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16. Abstract

The performance of flexible pavements relies heavily on the final quality of the hot-mix asphalt concrete (HMAC) as it is produced and placed in the field. To account for production and construction variability while ensuring the quality of the HMAC, TxDOT has established a set of relevant operational tolerances, which are incorporated into the 2004 Standard Specifications for Construction and Maintenance of Highways, Streets and Bridges. In particular, Items 340/341, 342, 344, and 346 provide acceptance criteria for all HMA mixes used by the Department. The operational tolerances for a series of key control variables that affect performance are given as a guideline in QC/QA practices. However, the relationship between these tolerance levels and mixture performance is not well known. This research project will establish this relationship: how do the operational tolerances affect the expected performance of the HMAC? Once this relationship is quantified, recommendations will be developed that indicate, if necessary, how the current tolerances should be modified.

The objective of the study was to determine the effects of variability in key mix design factors, such as asphalt content, gradation, and density, on the laboratory performance of different hot-mix asphalt samples that were mixed and compacted in the laboratory. Variability was kept within specified limits by the allowable operational tolerances, and performance was addressed through the evaluation of the results obtained from volumetric properties and laboratory tests, such as flexural fatigue test, Hamburg wheel-tracking test, and overlay tester. A series of statistical analyses were conducted to develop relationships between the key mix design factors and the observed laboratory performances of each type of mixture. From the analysis, the effects of the main variables on the results of the performance tests used in this study were found.

Finally, a statistically-based sensitivity analysis was conducted to reveal the relationship between different tolerance levels and mixture performance for the individual mixtures types. This research facilitates, for both TxDOT personnel and contractors, the evaluation of asphalt mixture performance under different tolerance levels, which will be performance-based and supported by a rigorous and sound statistical analysis.

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Laboratory Evaluation of Influence of Operational Tolerance (Acceptance Criterion) on Performance of Hot-Mix Asphalt Concrete

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1.1 Introduction

1.1.1 Background

The production of hot-mix asphalt (HMA) includes several stages, each of which introduces variability into the final composition of the job mix formula (JMF) and, consequently, into the volumetric properties and performance of a given mixture design.

This variability in the HMA is also due to the inherent variability of the components used in the HMA. This is the case of aggregates, which are obtained from a quarry where the stockpile gradation is variable due to the inherit randomness of the crushing process, but also due to segregation produced by an inadequate setup of the equipment, stockpile height and configuration, and loading process during construction. All these factors contribute to deliver an aggregate with a gradation slightly different from the one that was determined by sampling and used for design and, thus, affecting the physical properties of the mixture and its performance.

The variability can also come from the variability of the asphalt content. This is an important characteristic that influences the performance of pavements. An excess in asphalt binder content may result in the stability of the mixture being compromised, while a deficit in asphalt binder content results in a mixture that may not be durable and may be prone to cracking. Defects in the dosage equipment or intentional reduction in asphalt content (because of cost considerations), within acceptable operational tolerances permitted by specifications, may potentially lead to undesirable characteristics of the asphalt mixture.

It is important to identify the variables that influence changes in HMA performance and to have a good understanding of the magnitude of these effects on the outcomes of laboratory tests such as the flexural fatigue, Hamburg wheel-tracking, and overlay tests and, ultimately, on the performance of the mixture in the field.

1.1.2 Research Objectives

The objective of this research study is to determine the effects that changes in mixture properties, such as (1) asphalt content, (2) gradation, and (3) air-voids content, have on the laboratory performance of samples evaluated by mean of performance-related tests. To this effect, the following tests were considered: four-point bending beam fatigue test, Hamburg wheel-tracking device test, and Texas overlay tester.

1.1.3 Scope

The study is limited to the mixture design, production, and testing of five different asphalt mixes using one type of binder (PG 76-22S) and two different types of aggregate sources (limestone and gravel) while changing the aggregate gradation, the specimen density, and the asphalt content in order to account for the effects in performance. These volumetric properties are varied in a systematic way within specified tolerance limits as specified by the Texas Department of Transportation (TxDOT). The performance in the laboratory was evaluated using the beam fatigue test, Hamburg wheel-tracking device test, and overlay tester.

Out of the five asphalt mixtures produced, three were produced using limestone: two conventional dense-graded mixtures, Type C and Type D, and one Stone Matrix Asphalt mixture (SMA-D); and two were produced using gravel: conventional dense-graded, Type C and Type D. According to the classification system used by TxDOT, Type C has a maximum aggregate size of ¾ inch and Type D has a maximum aggregate size of ½ inch.

1.1.4 Outline

This report summarizes the research study and it is organized into six chapters. Chapter 1 presents an introduction to the study and sets its objectives and limitation. It also presents an outline of every chapter contained in this report. Chapter 2 offers a general review of hot-mix asphalt (HMA) technology. This chapter covers the definition, general classification and properties of HMA, and the properties of its constitutive materials: aggregates, binder, and modifiers. Chapter 3 presents a review of the performance testing methods used in this study. Chapter 4 offers a detailed explanation of the different mixtures produced for the current study and the methodology used to adjust them from a target mixture at optimum asphalt content. Chapter 5 compiles the obtained laboratory test results and analyses and discusses the observed trends of the data. Chapter 6 closes completes this report by providing conclusions and recommendations.

Chapter 2. Hot-Mix Asphalt (HMA)

The following chapter presents a general review of the definitions and the main properties of hot-mix asphalt (HMA) and its constitutive materials. It identifies the mixture properties that are relevant to HMA performance, and discusses their influence.

2.1 Definition

In principle, HMA consists of a controlled combination of coarse and fine aggregates, asphalt binder, and in some cases, additives. Because of the temperature dependency of the viscosity of the asphalt binder, the mixing has to be done at relatively high temperatures and, therefore, the term "hot mix" is used. The purpose and necessity of heating the materials to a specific temperature is to dry the aggregate and reduce the asphalt binder viscosity in order to achieve fluidity and effectively coat the aggregate particles with a reasonable mixing effort. When cooled to ambient temperatures, the HMA forms a relatively hard and flexible construction material. Depending on the grade of the asphalt binder, the mixing temperature for neat binders should be generally kept between 290 and 325^{o}F (143–163^{o}C) to allow for adequate workability when mixing and, at the same time, avoid overly aging and burning the binder, given that asphalt burns at a temperatures in the order of 442^{o}F (228^{o}C).

The most common types of HMA used in road pavement construction are (Muench et al., 2009):

1. **Dense-graded HMA** is a relatively impermeable well-graded (i.e., uniform distribution of all particle sizes) mixture intended for general purpose, for all pavement layers and all traffic conditions, and for structural reinforcement, friction improvement, and distress correction. They can be sub classified as fine-graded (coarse and fine leveling/surface course, and fine mixture) or coarse-graded (coarse and fine base course).

2. **Stone Matrix Asphalt (SMA)** is a relatively impermeable gap-graded HMA that is primarily designed to maximize rutting resistance and durability, but it also provides other benefits that include wet weather friction (due to a coarser surface texture), lower tire noise (due to a coarser surface texture), and less severe reflective cracking. The increase in rutting resistance is achieved by providing an intimate contact between aggregates and a stone-on-stone mixture. Under these conditions, stresses and deformations are carried by the aggregate skeleton rather than the asphalt binder, which reduces rutting due to the minimum deformation that aggregate presents. Consequently, SMA mixes are intended for surface courses on high volume interstates and roads. The only drawback is that SMA is, typically, 20 to 25% more expensive than regular dense-graded HMA because it requires more durable aggregates, higher asphalt contents, and modified asphalt binder as well as mineral or cellulose fibers to prevent excess asphalt in the mix that may cause drain down of the asphalt binder. The increase in cost is often justified by an improved durability and a reduction in maintenance costs.

3. **Open-graded HMA** is a mixture designed to be water permeable that uses only crushed stone or gravel, and a small percentage of manufactured sands. The three

types of open-graded mixes commonly used in road construction are (Muench et al., 2009):

- Porous European mixes (PEM): typically present 18-22% air-void content, specified minimum air voids, higher aggregate quality standards than OGFC (see below), and require the use of asphalt binder modifiers.
- Open-graded friction course (OGFC): typically present 15% air-void content, no minimum air voids specified, and lower aggregate quality standards than PEM.
- Asphalt treated permeable bases (ATPB): the specifications used to produce ATPB are less rigorous than OGFC or PEM because it is used as an intermediate layer, under dense-graded HMA, SMA, or Portland Cement Concrete (PCC) for drainage purposes.

PEM and OGFC are used exclusively as surface courses and, due to the high air-void content, they can act as water and air reservoirs. Hence, they are intended to reduce tire splash in wet weather conditions and to reduce tire-road noise. The disadvantage of such mixtures is that they are generally more expensive per ton of material.

According to the scope of the current study, only dense-graded HMA (TxDOT Item 340/341) and SMA (TxDOT Item 346) were analyzed to determine the differences in terms of laboratory performance.

2.1.1 Materials

As defined previously, HMA is a combination of aggregates, asphalt binder, and in some cases, additives. After compaction, it can be considered a three-phase composite with aggregates, binder, and air. Consequently, it is expected that the overall performance of HMA will depend on the performance of each individual component, as well as on the interaction between them. Therefore, the following paragraphs discuss the properties of aggregates, asphalt binder, and additives, and how their particular properties and variations found in these properties are related to the performance of the HMA in the laboratory and in the field.

Aggregates

HMA is the result of the combination of different components that contribute individually to the overall quality of the final product. Aggregates constitute the largest part of the composite system, whose quality characteristics start being shaped in the quarry, continue in the aggregate plant, then the asphalt plant, and end finally in the field during construction. All these steps influence aggregate quality, and therefore, pavement performance. The relationship between aggregate quality and HMA performance can be evaluated depending upon physical and chemical characteristics of the aggregate (Dukatz, 1989).

<u>Physical Properties of the Aggregate</u>

a) Shape and angularity

Aggregate shape affects fatigue and low temperature cracking resistance of HMA, and aggregate angularity generally increases the stiffness and the fatigue resistance of the mix. Cubical or angular aggregate particles tend to lock together, resulting in an increased mechanical stability due to the higher internal friction. Rounded aggregate particles, instead of locking together, tend to slide over each other (Figure 2.1). In addition, clean aggregates with 100 percent of crushed faces improve the adhesion between binder and particles, and result in increased temperature cracking resistance.

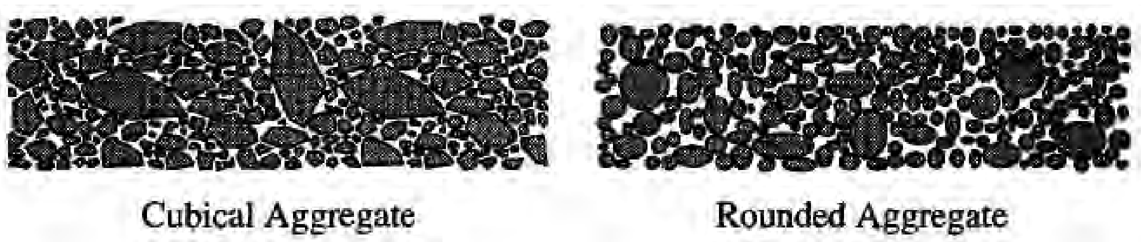


Figure 2.1: Aggregate interlocking (McGennis et al., 1995)

Monismith (1970) found that aggregate shape and surface texture characteristics have an influence on the fatigue life and stiffness characteristics of HMA. His research found that textured and densely graded materials have a better response to compaction and, therefore, provide better stability in the mix due to the higher initial stiffness achieved.

