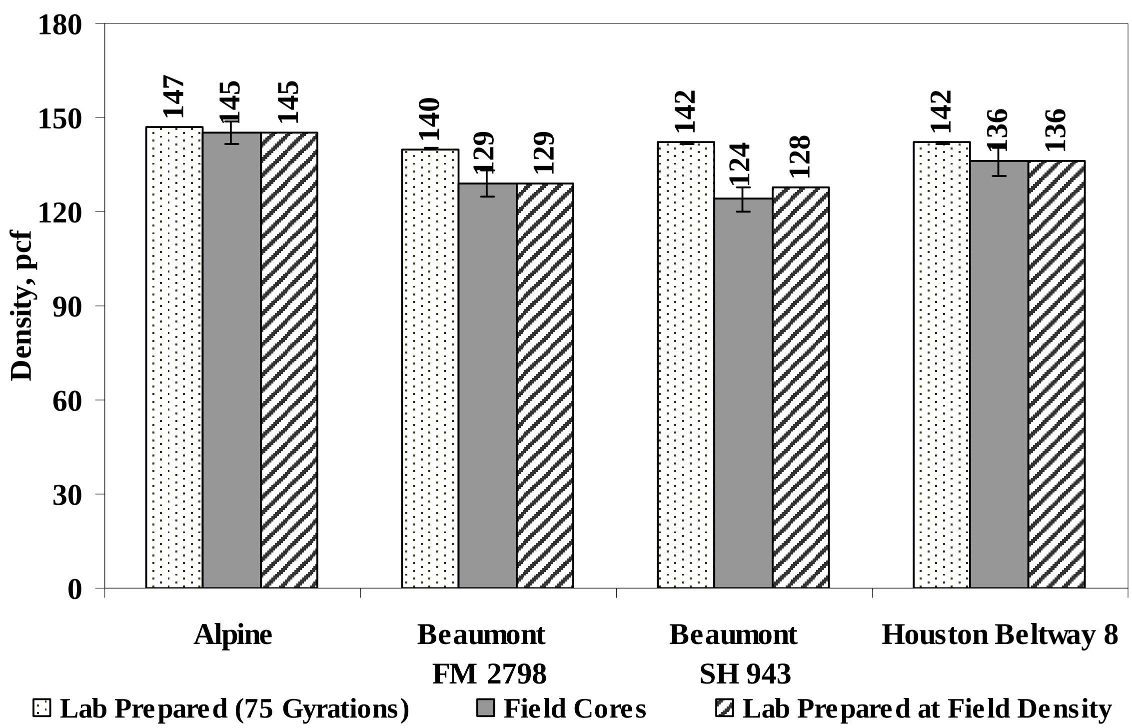


Figure 7.10: Comparison between Densities of Laboratory and Field Specimens.

In Figure 7.12, the moduli obtained from the cores, and PSPA are also included for comparison. Specimens molded using the 75 gyrations of SGC showed the highest moduli. The moduli acquired using the V-meter device from the field cores is comparable to the PSPA modulus 6 months after construction for all. V-meter and PSPA modulus compared in Figure 7.12 do not follow the same pattern because the PSPA modulus shown in this particular figure is the average of all PSPA modulus readings obtained along the roads.