Livneh and Greenstein (1972) recommended that aggregates with elongated particles (flaky aggregates) could be used in HMA production. Nevertheless, stability of samples with flaky aggregates was 15% to 20% lower than stability of samples with more uniform particle shape.

b) Surface texture and absorption

Aggregate absorption leads to changes in the asphalt content of a mix. If high absorptive aggregates are used, effective asphalt content will be reduced, thus reducing film thickness and potentially producing a less durable mixture. If a sound quality control is performed, the contractor will be required to increase the asphalt content to compensate for absorption, but if the quality control fails to correct this deficiency, the effective asphalt content may be lower than the original design, and in consequence, the produced mixture may be prone to raveling. This is particularly important for Texas, where a high percentage of limestone mixtures are used. Nevertheless, texture and high aggregate absorption improve adherence and reduces adhesion failure in fatigue cracking. Porous aggregates increase the specific surface, thus mechanically bonding the binder on the aggregate surface.

c) Gradation

Gradation is a key factor influencing permanent deformation. A gradation close to the Fuller curve for maximum density (a straight line in the 0.45 power gradation curve) will generally produce dense mixes with higher stiffness and increased fatigue resistance. TxDOT's dense-graded hot-mix asphalt (Item 341) and Superpave mixes (Item 344) are designed following a procedure based on this principle. Nevertheless, gradations of maximum density may not provide enough voids in the mineral aggregate (VMA) for air voids and asphalt binder, thus reducing the asphalt film thickness. In consequence, a compromise between VMA and asphalt film thickness should be adopted to improve stiffness at the same time that aging effects are reduced.

Kandhal and Mallick (2007) showed that the maximum nominal size of the aggregate had a significant effect on rutting potential. They found that the rutting potential for binder courses with larger aggregates is less than surface courses with finer aggregates and higher binder content. Earlier, Barksdale et al. (1992) had observed a similar trend in their data, and found that rutting in the surface courses was reduced by approximately 13% using a coarser HMA. The same trends were found in the current study for coarse mixtures (limestone Type C) and fine mixtures (limestone Type D) at the coarse and fine gradation levels with optimum asphalt content. These results are described later in this report. In addition, Barksdale et al. (1992) found that fatigue life decreased by 28%, and the exact percentage reduction depends on the aggregate source. Under these considerations, a compromise between fatigue life and rutting resistance may be achieved by choosing the appropriate coarseness of the HMA.

Chemical Properties of the Aggregate

Mineral components present in the aggregate can provide a fair idea of the potential chemical properties of the aggregate, and most important, the possible reactions and transformations that an aggregate can undergo due to chemical action. However, most aggregates are composed by different minerals and the predominant chemical properties may be difficult to predict. Even if the aggregates are uniform in terms of mineralogy, the chemical properties can be altered by oxidation, hydration, carbonation, leaching, weathering, and foreign coatings (Roberts et al., 1996).

Chemical properties of the aggregate have shown no direct effect on HMA performance, except for the effects on the wetting and chemical affinity between asphalt binder and aggregate that determine the mechanisms of adhesion and stripping, respectively. In consequence, mineralogical composition and aggregate surface charge are directly correlated with moisture sensitivity and damage.

Petersen (1984) suggested that chemical properties of asphalt binder and aggregate may cause a molecular restructuration of the binder at the contact surface with the aggregate that affects bonding properties. This is the reason why asphalt binder bonds better to carbonate aggregates (limestone) than siliceous aggregates (gravel).

Aggregate Gradation

Aggregate gradation is the distribution of particles sizes expressed as a percentage of the total volume or weight. The gradation by volume and weight are approximately the same when the specific gravities of the different aggregates in the blend are similar. Gradation can be graphically represented by a Fuller-Thompson gradation curve that plots cumulative percentage passing per weight in the vertical axis (arithmetic scale), against the correspondent standard sieve opening (particle size) on the horizontal axis (logarithmic scale). Maximum density curves are helpful for performing necessary adjustments of volumetric properties of the HMA design (i.e., VMA) and can be defined by Fuller’s maximum density curve:

$$P = 100\left(\frac{d}{D}\right)^{n}$$

Where:
P: total percent passing the sieve.
d: sieve diameter.
D: maximum size of the aggregate, defined as the largest sieve with material retained.
n: exponent of the equation.

While Fuller's original value for the exponent of the equation was n = 0.50, subsequent studies by Federal Highway Administration (FHWA) recommended modifying the exponent to n = 0.45 for hot-mix asphalt. More recent studies showed that for n = 0.5, the VMA may be too low to ensure both sufficient air void and sufficient asphalt content for durability without encountering field stability problems (Roberts et al., 1996).

The maximum density line is obtained by connecting with a straight line the origin and the maximum size of aggregate at the upper right corner of the 0.45 power gradation chart. This is the approach also recommended by Superpave after the Strategic Highway Research Program (SHRP) in the mid 1990s. The gradation curves are only a guidance and do not imply that mixtures will present a poor performance if they lay above or below the maximum gradation line. It should be noted that maximum density gradations that lay close to the maximum density line will not provide enough VMA to hold adequate air voids and binder that translates into inadequate asphalt film thickness and sensitivity to small changes in asphalt content. Therefore, deviations from the maximum density line are necessary to ensure mix stability.

Gradation is an aggregate property that can be easily controlled through the design and has significant influence on HMA stiffness, stability, durability, workability, fatigue, permanent deformation resistance, and moisture damage.

General recommendations for the mixture design regarding the aggregate selection are (Roberts et al., 1996):

- Stay below the maximum density line if the fine aggregate is predominately round natural sand in order to minimize the amount and effect of sand.
- Stay above the maximum density line if the fine aggregate is crushed or very fine. This approach may not be economical because the voids must be filled with asphalt cement to meet the air voids criteria in the mix design.
- Avoid humps in the gradation curve, especially in the No.30 (0.6 mm) to No.50 (0.3 mm) size range if natural sands predominate in the fine aggregate.

For example, TxDOT specifications do not allow more than 15% natural sands while others do not allow any. There is not a unique gradation that works for a given type of HMA and a given maximum aggregate size, because it depends upon local conditions. Agencies have adopted a range of gradations that have proved to perform adequately under different field conditions, and have recommended master gradation bands for different types of mixtures that define the acceptable upper and lower gradation limits. When the mixture gradation lies outside these bands, potential problems might be present, such as critical mixtures that become readily unstable with slight excess of asphalt or water; porous mixtures, with reduced tensile strength; harsh mixtures, inclined to segregate; and costly mixtures, due to the reduced size of coarse aggregate and the increase in required asphalt binder. TxDOT Master Gradation bands for dense-graded HMA and stone-matrix asphalt (used in this study) are shown in Table 2.1 (TxDOT Standard Specifications, 2004). The specific gradation of the mixtures tested in this study is reported later in this document.

Table 2.1: Master Gradation Bands for Selected Mixtures

Sieve Size	ITEM 340/341 Dense-Graded Hot-Mix Asphalt		ITEM 346 Stone Matrix Asphalt
	Type C	Type D	SMA-D
	Coarse Surface	Fine Surface	Medium
3/4"	95.0 – 100.0	–	100
1/2"	–	98.0 – 100.0	85.0 – 99.0
3/8"	70.0 – 85.0	85.0 – 100.0	50.0 – 75.0
#4	43.0 – 63.0	50.0 – 70.0	20.0 – 32.0
#8	32.0 – 44.0	35.0 – 46.0	16.0 – 28.0
#16	–	–	8.0 – 28.0
#30	14.0 – 28.0	15.0 – 29.0	8.0 – 28.0
#50	7.0 – 21.0	7.0 – 20.0	8.0 – 28.0
#200	2.0 – 7.0	2.0 – 7.0	8.0 – 12.0

When the aggregate gradation is selected and HMA is produced, it is still possible to find some variation between the original laboratory mixture design (designated as JMF1 in TxDOT Specification) and the approved mixture design (designated as JMF2 in TxDOT Specification) that will be used during the construction period. In this case, TxDOT Standard Specifications, Item 340/341 and 346, allows a second variation in the gradation curve, asphalt content, and laboratory-molded density, as shown in Table 2.2.

Table 2.2: Operational Tolerances for Dense-Graded HMA and SMA

Description	Allowable Difference from JMF Target	
	Item 340/341 Dense-Graded HMA	Item 346 Stone-Matrix Asphalt
Individual % retained for #8 sieve and larger	±5.0 [1]	±5.0 [1]
Individual % retained for sieves smaller than #8 and larger than #200	±3.0 [1]	±3.0 [1]
Percentage passing the #200 sieve	±2.0 [1]	±2.0 [1]
Asphalt content, %	±0.3 [1]	±0.3 [1]
Laboratory-molded density, %	±1.0	±1.0

If for any reason JMF1 approaches the upper or lower grading bands, the design is still acceptable if it falls within the operational tolerances. Nevertheless, the percent passing the #200 sieve will be considered out of tolerance when outside the master grading limits. The same consideration is valid for asphalt content and laboratory density. These are the basic considerations that prompted this research study and the effects of these changes in mixture performance, as measured in the laboratory.

Asphalt Binder

Asphalt binder is a cementitious material obtained from the heavy fractions produced in the high temperature distillation of crude oil. At normal air temperatures, asphalt binder varies in consistency from solid to semisolid (Lavin, 2003). The American Society for Testing and Materials (ASTM) defines bitumen as a class of black or dark colored, solid, semisolid, or viscous, cementitious material, natural or manufactured, composed principally of high molecular weight hydrocarbons.

The terms *binder*, *asphalt*, *asphalt cement*, *bitumen*, and *asphalt binder* have been used to describe the same material in the literature. From those terms, the term "asphalt binder" can be used to describe the virgin material mixed with any other ingredient that modifies favorably the engineering properties of the material.

Different types and grades of asphalt with specific characteristics can be produced by refining blends of crude oil (petroleum stocks) coming from different sources, or by subtle adjustments of temperature and vacuum in a high temperature and vacuum distillation process (Asphalt Institute, 2001). These adjustments in the distillation process remove part of the soft compounds and leave the heaviest fractions that will yield the physical properties in the final desired asphalt grade.

The influence of binder on HMA performance can be analyzed from the point of its influence on the different distress mechanisms, such as fatigue cracking, thermal cracking, and permanent deformation.

<u>Fatigue cracking</u>

Cracking due to fatigue is produced for either one or a combination of the following conditions:

- Applied loads that exceed the tensile strength of the material (fracture).
- Repeated traffic wheel loads that generate tensile strains of sufficient magnitude to initiate cracking.
- Inadequate pavement structure, deficient bearing capacity of underlying materials, and mixtures with poor material properties that affect the rheological behavior.
- Environmentally induced stresses.

The fatigue life of laboratory samples is related to the stiffness of the mixture, which is a function, among other factors, of the stiffness of the asphalt binder. However, stiffness does not control, by itself, the fatigue life of pavement. Fatigue cracking resistance depends on the thickness of the asphalt pavement. Low stiffness asphalt mixtures used in thin pavements as well as high stiffness asphalt mixtures used in thick pavements will have a long fatigue life (Decker and Goodrich, 1989). In the first case, HMA has enough flexibility to prevent the initiation of cracking at high levels of tensile strain, and in the second case, the pavement has sufficient thickness to limit the tensile strain level at the bottom of the structure so that fatigue cracking is not initiated.

<u>Thermal cracking</u>

At low temperatures, the behavior of the asphalt mixture is controlled by the behavior of the asphalt binder. As the temperature decreases, the asphalt mixture becomes purely elastic and starts shrinking, which produces accumulation of internal stresses that will eventually lead to a brittle failure and the formation of cracks once the tensile strength of the binder is exceeded. Superpave research has shown that, if the asphalt binder can elongate to more than 1% of its original length during shrinkage, cracks are less likely to develop (McGennis et al., 1995). In contrast, at normal operation temperatures, the temperature stresses are relieved through viscous flow, except for age-hardened mixtures that also present a brittle behavior.

Rutting

As indicated previously, rutting is composed by two mechanisms: densification, and shear deformation. Aggregate and binder properties contribute to reduce accumulation of permanent deformation as described by the Mohr-Coulomb equation (McGennis et al., 1995):

$$\tau_f = \sigma \tan \emptyset + c$$

Within a range of temperatures where the binder behaves as a viscoelastic material, its contribution to the overall mixture shear strength, τ_f, is given by the cohesion factor, *c*. The magnitude of the cohesion factor will be temperature dependant; thus, stiffer binders that maintain their viscoelastic properties at high temperature are able to reduce the amount of permanent deformation (Figure 2.2).

In general, an increase in the stiffness of the HMA causes an increase in the rutting resistance of the pavement. An increase in the viscosity of asphalt binder, or the use of binders with good elastic recovery after a load induced deformation, has the same effect.

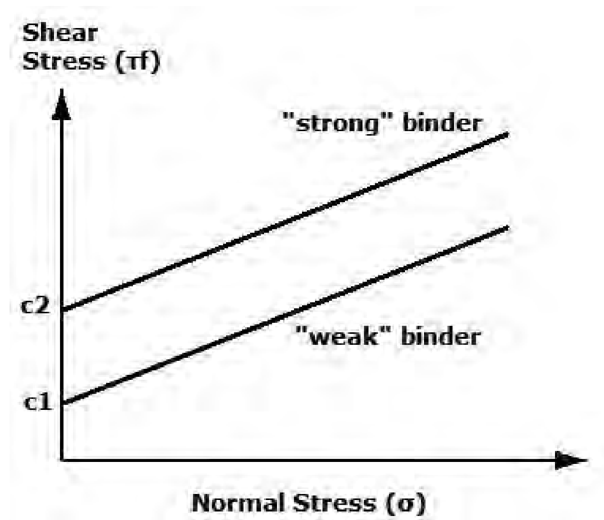


Figure 2.2: Asphalt Contribution to Shear Strength (McGennis et al., 1995)

The HMA performance can also be analyzed from the point of view of the binder characteristics, such as physical and chemical properties.

Physical Properties of the Binder

Physical properties of the binder, such as durability, rheology, and specific gravity are closely related to the original characteristics of the binder, determined by the refining process, and the change in characteristics due to the age hardening phenomena.

Binder aging effects can be evaluated indirectly by subjecting the binder samples to a simulated short-term or long-term aging process, and performing standard physical tests, such as: absolute and kinematic viscosity, rotational viscosity (RV), dynamic shear rheometer (DSR), bending beam rheometer (BBR), and direct tension test (DTT). The comparison of results between neat and aged samples will indicate how sensitive the binder is to aging, and will determine the physical and rheological properties of the material after mixing and construction.

Binder viscosity and rheology define rutting and fatigue cracking resistance of the HMA. At a reference temperature, asphalt mixes that have higher deformation and flow may be susceptible to rutting and bleeding, while those that have higher stiffness may be susceptible to fatigue or thermal cracking when used in thin pavements, as discussed previously. Binders susceptible to loss in viscous flow at low temperatures are prone to thermal cracking.

Chemical Properties of the Binder

The performance of asphalt binder can be explained in part from the point of view of its chemical composition and the chemical model offers useful information on alteration of properties by showing which components cause specific changes in physical properties (Robertson, 2000). Further, composition is the only property that can be altered physically. For instance, the presence of metals and heteroatoms influence the aging process due to the fact that they are more chemically reactive and tend to oxidize more easily (a more detailed explanation of the effects of aging on performance is presented in the following section). Additionally, the presence of heteroatoms in a molecule can increase its polarity and, in consequence, can increase the probability to react with other molecules. In the same way, low temperature viscous flow improves in binders coming from crudes with low molecular weight aromatics, and improved viscous flow at low temperature might help to reduce thermal cracking. On the other hand, a purely elastic behavior at relatively high temperatures is observed in binders distilled from crude oils that contain high levels of branched saturates (wax).

High polar asphalts develop an elastic internal structure due to the intermolecular bonding, and exhibit non-Newtonian (elastic) flow properties at moderate ambient temperatures, which make them more resistant to fatigue cracking and rutting.

Binder Aging

Organic molecules present in asphalt are prone to react with oxygen from the environment in an oxidation reaction. This reaction changes irreversibly the composition of asphalt molecules that results in more brittle structures. Hence the term “oxidative hardening” or “age hardening” used to describe this process.

Aging can occur during three most critical stages: 1) during transport, storage and handling of the binder; 2) during asphalt mix production and construction (mixing, hauling, and compaction); and 3) during long-term service periods of the pavement. During construction, asphalt oxidizes and hardens due to loss of volatile components, given the high work temperatures, and the relatively high surface area and thin thickness of the binder film surrounding the aggregates in the loose stage before compaction. During service, oxidative hardening occurs at a relative slow rate and is climate and season-dependant, being faster in hot climates or during summer, and slower in cold climates or during winter (McGennis et al., 1994).

Deficiencies in compaction (lower than specified density) of hot asphalt mixtures might lead to high air void-content mixtures that are more permeable and thus, permit more air or water to penetrate and contribute to aging. Oxidative hardening results in an increased viscosity of the binder and a general brittle behavior of the mixture. In consequence, it is believed that asphalt binder aging leads to an increase in stiffness, and this property reduces fatigue cracking resistance in thin pavements, but at the same time, improves rutting resistance. Research by Harvey et al. (1995) found that the increase in mix stiffness due to long term aging has little effect, if any, on laboratory fatigue life.

Additives and Modifiers

Asphalt Modifiers

Modification of neat asphalt binder is performed through the use of chemicals, particles, or polymers to enhance particular characteristics. Chemical modification can alter physical properties of the asphalt binder in a favorable manner and produce a binder that presents, for example, adequate stiffness at high temperatures or less cracking at low-temperatures. Particles (e.g., fillers and cellulose fibers) and polymers that form discrete particles serve as thickeners for the neat binder and increase its viscosity. An increased viscosity might control permanent deformation at normal traffic loads and operation temperatures, and asphalt bleeding during construction and service. Finally, polymers, broad used in modified asphalts, form a continuous network in the asphalt binder and transmit their elastic properties to the homogeneous blend. Polymers serve to positively modify high and low-temperature response of the asphalt.

A relatively recent study sponsored by the National Cooperative Highway Research Program (NCHRP) on the characterization of modified asphalts (Bahia et al., 2001) provided a comprehensive classification of asphalt modifiers based on the review of published literature and survey information collected in 1997 from users, producers, and researchers. The distresses shown in Table 2.3 and marked with “x” correspond to the main distress the additive is expected to reduce, yet the table is intended as a guide and should not be generalized to all asphalt sources.

Table 2.3: Generic types of asphalt modifiers for paving applications

Modifier Type	Class	Effect on Distress				
		PD	FC	LTC	MD	AG
Fillers	Carbon black	x				x
	Mineral: Hydrated lime	x				x
	Fly ash	x				
	Portland cement	x				
	Baghouse fines	x				
Extenders	Sulphur	x	x	x		
	Wood lignin				x	
Polymers - Elastomers	Styrene butadiene di-block (SB)	x		x	x	
	Styrene butadiene tri-block/radial (SBS)	x	x	x		
	Styrene isoprene (SIS)	x				

Modifier Type	Class	Effect on Distress				
		PD	FC	LTC	MD	AG
	Styrene butadiene rubber (SBR)	x		x		
	Natural rubber	x				
	Acrylonite butadiene styrene (ABS)	x				
Polymers - Plastomers	Ethylene vinyl acetate (EVA)	x	x			
	Ethylene propylene dine monomer (EPDM)	x				
	Ethylene acrylate (EA)	x				
	Polyethylene (LDPE and HDPE)	x		x		
	Polypropylene	x				
Crumb rubber	Different sizes, treatments, and process	x	x	x		
Oxidants	Manganese compounds	x				
Hydrocarbons	Aromatics			x		
	Paraffinics (wax)			x		
	Asphaltenes	x				
	Shale oil				x	x
Antistrips	Amines				x	
	Polyamides				x	
	Hydrated lime				x	
Fibers	Polypropylene	x	x	x		
	Polyester	x		x		
	Steel	x	x	x		
	Natural cellulose	x				
Antioxidants	Carbamates (lead and zinc)			x		x
	Carbon black	x				x
	Hydrated lime				x	x
	Phenols					x
	Amines				x	x
PD: permanent deformation		MD: moisture damage				
FC: fatigue cracking		AG: oxidative aging				
LTC: low-temperature cracking						

Elastomers are reported to mainly help in mitigation of permanent deformation. To produce “styrenic” polymers, the most common co-monomer used is butadiene. Styrene-butadiene rubber (SBR), also called synthetic latex, is a product of random copolymerization of the two monomers (styrene and butadiene). This product is commonly used in the Fort Worth District. Meanwhile, styrene-butadiene (SB) or styrene-butadiene-styrene (SBS) come from the block copolymerization (i.e., discrete polymerization of connected blocks) (Asphalt Institute, 2007). SBS is the predominant polymer used in Texas to obtain PG76-22 asphalt grade. An increase in the percentage of elastic modifiers added to the binder will result in an increase of the high-temperature stiffness and the strain recovery response.

Plastomers are said to increase high-temperature stiffness of the asphalt, but do not contribute to the elastic response due to the fact that when stretched, they will yield and remain in the stretched position even after the load is released. The most commonly used in asphalt modification are low density polyethylene (LDPE) and ethylene vinyl acetate (EVA).

Antistripping additives added to the binder reduce moisture susceptibility of the asphalt mixture and improve adhesion between aggregates and binder. Liquid additives (i.e., amines and polyamides) are mixed with the neat binder in a concentration close to 0.5% of the total weight of the asphalt binder. This procedure slightly reduces the viscosity of the binder. Dry additives (i.e., hydrated lime) are added in doses of 1% of the total weight of the dry aggregate. In Texas, antistripping agents are used to help meet Hamburg Wheel Tracking Test requirements.

Aggregate Additives

Aggregate additives are materials incorporated to the natural aggregate blend to improve physical characteristics of the final asphalt mix. They can be grouped in three categories: mineral fillers, antistripping additives, and other additives.

Mineral filler refers to the aggregate fraction passing the No.200 (0.075 mm) sieve. The function of mineral fillers is to improve cohesion and stiffness of the asphalt binder, and “fill” a portion of the voids in the asphalt mixture. If the natural aggregates do not provide the necessary filler fraction, then a supplementary material has to be added to the aggregate blend, such as: dust from crushed aggregates, slag dust, hydrated lime, hydraulic cements, fly ash, loess, kiln dusts, etc. The added material has to conform to ASTM D 242-04: “Standard Specification for Mineral Filler for Bituminous Paving Mixtures,” and AASHTO M 17-07: “Standard Mineral Filler for Bituminous Paving Mixtures,” has to flow freely and be free from agglomerations and organic impurities, and needs to have a plasticity index (PI) no greater than 4.0.

Mineral filler is widely used in stone-matrix asphalt (SMA) mixtures in quantities that vary from 5 to 7%, in order to improve mastic stiffness and mixture compactability and workability, as well as to reduce permeability and drain down.

Stripping is defined as loss of adhesion between the aggregate surface and asphalt binder in the presence of moisture with, or without, visible debonding. Antistripping additives added to the aggregate blend commonly are hydrated lime, fly ash, flue dust, and polymeric treatments (see previous section). Addition of hydrated lime produces asphalt mixtures that are more stable, less susceptible to rutting, and less susceptible to moisture damage when compared with mixtures modified with liquid antistripping agents (Sebaaly et al., 2007).

In tests performed in limestone mixtures with different lime treatments to determine rutting damage caused by moisture sensitivity under the Hamburg Wheel-Tracking device (HWTD), it was found that the addition of 1% hydrated lime reduces the Hamburg rut depth by 50% for all binder grades, which is equivalent to an increase of one PG binder grade

(Tahmoressi, 2002). The study also concluded that limestone, gravel, and igneous aggregates, independently of binder grade, showed a significant increase in the number of mixes passing the TxDOT Hamburg criterion, in comparison with the mixtures without lime addition.