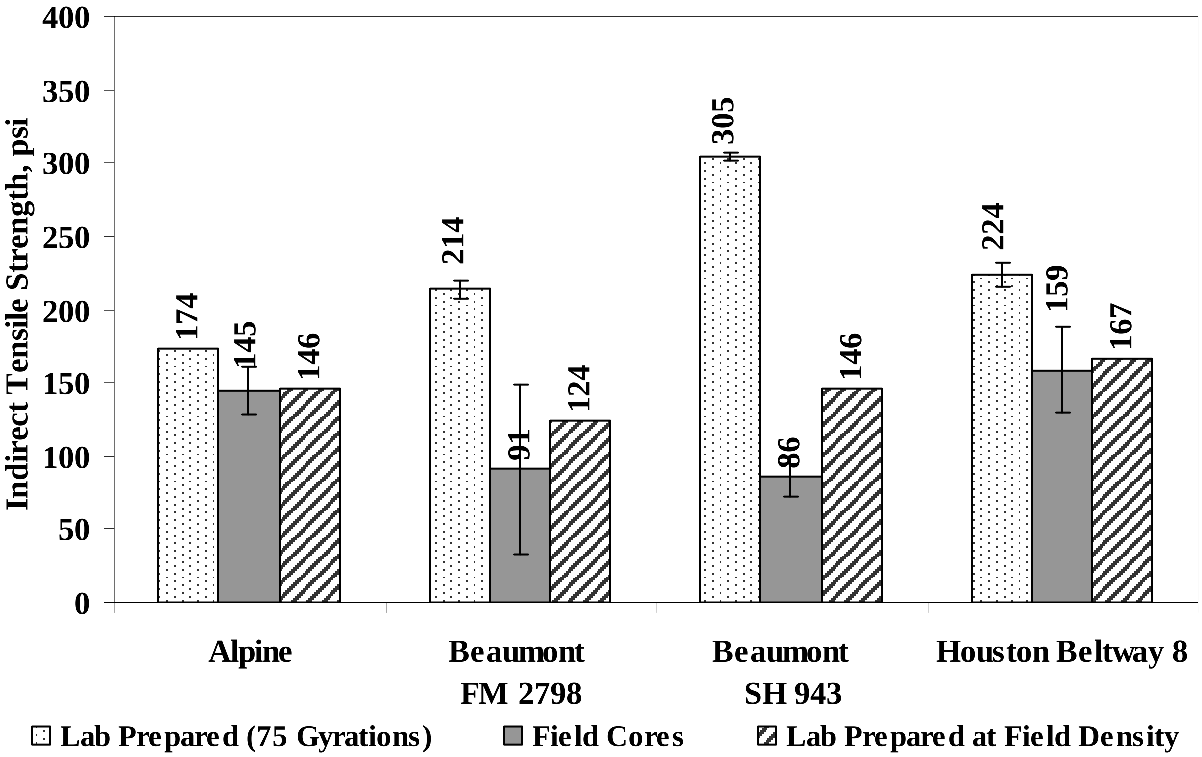


Figure 7.11: IDT Comparison between Laboratory and Field Specimens.