2.1.2 HMA Volumetric Properties

Density and Air Voids

Air voids (which are inversely related to density) are a critical control criterion for HMA. Air voids are closely associated with the two factors mentioned before, aggregate gradation and asphalt content, as well as construction temperature and compaction. It is accepted that air voids contents of below approximately 8% are preferred to mitigate the effect of water permeability and accelerated oxidation of dense graded mix (Roberts et al., 1996). Under-compacted HMA can be attributed to various factors during construction, such as low compaction effort (light rollers or too few passes) and inappropriate temperature control (temperature segregation).

On the other hand, too low air voids (below 3%) may contribute to bleeding because there is not enough space to meet asphalt binder expansion at high temperature. This is particularly important for softer grades such as PG64-22. Primarily for these reasons, the level of air voids of in-place hot mix is adopted by TxDOT as one of the acceptance criteria to determine placement pay adjustment factors.

Film Thickness

The asphalt binder in HMA coats the aggregate and forms a film, variable in thickness, but thin enough to present different behavior to the bulk asphalt in terms of oxidative aging and change in asphalt stiffness. Even though film thickness is not part of the PG specifications or mixture design in a direct way, it has consequences related to premature distress in the performance of the asphalt mixture and is directly related to the asphalt content in the HMA and other volumetric properties such as VMA and VFA.

For a more accurate prediction of physical properties of asphalt binder, and the overall asphalt mixture performance, the chemical composition of asphalt must be considered once the asphalt film is less than 50 microns thick, due to the fact that, even with the same aggregate, asphalt may stiffen soften depending on its source (Robertson, 2000). At the present time, researchers are studying the compositional factors that cause this unpredictable change of stiffness after oxidation. The asphalt cement film thickness can be defined as:

$$T_F = \frac{V_{asp}}{SA * W} * 1000$$

Where:

T_F: average film thickness, (microns).

V_{asp}: effective volume of asphalt binder, (liters); defined as total asphalt binder added to the mix minus asphalt binder absorbed by the aggregates.

SA: surface area of the aggregate (m^2/Kg of aggregate).

W: weight of aggregate, (Kg).

Asphalt mixtures with thin film thickness will present problems associated with durability, such as 1) oxidation (brittle HMA and failure for cracking) and 2) water damage (rutting, shoving, raveling and bleeding).

The HMA is expected to present reduced hardening and resistance to the aging process. To fulfill these requirements and obtain a durable mixture, sufficient asphalt binder should be provided to obtain an adequate film thickness and controlled permeability of the mixture.

Weight-Volume Relationships: Volumetrics

In general, the design of asphalt mixtures is a volumetric-based process that has the purpose of determining the volume of asphalt binder, aggregates, and air to produce a mixture with desirable properties. In order to simplify the process, weights are used instead of volume, and the conversion is done through the use of specific gravity. The so-called component diagram (Figure 2.3) describes and illustrates the mass and volume relationships of an HMA specimen. It considers a compacted sample of HMA of unit volume with its constituent materials shown as discrete components.

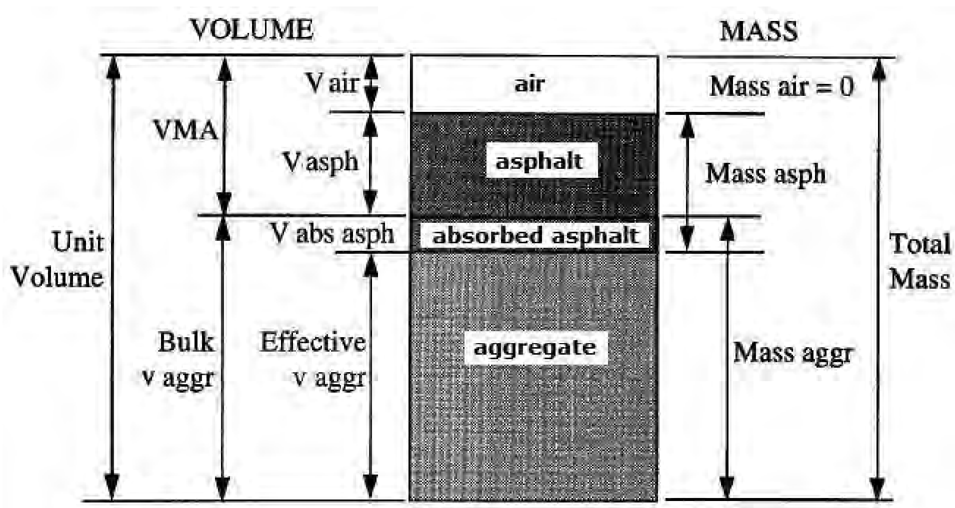


Figure 2.3: Diagram of a Compacted Sample of HMA

The following are the definitions used to determine the volumetric properties of the asphalt mixture:

Voids in the mineral aggregate (VMA)

VMA is the volume of void space between the aggregate particles of a compacted asphalt mixture. It includes the air voids and volume of effective asphalt (i.e., not absorbed). Specified minimum values for VMA at the design air void content of 4% are a function of nominal maximum aggregate size, as shown in Table 2.4.

Table 2.4: Superpave VMA Requirements

Nominal Maximum Aggregate Size	Minimum VMA, %
9.5 mm	15.0
12.5 mm	14.0
19.0 mm	13.0
25.0 mm	12.0
37.5 mm	11.0

The equation for VMA is as follows:

$$VMA = \frac{V_V + V_{EAC}}{V_T} * 100$$

Where:
V_V = volume of air voids.
V_{EAC} = volume of effective asphalt cement.
V_T = total volume of compacted specimen.

TxDOT notation (Tex-204-F):

$$VMA = (100 - \%Density) + \frac{G_a * A_S}{G_s}$$

Where:

$$\%Density = \left(\frac{G_a}{G_t}\right) * 100$$

Ga = Bulk specific gravity of the mixture. Superpave equivalent: Gmb.
Gt = Calculated theoretical maximum specific gravity of the mixture at the specified asphalt content. Superpave equivalent: Gmax-theo.
Gs = Specific gravity of the asphalt binder determined at 25°C (77°F). Superpave equivalent: Gb.
As = Percent by weight of asphalt binder in the mixture. Superpave equivalent: Pb.

Anderson and Bentsen (2001) conducted an investigation to evaluate the influence of change in VMA on the performance-related properties of coarse and fine asphalt mixtures, at two different levels of temperatures. An intermediate temperature level, related to fatigue cracking resistance, and a high temperature level, related to rutting resistance. The concern was the differences in film thickness and, in consequence, in durability that VMA minimum requirements can produce on fine graded mixtures (for example, one mixture graded above the maximum density line) and on coarse graded mixtures (passing below the maximum density line), given an air void content and a maximum aggregate size.

For the experimental design, researches used crushed fine and coarse dolomitic aggregate with a 12.5-mm nominal maximum size. The gradation curves were kept above or below the Superpave restricted zone, and the amount of material passing the No.200 (0.075 mm) sieve was between 4.4 and 5.0%. The asphalt binder used was a PG64-22. The fine and coarse mixtures were designed using the Superpave volumetric procedure, each at a 13% and 15% VMA level. The VMA level was achieved increasing the distance between the sieve curves and the maximum density line (power 0.45). VMA is typically reduced in gradation curves that run parallel to the maximum curve.

Voids in total mix (VTM)

VTM is the total volume of the "pockets" of air between the coated aggregate particles of a compacted paving mixture.

Voids filled with asphalt cement (VFA):

VFA is the percent of the volume of the VMA that is filled with asphalt cement.

$$VTM = \frac{V_V}{V_T} * 100$$

TxDOT notation (Tex-204-F):

$$VFA = \left(\frac{VMA - \%Air\ Voids}{VMA}\right) * 100$$

Asphalt content by weight of mix

$$AC = \frac{W_{AC}}{W_T} * 100 = \frac{W_{EAC} + W_{AAC}}{W_T} * 100$$

Where:

Volume (V_T) and Weight (W_T) of compacted specimen.

Air voids (V_V).

Volume (V_{EAC}) and Weight (W_{EAC}) of effective asphalt cement.

Volume (V_{AAC}) and Weight (W_{AAC}) of absorbed asphalt cement.

Chapter 3. Performance Testing

Superpave volumetric mix design procedure, once known as Level 1 mix design (out of three potential levels), consists of selecting and finding the adequate proportions of aggregate and binder that meet the Superpave quality criteria, and satisfy the volumetric specifications once specimens are compacted in the Superpave gyratory compactor (SGC). The next steps in the original design were mix analysis procedures (former Levels 2 and 3 mix designs), which were used to predict how well a mix would perform in the field by conducting performance based tests. Superpave recommended the use of two devices: the Superpave Shear Tester (SST) and the Indirect Tensile Tester (IDT).

According to Superpave, these devices can determine properties of the HMA that can be used to predict, using analysis tools, fatigue cracking, thermal cracking, and permanent deformation in the field. Therefore, the goal in the Superpave asphalt mixture design procedure consisted of developing an analysis tool to successfully correlate the predictions based on laboratory testing and predict the field performance of the asphalt mixture prior to be used on any transportation facility. Mix analysis procedures are useful to identify, during the early stage of design, premature failures of the asphalt mixture and provide a measurable parameter for quality control (QC) and quality assurance (QA) purposes.

Due to lack of consensus on the performance prediction capabilities of the Superpave Shear Tester and the Indirect Tensile Tester and their complexity, alternative performance-related tests were later evaluated. Simple performance tests (SPT) are defined as test methods that accurately and reliably measure a mixture characteristic or parameter that is highly correlated to the occurrence of pavement distress (e.g., cracking and rutting) over a diverse range of traffic and climatic conditions (Witczak et al., 2002). The current set of SPT comprises dynamic modulus (under dynamic compressive loading), flow time (under static creep conditions), and flow number (under repeated compressive loading).

Performance testing or historical data of HMA pavements can also detect possible problems that mix design (Level 1 mix design) cannot detect, and that is the proper selection of an insensitive HMA that will be able to tolerate variations in asphalt content and gradation due to flaws in the production and construction process. For instance, in a sensitive mixture, a small increase in asphalt content will produce a large decrease in stability and will affect rutting performance.

The concept of sensitivity can be defined as the change in performance due to variation in physical properties of the asphalt-aggregate mixture. Mixture sensitivity is expressed as the ratio between the change of a performance-related property to the change in mixture or material property (Epps and Hand (b); 2001). Historically, sensitive mixtures or critical mixtures have been defined as those whose stability decreased rapidly with an increase in asphalt content (NCHRP, 2002).

Two important aspects should be highlighted at this point. First, there will be mixes that are either sensitive or insensitive, depending on the degree of change in performance due to the variability of material properties, such as: asphalt content, amount of filler. Second, the accuracy of the sensitivity analysis will depend upon the reliability of the performance test used to evaluate the behavior of the mixture.

WesTrack project (1996) was conducted to evaluate the impact of variations in material properties, such as asphalt content, filler content, and in-place air void content (AV), on HMA pavement performance, in order to contribute to the development of performance related specifications. The project consisted of monitoring 26 sections constructed in a full-scale accelerated test facility and subjected to traffic of driverless vehicles with specific axle combination that served to evaluate fatigue cracking and permanent deformation (rutting). The project identified several characteristics inherent to the HMA that might lead to insensitive mixtures:

- Blocky or angular shaped aggregates,
- Aggregates with rough surface texture,
- Aggregate blends with a small fine fraction (passing the 4.75-mm sieve),
- Aggregate blends with a high percentage of crushed fine aggregate,
- Aggregate blends with a low to intermediate sand content,
- Stiff asphalt binder.

On the other hand, the research study found that the presence of one or more of the following characteristics might result in sensitive mixtures:

- Rounded or sub-rounded shaped aggregates,
- Aggregates with smooth surface texture,
- Aggregate blends with a large fine fraction (passing the 4.75-mm sieve),
- Aggregate blends with a high natural sand content,
- Aggregate blends with a high to intermediate sand content,
- Soft asphalt binder.

The experimental design of WesTrack project was similar to the one followed in the current study. It consisted on designing and preparing three different blends (plus one replacement) using crushed andesite, river gravel, and sharp natural sand, and one binder type (PG64-22). Additional mixtures at high and low levels of asphalt content were obtained by varying the target/optimum asphalt content ±0.7%. This value was set assuming a typical variability with a deviation of 0.3%, and expecting to ensure significant differences in rutting and fatigue performance. The air void content was set at three different levels: low (4.0%), medium (8.0%), and high (12.0%).

Ten sections of the WesTrack project showed premature rutting and fatigue cracking in the coarse-graded mixtures and were replaced. A forensic team found that the cause of premature rutting in the replacement sections was caused by high design binder contents coming from over-asphalting during construction, as well as high VMA in the mixture design, associated with high binder content to achieve the target density. The results of the study served for limiting VMA and incorporating a higher minimum dust-to-binder proportion in the Superpave volumetric mix design.

In terms of VMA, Brown et al. (1998) determined that the coarse mixes and SMA had a better rutting performance if the dust-to-binder ratio was kept between 0.8 and 1.6; and the TRB Superpave Committee (2002) advised that mixtures with VMA more than 2% higher than the minimum may be prone to flushing and rutting and should be avoided, unless satisfactory experience with previous high VMA mixtures is available.

From the data collected from WesTrack sections, Epps and Hand (Epps and Hand (b); 2001) investigated the sensitivity of aggregate gradation to variations in asphalt content and filler content, and the combined effects of the previous two variables in air voids (AV). The results showed that the most sensitive mixtures to asphalt content change were the coarse mixtures. For the fine mixtures, researchers found that the ratio of change in AV to the change in asphalt content was a near-linear relationship in the range of analysis, approximately equal to -1.5, indicating a 1.5-percent decrease in AV with a unit increase in asphalt content.

Scholz and Seeds (2001) used WesTrack data to develop performance models to determine the sensitive of HMA to AV. The models predicted that the lower the AV, the lower the rut depths. This is reasonable to a point, but as AV decreases to less than about 4%, dense-graded HMA begins to become unstable and will be more susceptible to rutting.

The particular interest of this study is with those characteristics or parameters of the mixtures that can be measured in the laboratory and that can be used as an indicator to determine how well a given HMA will perform compared to an alternative design (different binder content, density of gradation), and how sensitive the HMA will be when these properties are changed, and how this sensitivity compares to others mixtures.

The performance tests used in this study are flexural fatigue test (BFT), Hamburg wheel-tracking device (HWTD), and overlay tester (OT). These tests are simulative tests that are relatively simple to run. In addition, HWTD is a control test included in TxDOT specifications for the acceptance or rejection of a give job mix formula (JMF), and for quality control purposes. In terms of asphalt content, the expected results of the current study are that a given HMA will have better rutting performance at lower asphalt content levels, but lower fatigue cracking resistance. BFT and OT results are expected to favor HMA mixtures with higher asphalt content. Therefore, a compromise has to be adopted between rutting and cracking resistance when the optimum asphalt content for design is chosen, or when changes to the asphalt content have to be done to adjust JMF and account for production variations.

3.1 Flexural Fatigue Test

The flexural fatigue test is used to evaluate the fatigue characteristics of HMA at intermediate pavement operating temperatures and provide estimates of HMA pavement layer fatigue life under repeated traffic loading for mechanistic-empirical pavement design methods.

3.1.1 Distress: Fatigue Cracking

Fatigue cracking occurs when the asphalt materials are subjected to repeated loads at tensile stress levels that are lower than the tensile strength of the material. Consequently, the HMA, in accordance with the pavement lift thickness, should be designed to resist the maximum number of repetitive stresses and strains applied before significant cracking is observed. This distress is considered to be a structural problem affected by other external factors such as underlying support, placement and compaction quality, age of the asphalt layer, and traffic volume. The main mixture variables affecting fatigue response of properly compacted HMA specimens are asphalt grade and content and air void content, while aggregate characteristics seem to have less influence (Rao et al., 1990). Consequently, to increase fatigue resistance, as much asphalt (as adequate stability allows) should be used in the mixture design and adequate density has to be achieved at the time of compaction.

Resistance to fracture cracking can be evaluated by directly measuring the number of cycles to failure in laboratory specimens or wheel-tracking facilities, or by determining fundamental properties and using fracture mechanics and energy-based approaches. Laboratory tests to evaluate fatigue include (but are not limited to) supported flexure methods: third-point flexural beam, center-point flexural, and cantilever flexural, rotating cantilever, uniaxial, and diametral. The load can be applied following different configurations, waveforms and frequencies, and the mode of loading can be selected to produce either constant stress or constant strain in the sample. The controlled-stress mode of loading is adequate to represent the response of thick asphalt pavements to repetitive loading, while the controlled-strain approach is adequate for thin pavements. The controlled-strain mode of testing results in a greater fatigue life for the same mixture than the controlled-stress testing (Rao et al., 1990) because the applied load is progressively reduced to maintain constant deformation. As discussed later in this study, the controlled-strain approach was used to determine the fatigue life of the laboratory samples. Valid fatigue tests are conducted at temperatures, frequencies and strain levels consistent with those expected in the field.

3.1.2 Test Description: Beam Fatigue Test

A Beam Fatigue Test (BFT) is performed by applying a sinusoidal load, at a specified frequency (typically 10 Hz), to a HMA specimen. The apparatus configuration subjects the specimens to a four-point loading configuration with simple supports at all the reaction points that allow freedom of horizontal displacement and rotation, thus applying a uniform moment in the middle of the beam. The scheme of loading and the apparatus itself are shown in Figure 3.1.

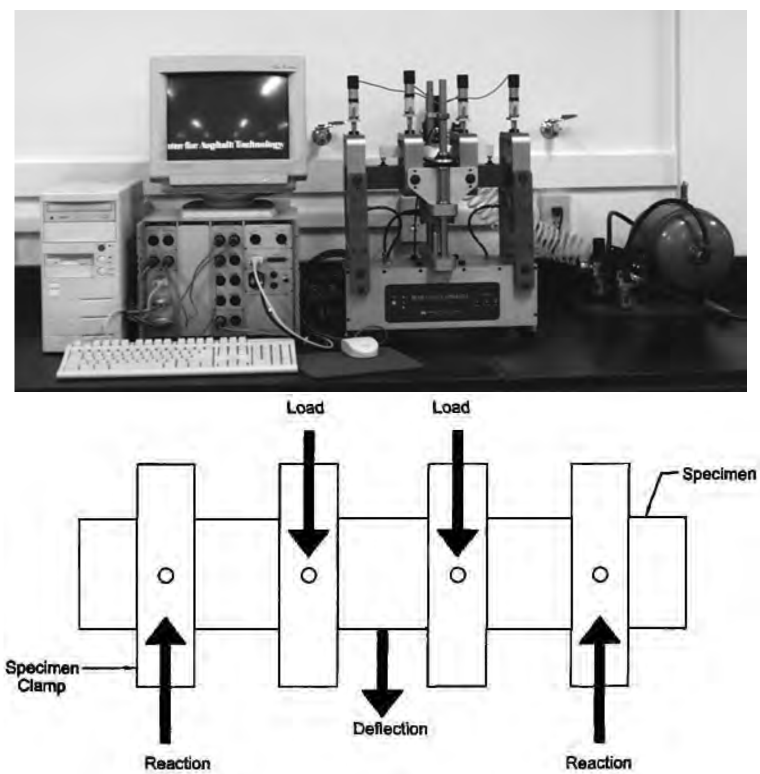


Figure 3.1: Beam Fatigue Apparatus.

BFT is performed according to AASHTO T 321-03: “Determining the Fatigue Life of Compacted Hot-Mix Asphalt (HMA) Subjected to Repeated Flexural Bending.” The dimensions of the samples are 380 mm (15 in) length x 50 mm (2 in) height x 63 mm (2.5 in) width, and the compaction method used to produce them may influence the results. The specimen is tested in an environmental chamber at a temperature of 20.0 ±0.5°C (68.0±1.0°F).

The fatigue test can be run in controlled stress or controlled strain mode. For the controlled strain mode, the load is allowed to decrease with increasing load cycles. The failure criterion has been arbitrarily defined as the load cycle where the beam stiffness reaches a 50% of the initial stiffness. Particular details regarding the setup for the current project testing program can be found in Chapter 5 in this document.

The parameters that are registered at each load application during the test are: maximum tensile stress (σ_t) in Pa, maximum tensile strain (ε_t) in m/m, and flexural stiffness (S) in Pa. The relationships come from simple mechanics theory, as follows:

$$\sigma_t = \frac{0.357\,P}{bh^2}$$

$$\varepsilon_t = \frac{12\delta h}{3L^2 - 4a^2}$$

$$S = \frac{\sigma_t}{\epsilon_t}$$

Where: P is the load applied by the actuator in N; b is the average specimen width in m; h is the average specimen height in m; δ is the maximum deflection at the center of the beam in m; and L is the length of the beam between outside clamps (0.357m).

The terms that are useful for the current study are the initial stiffness (S_0) and 50% S_{50th} that defines failure. The higher strain levels for the test are selected such that S_{50th} is not reached before 10,000 loading cycles. Lower strain levels are selected based on practical considerations. For this research study, the criterion was that failure was reached within reasonable time, e.g., within 24 hours. Too low or too high strain levels produce unrealistic test results that are not necessarily correlated to fatigue performance.

The influence of fatigue on HMA pavement distresses is affected by aggregate characteristics, such as durability, hardness, and toughness. Hard and tough aggregates may give a longer fatigue life to the pavement (Dukatz, 1989).

Harvey et al. (1995) evaluated the effects of asphalt content and air-void content on the fatigue response of a typical California asphalt concrete mix. The researchers, based on SHRP testing and previous experience, proved that, within practical ranges, increased asphalt content and decreased air-void content resulted in increased pavement fatigue life. The rationale of the hypothesis was considered valid because more asphalt increases the film thickness and, as a consequence, reduces the binder strain. In addition, more asphalt area present in the cross-section to transmit tensile stresses reduces the stress level in the asphalt phase. On the other hand, reduced air-void content contributes to a better transmission of stresses (air does not contribute to transmit stresses) and reduces the stress level in the solid components. Less air in the mixture also contributes to a more uniform material with less stress concentration zones at solid-air interfaces, where crack propagation might initiate and interconnect.

The research study also found that for controlled-strain testing, an increase in asphalt content resulted in an increase in laboratory fatigue life and a decrease in mix stiffness. For controlled-strain testing, an increase in air-void content resulted in a decrease of laboratory fatigue life and a decrease in asphalt concrete stiffness. In practical terms, considering a target mixture containing 5% asphalt and 5% air voids, a decrease of 1% in asphalt content results in a 12% decrease in fatigue life, an increase of 1% in air voids results in a 30% decrease in fatigue life, and a decrease of 1% in asphalt combined with an increase of 1% in air voids results in a 39% decrease in fatigue life (Harvey et al., 1995).

3.2 Overlay Tester

Overlay tester (OT) is a performance-related test that was designed in the late 1970s to simulate the reflective cracking process of HMA placed over jointed concrete pavements (Zhou et al.; 2007). The test attempts to replicate the effects of the opening and closing of concrete joints, which were considered the main driving force inducing reflective crack initiation and propagation.