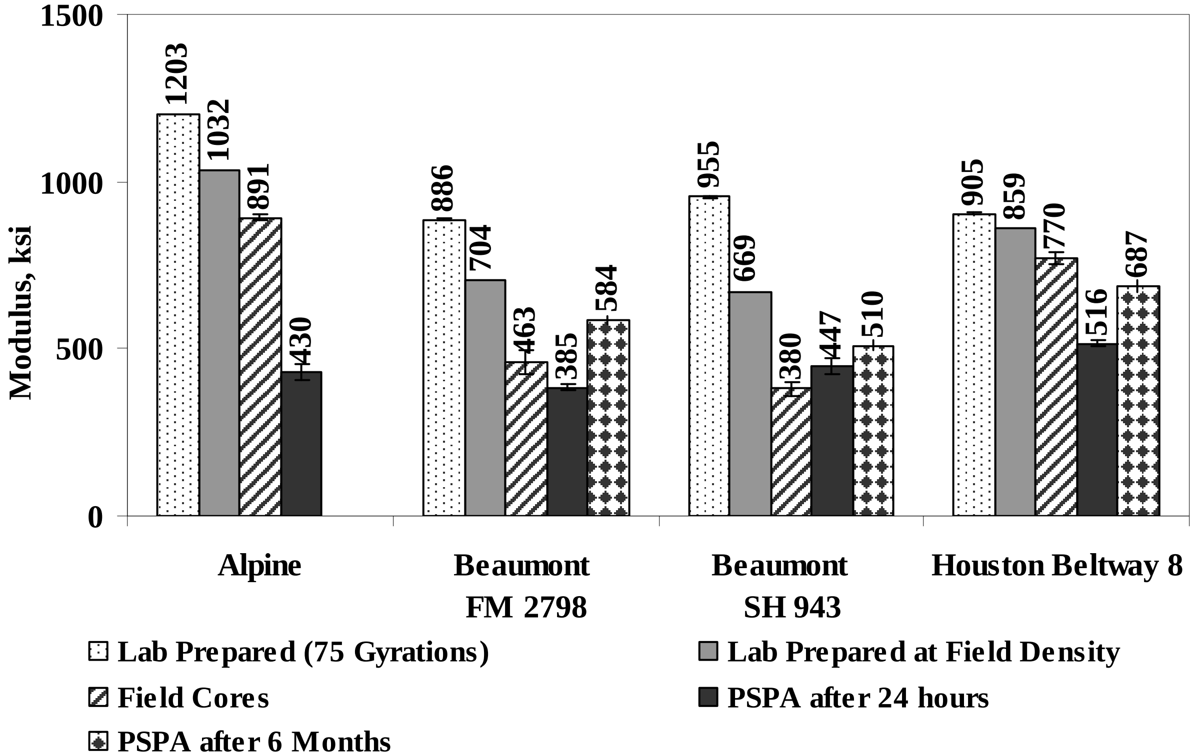


Figure 7.12: Modulus Comparison between Laboratory and Field Specimens.

Chapter Eight: Conclusions and Recommendations

Based on the overall results of this study, the following conclusions can be outlined:

- The Superpave Gyratory Compactor results were found to be more uniform and consistent compared to the Texas Gyratory Compactor. Regarding operational factors, SGC resulted to be easier and more efficient compared to the TGC.
- When determining the OAC of an ATB material following the method explained in this report using the SGC resulted to be lower than the OAC obtained from the TGC.
- The asphalt content variations caused no significant changes in density of molded specimens using the SGC.
- Specimens molded at 80 gyrations were found to have the highest sensitivity to asphalt content compared to the specimens molded at 60 and 100 gyrations. Since the 75 gyrations are already included in Tex-204-E; 75 was selected as the number of gyrations.
- Superpave Gyratory Compactor specimens usually exhibit similar or higher UCS as compared to those prepared with Texas Gyratory Compactor.
- Superpave Gyratory Compactor specimens usually exhibit similar or higher IDT as compared to those prepared with Texas Gyratory Compactor
- Specimen Standard Size 6 in. in diameter and 4.5 in. in height were selected to be the more efficient because of the ease in preparation; they require less material for mix design and resulted to have a higher uniformity.
- Indirect Tensile Strength was found to be more sensitive to the asphalt content, so it was selected to be as a parameter to select the OAC of a mix design. The OAC was selected to be any value in a standard Asphalt-Content Curve which meets the following two requisites:
    - 97% of the relative density

- 85 psi of IDT as per TxDOT Item 344 is recommended

Curing temperature of 77°F during 24 hours after compaction is recommended. Testing temperature of 77°F is also recommended.

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Curriculum Vita

Hector A. Hernandez was born on November 30, 1985 in Chihuahua, Chihuahua, Mexico, the second son of Humberto A. Hernandez and Guillermina Sandoval Ortiz. He graduated from Escuela Federal por Cooperacion in the spring of 2004 and entered University of Texas at El Paso (UTEP) in the fall of the same year. He received his bachelor's degree in Civil Engineering from University of Texas at El Paso in 2009. While pursuing his bachelor's degree he joined many honors societies such as: National Honors Society, Alpha Chi, and Chi Epsilon. He is an Engineer in Training by the Texas Board of Professional Engineers (2009). In the spring of 2010, he entered the graduate program of the Civil Engineering Department at UTEP. He received the SemMaterial Scholarship and the Eisenhower Fellowship in the spring of 2009 and 2010, respectively for his academic achievements. He graduated with a Grade Point Average of 3.7 and 3.8 during his undergraduate and graduate studies, respectively. Upon graduation, he was prized with the Andrew Jones Award for his performance during his undergraduate studies.

During his undergraduate studies he joined the Center for Transportation Infrastructure Systems (CTIS) at UTEP as an Undergraduate Research Assistant. After completion of his undergraduate studies, he became Graduate Research Assistant. He started working on a solar pavement markers project and then continued working in project named: "Development of a New Mix Design Method and Specification Requirements for Asphalt Treated Base", funded by the Texas Department of Transportation. He worked at CTIS for approximately 3 years. He is currently a Geotechnical Project Engineer at the Construction Quality Control (CQC) company in El Paso, Texas.

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