The apparatus consists of two steel plates, one fixed and the other movable horizontally, to simulate the opening and closing of joints or cracks in the old pavements beneath an HMA overlay (Figure 3.2). The specimens can be trimmed from 150-mm-diameter standard field cores or SGC specimens.

The test is performed in accordance to TxDOT standard procedure Tex–248–F: “Overlay Test.” According to the specification, the test is performed at a constant temperature of 77 ±3°F (25 ±2°C). The sliding block applies the necessary tension in a 10-second length cyclic triangular waveform to achieve a constant maximum displacement of 0.025 in. (0.06 cm), as shown in Figure 3.3. The test measures the number of cycles to failure, defined as the point where the pulling force drops 93% of the initial value.

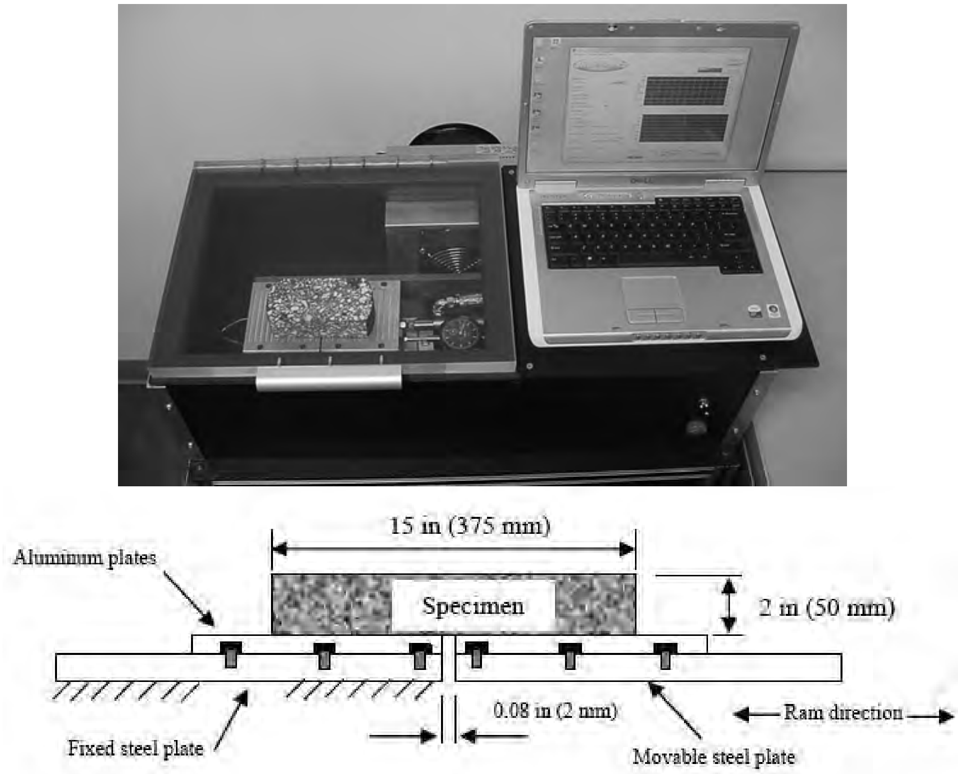


Figure 3.2: OT Apparatus

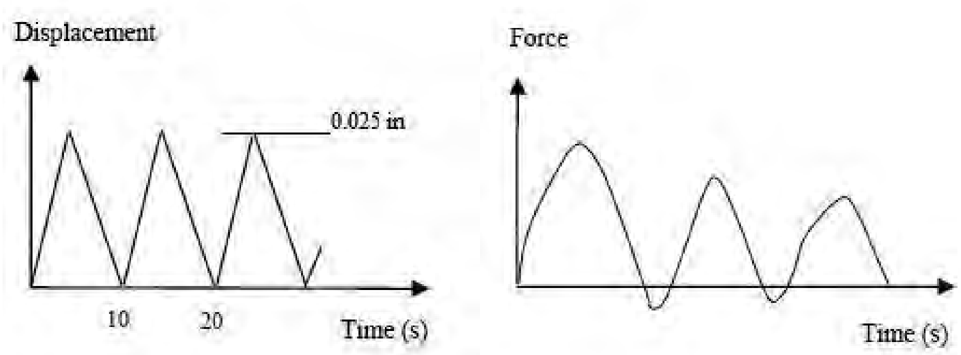


Figure 3.3: OT Load Scheme

In the study performed by Zhou et al. (2007) to determine the sensitivity of the OT to capture variations in HMA properties, it was found that the test was able to detect the effects of temperature, aggregate type (related to absorption), asphalt content, and air-void content. The higher the asphalt content, the better the reflective cracking resistance of the asphalt mixture.

For comparative purposes between OT and BFT, it should be noted that the OT was developed to assess reflective cracking resistance; consequently, higher strain levels are applied to the specimen. Another significant difference is given by the loading time. The loading time in the OT is 100 times longer than the BFT. The parameters for the BFT are selected to assess fatigue cracking. Given that the asphalt mixtures can be considered a visco-elasto-plastic material, the results of the Overlay Tester and the Beam Fatigue Tests are not expected to be similar.

3.3 Wheel-Tracking Tests

Wheel tracking tests are empirical tests that attempt to evaluate the mixture behavior with respect to permanent deformation under a rolling wheel. In the case of the HWTD, when it is performed in a water bath, an assessment of the moisture sensitivity of the mixture can also be obtained.

3.3.1 Distress: Permanent Deformation (Rutting)

HMA should be stable enough to minimize permanent deformation under repeated traffic loads. Permanent deformation occurs as a combination of two failure mechanisms (Figure 3.4): densification (volume change) and shear deformation (shape change) (SHRP, 1994).

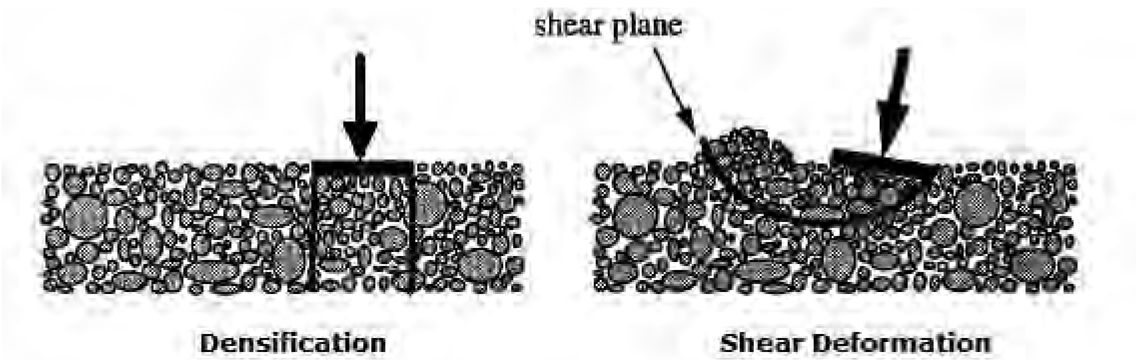


Figure 3.4: Permanent Deformation Mechanisms (McGennis et al., 1995)

Permanent deformation (or rutting of the mixture) is directly related to internal friction provided by the aggregate, and cohesion provided by the asphalt binder. The primary factors affecting rutting are mix properties, temperature, number of load applications, load frequency, and state of stress. The critical condition for permanent deformation accumulation is found at elevated temperatures and lower load frequencies (i.e., slow moving traffic), given that these parameters decreases the viscosity of the binder and, consequently, the material deforms to the extent that traffic loads are carried predominantly by the aggregate structure (aggregate skeleton).

Rutting can be minimized by the selection of materials with appropriate characteristics, such as aggregate maximum size, shape, surface texture, and binder stiffness (e.g., asphalt binder grade); and by the selection of the adequate combination of materials in terms of gradation characteristics, asphalt content, and compaction effort. These factors influence the overall stiffness and the air-void content of the mixture. Permanent deformation is affected by shape, gradation, durability, and toughness of the aggregate (Dukatz, 1989)

Permanent deformation can be evaluated by using predictive models along with laboratory test to determine the model parameters influencing rutting. These models or methods can be empirical regression equations, typical plastic strain laws, and functional equations directly based on laboratory test results. There are several laboratory tests that attempt to characterize this distress and transfer to field performance; for example: uniaxial and triaxial creep load tests, uniaxial and triaxial repeated load tests, dynamic tests, diametral tests, torsion shear tests, simple shear tests, wheel-tracking tests, etc.

3.3.2 Distress: Resistance to moisture induced damage (stripping)

Moisture has a direct impact on the loss of adhesion between the asphalt binder and the aggregate as a result of stripping, and may accelerate the effect of other distresses such as permanent deformation and fatigue cracking. The negative effects of water can be controlled by making the mix as impermeable as possible, and by ensuring the appropriate adhesion between the aggregate surface and asphalt binder. This characteristic is an inherit property of the aggregate and the binder surface energy, and the affinity between both components can be described by the laws of thermodynamics.

Moisture damage can be evaluated qualitatively by the use of laboratory tests: wheel-tracking tests (including but not limited to HWTD), boiling water test, ultrasonic method, and chemical immersion test; or by comparison of tests performed on dry and moisture conditioned specimens by indirect tensile strength, static and dynamic creep tests. Moisture damage appears to be more affected by surface chemistry (composition, solubility, and surface charge) than aggregate shape (Dukatz, 1989).

Laboratory wheel-tracking devices are intended to determine a set of parameters in the asphalt mixture by rolling repeatedly a reduced-scale loaded wheel device across a prepared specimen. The results of the test could be somehow correlated to actual in-service material performance in order to try to make a prediction about rutting, fatigue, moisture susceptibility, and stripping distresses. A more practical application is to compare between two or more mixture designs to determine the one that will present the better characteristics to rutting and moisture-susceptibility.

3.3.3 Test Description: Hamburg Wheel-Tracking Device (HWTD)

The Hamburg Wheel-Tracking Device (HWTD) measures the combined effects of rutting and moisture damage, related to stripping potential, on HMA mixtures. The apparatus is shown in Figure 3.5.

The test's principle consists of rolling a normalized steel wheel (8-inch diameter, 1.85-inch width, and 154-pound weight) across the surface of an specimen immersed in hot water (122 ± 2°F or 50±1°C) for a maximum of 20,000 cycles (or specific deformation depending on the binder grade occurs). The tests are typically conducted on slabs or SGC compacted samples to 7% air voids. The average rut depth is measured and registered by the mid and two adjacent LVDT sensors. The test is performed in accordance with Tex-242-F: "Hamburg Wheel-tracking Test."

Figure 3.5: Hamburg Wheel-Tracking Device.

Chapter 4. Experimental Design

For the current study, an experimental design was developed that contemplates the production of three different types of asphalt mixes, the use of two different aggregates, and four different asphalt contents. This three-level experimental design enabled the quantification of the effects of these variables on performance because these relationships are expected to be non-linear.

The asphalt mixtures used are a coarser dense-graded hot-mix asphalt (Type C), a finer dense-graded (Type D) hot-mix asphalt, and a stone-matrix asphalt (SMA-D). Materials and design were selected according to TxDOT Standard Specifications for Construction, Items 340/341, and 346, respectively, and general TxDOT Departmental Material Specifications.

Fine and coarse aggregates used in this experimental design come from two different quarries: limestone comes from RTI Materials, located in Georgetown, Texas; and gravel comes from Price Construction (Galo quarry), located in Laredo, Texas. The asphalt binder used for this part of the experimental design is an SBS polymer modified performance-graded binder PG 76-22S, from Valero.

The first asphalt mixture produced for each mix type was designed with a target aggregate blend well within the specified gradation band limits. The individual gradations of the different stockpiles used in the design, as well as the gradation of each aggregate blend are shown in Appendix A.

The optimum asphalt content (OAC) was determined using the design procedure by weight specified by Test Procedure Tex-204-F: "Design of Bituminous Mixtures." Table 4.1 shows the different OACs and compaction method used for the HMAs used in this study.

Table 4.1: OAC of the mixtures in the experimental design

Mixture Type	Aggregate	Asphalt Binder	Compaction Method	OAC
Type C	Limestone	PG 76-22S	TGC	4.6%
Type D	Limestone	PG 76-22S	TGC	5.3%
SMA-D	Limestone	PG 76-22S	SGC	6.2%
Type C	Gravel	PG 76-22S	TGC	5.6%
Type D	Gravel	PG 76-22S	TGC	5.9%

The specimens for Type C and Type D mixtures were compacted using the Texas gyratory compactor (TGC) according to Test Procedure Tex-206-F: "Compacting Test Specimens of Bituminous Mixtures." This procedure specifies that the specimens are compacted using rotation to the point where the pressure applied by the hydraulic ram reaches 150 psi (1,034 kPa), and then are compacted without rotation until the pressure dial reads 2,500 psi (17,238 kPa). The Superpave gyratory compactor (SGC) was used for compacting SMA-D specimens applying a normal stress of 600 kPa during 50 gyrations, according to Tex-241-F: "Superpave Gyratory Compacting of Test Specimens of Bituminous Mixtures." The target density for design corresponds, in both of the cases, to the 96% of the maximum theoretical specific gravity (G_{mm}).

The following paragraphs explain the procedure used to produce the five mixtures used in the current project, and how the gradation was varied from the target gradation level to the so-called "coarse" and "fine" levels.

4.1 Dense-Graded HMA Limestone Type C mixture

In order to determine the effects of varying mixture key properties within operational tolerances on laboratory performance tests, additional mixtures were obtained by changing the asphalt content, the gradation, and the density of the original target mixture. It should be noted that, due to the test specifications, the density could only be changed from 92 to 96% of the maximum theoretical specific gravity.

The change in gradation for limestone Type C mixture was achieved by changing the proportions of the different aggregate stockpiles used in the original target design. The aggregate types used in limestone Type C mixture and the corresponding proportions for each gradation level are shown in Table 4.2.

Table 4.2: Aggregate blending for limestone Type C

Aggregate Stockpile	Individual Percentage by Weight		
	Target Gradation	"Fine"	"Coarse"
Delta C-Rock	25.0%	21.0%	25.0%
Centex D-Rock	18.0%	18.0%	23.0%
Centex F-Rock	23.0%	23.0%	18.0%
Centex Manufactured	26.0%	30.0%	26.0%
Travis Field Sand	8.0%	8.0%	8.0%

The fine gradation was obtained from the target gradation by reducing 4.0% the original amount of C-Rock and increasing 4.0% the original amount of manufactured sand. The coarse gradation was obtained by switching the percentage of D-Rock and F-Rock with respect to the target gradation. In other words, D-Rock content was increased 5.0% with respect to the target gradation, and F-Rock was reduced 5.0% from the target gradation. At all times, gradation curves for the fine and coarse gradation, were kept within aggregate specification bands, as shown in Figure 4.1.

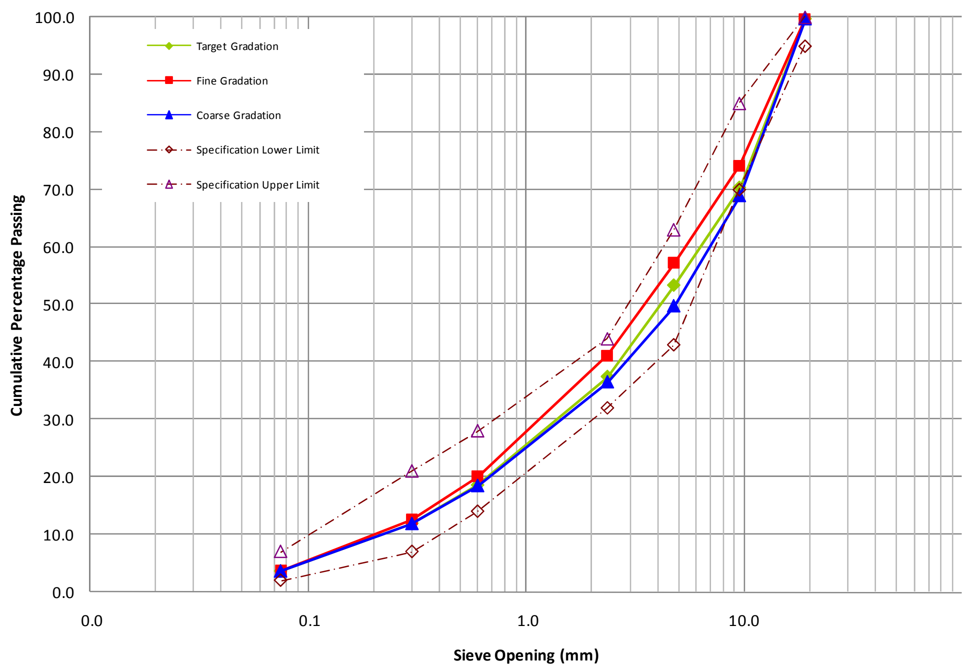


Figure 4.1: Gradation Curves: Limestone Type C mixture

Additionally, the asphalt content was changed for each gradation level. Starting from the OAC, the asphalt content was increased one level towards the richer side, and reduced two levels towards the drier side. The total variation in asphalt content from the dry side to the rich side was one percent.

The purpose behind the procedure of changing gradation level and asphalt content was to minimize and control the number of variables changed in the target mixture to obtain the alternative mixtures. Consequently, there is a tight control of the properties of the specimens molded and tested according to the performance testing procedure discussed previously. Twelve mixes were prepared out of the original limestone Type C design, corresponding to an experimental design with one aggregate type, three gradation levels (target, fine, and coarse), and four asphalt contents (4.0, 4.3, 4.6 (OAC) and 5.0%).

The sieve analysis of the aggregates used, the combined gradation for the different gradation levels, and 0.45 power gradation curves for the produced mixtures are shown in Appendix A.

4.2 Dense-Graded HMA Limestone Type D mixture

Similar to the limestone Type C mixture, all the limestone Type D mixtures produced for the experimental design are derived from a target mixture with OAC. The job mix formula (JMF) used to prepare the limestone Type D mixture at the target gradation level is shown in Table 4.3.

Table 4.3: Aggregate blending for limestone Type D

Aggregate Stockpile	Percentage by weight
	Target Gradation
Delta Grade 4	28.0%
Centex D-Rock	20.0%
Centex F-Rock	15.0%
Centex Manufactured Sand	29.0%
Travis Field Sand	8.0%

The procedure for obtaining the coarse and fine mixes for the limestone Type C produced mixes that were statistically insensitive to the change in the gradation. Therefore, the procedure followed for the limestone Type D mixture to achieve the fine and coarse gradation level was slightly different from the procedure followed for limestone Type C mixture. Once the gradation for the target level was defined, the individual percentage retained in specific sieves was either increased or decreased to the maximum allowed by the operational tolerances, as specified in TxDOT Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges, operational tolerances (refer to Table 2.2).

As shown in Table 4.4, Type D fine gradation was achieved by decreasing 2.1% and 2.9% the individual percent retained in 1/2" and 3/8" sieves, and increasing 2.0%, 1.0%, and 2.0% of the individual percent retained in No.8 sieve, No.200 sieve, and the collecting bin (minus No.200), respectively. On the other hand, Type D coarse gradation was obtained by increasing 5.0% the individual percent retained in 1/2" sieve, and decreasing 2.0%, 2.3%, and 0.7% the individual percent retained in No.8 sieve, No.200 sieve, and the collecting bin (minus No.200), respectively.

Table 4.4: Shifts in gradations for limestone Type D

Sieve Size		Individual Percentage Retained		
US Standard	Metric Standard	Target Gradation	Fine Gradation	Coarse Gradation
1/2"	12.50	2.1%	0.0%	7.1%
3/8"	9.50	18.6%	15.7%	18.6%
#4	4.75	30.8%	30.8%	30.8%
#8	2.36	11.6%	13.6%	9.6%
#30	0.60	17.5%	17.5%	17.5%
#50	0.30	7.4%	7.4%	7.4%
#200	0.075	9.2%	10.2%	7.0%
passing #200		2.7%	4.7%	2.0%

The deviation from the target gradation was the primary criteria to produce the new gradation curves even though these curves were slightly out of the original specification at some points. However, the percent passing the No.200 has been kept within tolerances to meet the specification criteria, as shown in Figure 4.2.

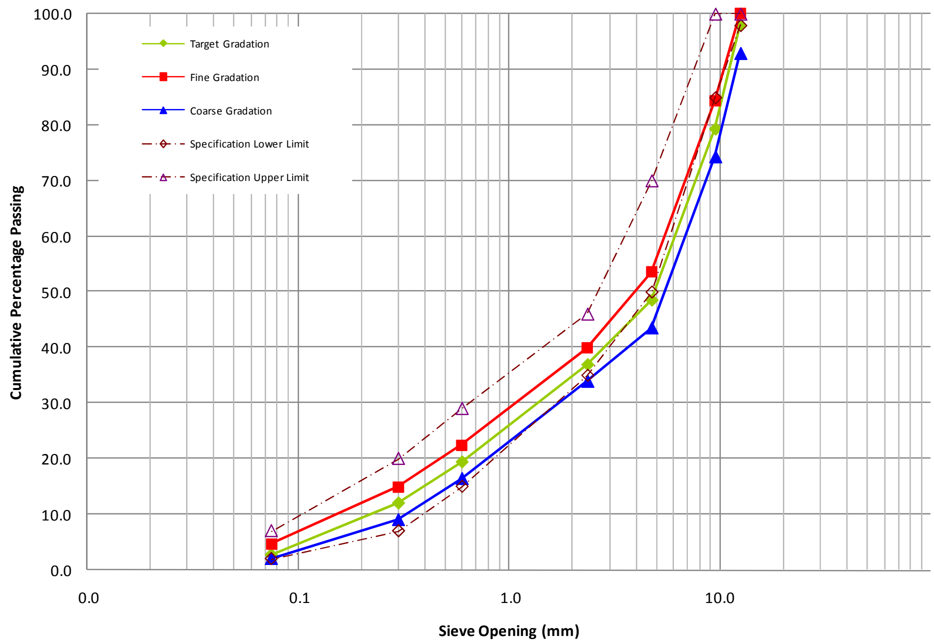


Figure 4.2: Gradation Curves: Limestone Type D mixture

The criteria used to determine the asphalt content for the Type C and Type D mixtures was to increase one level to the "rich" side, and decrease two levels to the "dry" side, from the optimum asphalt content obtained for the target gradation at 4.0% air voids content. Fine and coarse mixtures were prepared with an asphalt content close to the optimum for the target mixture and, depending on the workability of the material, a decision was made to either keep the same asphalt content levels equal to the target mixture, or increase or decrease the asphalt content one level further.

Twelve mixes were prepared out of the original limestone Type D design, corresponding to an experimental design with one aggregate type, three gradation levels (target, fine, and coarse), and four asphalt contents (4.6, 5.0, 5.3 (OAC) and 5.6%). The combined gradations corresponding to the different gradation levels (i.e., target, fine, and coarse) for limestone Type D mixture are shown in Appendix A.

4.3 Limestone SMA Type D mixture

The procedure followed to produce SMA-D mixture is similar to the one used to produce Type D mixture. A total of nine different mixes were prepared using limestone at three different gradation levels, and mixing them at three different asphalt contents. The job mix formula (JMF) used to prepare limestone SMA-D mixture at the target gradation level is shown in Table 4.5.

Table 4.5: Aggregate blending for limestone SMA-D

Aggregate Stockpile	Percentage by weight
	Target Gradation
Centex C-Rock	40.0%
Centex D-Rock	41.0%
Centex Screenings	15.0%
Boral Fly Ash	4.0%
Cellulose fibers	0.3%

The filler used as part of the design is Class F fly ash from Boral (Rockdale, Texas). Fly ash was incorporated into the final blend of aggregates as a separate component in the fine and coarse blend. The fly ash content by weight was slightly modified from gradation level to gradation level in order to meet the specification regarding minus No.200 material. Thus, target gradation has 4.0% content of fly ash, fine gradation, 3.5%, and coarse gradation, 6.0%, in percentage weight. In addition, cellulose fibers were added to avoid drain down of the mixture, in a constant amount of 0.3% of the total weight of mixture.

Figure 4.3 shows the gradation curves for target, fine, and coarse aggregate gradation. Target gradation was kept within the specification bands, but coarse gradation was allowed to lie outside these limits, as far as the operational tolerances in Table 2.2 were met.

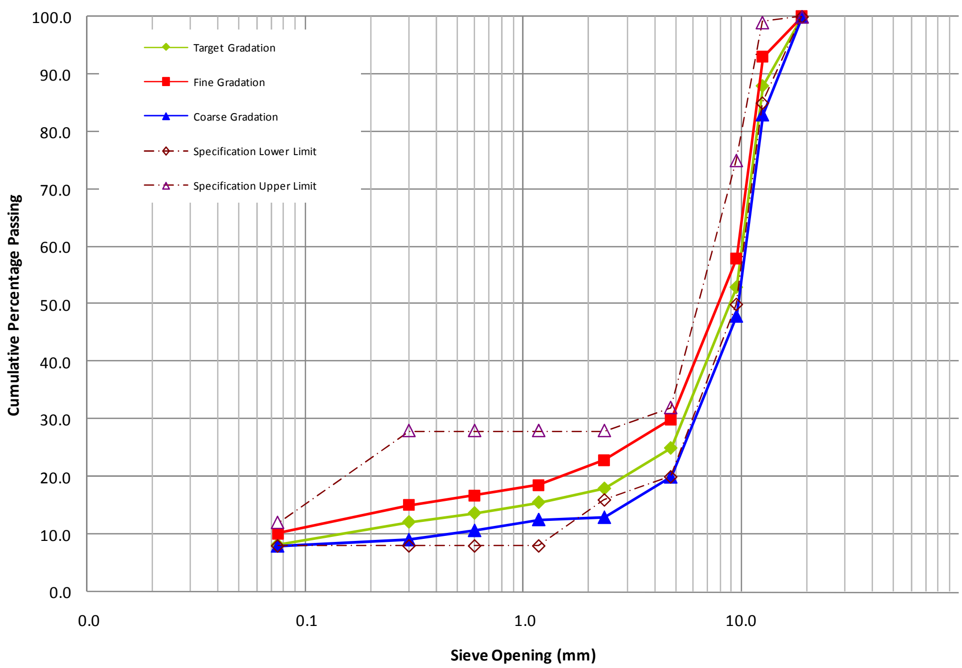


Figure 4.3: Gradation Curves: Limestone SMA-D mixture

The procedure followed to achieve the fine and coarse gradation levels was to define the gradation for the target level and then increase or decrease the individual percentage retained in specific sieves (Table 4.6). SMA-D fine gradation was achieved by decreasing 5.0% the individual percent retained in 1/2” sieve, and increasing 2.0%, 1.0%, and 2.0% the individual percent retained in No.16 sieve, No.200 sieve, and the collecting bin (minus No.200), respectively. On the other hand, SMA D coarse gradation was obtained by doing the opposite. This means, increasing 5.0% the individual percent retained in 1/2" sieve, and decreasing 2.0%, 2.8%, and 0.2% the individual percent retained in No.16 sieve, No.200 sieve, and the collecting bin (minus No.200), respectively.

Table 4.6: Shifts in gradations for limestone SMA-D

Sieve Size		Individual Percent Retained		
US Standard	Metric Standard	Target Gradation	Fine Gradation	Coarse Gradation
3/4"	19.0	0.0%	0.0%	0.0%
1/2"	12.5	12.0%	7.0%	17.0%
3/8"	9.5	35.0%	35.0%	35.0%
#4	4.75	28.0%	28.0%	28.0%
#8	2.36	7.0%	7.0%	7.0%
#16	1.18	2.4%	4.4%	0.4%
#30	0.60	1.9%	1.9%	1.9%
#50	0.30	1.6%	1.6%	1.6%
#200	0.075	4.0%	5.0%	1.1%
passing #200		8.2%	10.2%	8.0%

Nine mixes were prepared out of the original gravel SMA-D design, corresponding to an experimental design with one aggregate type, three gradation levels (target, fine, and coarse), and three asphalt contents (5.7, 6.2 (OAC), 6.5%). The sieve analysis of the aggregates used, the combined gradation for the different gradation levels, and 0.45 power gradation curves for the produced mixtures are shown in Appendix A.

4.4 Dense-Graded HMA Gravel Type C mixture

Following a similar procedure to the ones used with previous mixtures, the job mix formula (JMF) used to prepare gravel Type C mixture at the target gradation level is shown in Table 4.7.

Table 4.7: Aggregate blending for gravel Type C

Aggregate		Percentage by weight
Source	**Stockpile**	**Target Gradation**
Price-Galo	5/8"-1/2"	17.0%
Price-Galo	1/2"-3/8"	13.0%
Price-Galo	3/8"-1/4"	15.0%
Price-Galo	1/4"-1/8"	15.0%
Price-Galo	Screenings	25.0%
RTI-South	Centex Manufactured Sand	15.0%

The procedure followed in gravel Type C mixture to achieve the fine and coarse gradation level was similar to the procedure followed in limestone Type D mixture. Once the gradation for the target level was defined, the individual percentage retained in specific sieves was either increased or decreased to the maximum allowed by the operational tolerances (Table

4.8). Gravel Type C fine gradation was achieved by decreasing 5.0% the individual percent retained in 3/8” sieve, and increasing 2.0%, 1.0%, and 2.0% the individual percent retained in No.30 sieve, No.200 sieve, and the collecting bin (minus No.200), respectively. On the other hand, gravel Type C coarse gradation was obtained by increasing 5.0% the individual percent retained in 3/8" sieve, and decreasing 2.0%, 1.0%, and 2.0% the individual percent retained in No.30 sieve, No.200 sieve, and the collecting bin, respectively.

Table 4.8: Shifts in gradations for gravel Type C

Sieve Size		Individual Percent Retained		
US Standard	Metric Standard	Target Gradation	Fine Gradation	Coarse Gradation
1"	25.0	0.0%	0.0%	0.0%
3/4"	19.0	0.0%	0.0%	0.0%
3/8"	9.5	21.3%	16.3%	26.3%
#4	4.75	27.9%	27.9%	27.9%
#8	2.36	13.7%	13.7%	13.7%
#30	0.60	20.8%	22.8%	18.8%
#50	0.30	5.8%	5.8%	5.8%
#200	0.075	5.8%	6.8%	4.8%
passing #200		4.7%	6.7%	2.7%

Figure 4.4 shows the gradation curves for target, fine, and coarse gradation. The same as explained previously, target gradation was kept within the specification bands, but coarse gradation was allowed to lay outside these limits, as far as the variations are kept within the range required by the operational tolerances in Table 2.2.

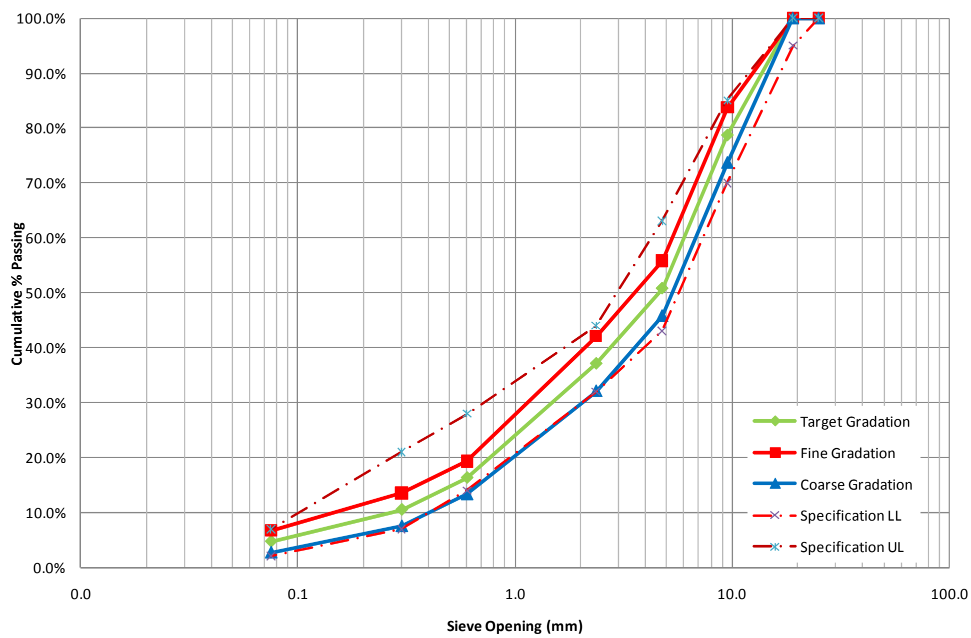


Figure 4.4: Gradation Curves. Gravel Type C mixture

Twelve mixes were prepared out of the original gravel Type C design, corresponding to an experimental design with one aggregate type, three gradation levels (target, fine, and coarse), and four asphalt contents (5.0, 5.3, 5.6 (OAC) and 6.0%). The combined gradations corresponding to the different gradation levels (i.e., target, fine, and coarse) for gravel Type C mixture are shown in Appendix A.

4.5 Dense-Graded HMA Gravel Type D mixture

The job mix formula (JMF) used to prepare gravel Type D mixture at the target gradation level is shown in Table 4.9.

The procedure followed for the gravel Type D mixture to achieve the fine and coarse gradation level was similar to the procedure followed in limestone Type D mixture. Once the gradation for the target level was defined, the individual percentage retained in specific sieves was either increased or decreased to the maximum allowed by the operational tolerances (Table 4.10). Gravel Type D fine gradation was achieved by decreasing 0.6% and 4.4% the individual percent retained in 1/2" and 3/8" sieve, respectively, and increasing 2.0% and 1.0% the individual percent retained in No.30 sieve and the collecting bin, respectively. On the other hand, gravel Type D coarse gradation was obtained by increasing 5.0% the individual percent retained in 1/2" sieve, and decreasing 2.0% and 1.0% the individual percent retained in No.30 sieve and the minus No.200), respectively.

Table 4.9: Aggregate blending for gravel Type D

Aggregate		Percentage by weight
Source	**Stockpile**	**Target Gradation**
Price-Galo	1/2"-3/8"	20.0%
Price-Galo	3/8"-1/4"	20.0%
Price-Galo	1/4"-1/8"	20.0%
Price-Galo	Screenings	25.0%
RTI-South	Centex Man Sand	15.0%

Table 4.10: Shifts in gradations for gravel Type D

Sieve Size		Individual Percent Retained		
US Standard	**Metric Standard**	**Target Gradation**	**Fine Gradation**	**Coarse Gradation**
3/4"	19.0	0.0%	0.0%	0.0%
1/2"	12.5	0.6%	0.0%	5.6%
3/8"	9.5	9.0%	4.6%	9.0%
#4	4.75	35.9%	35.9%	35.9%
#8	2.36	16.8%	16.8%	16.8%
#30	0.60	21.3%	23.3%	19.3%
#50	0.30	5.8%	5.8%	5.8%
#200	0.075	5.9%	6.9%	4.9%
passing #200		4.7%	6.7%	2.7%

Figure 4.5 shows the gradation curves for target, fine, and coarse gradation. The same as explained previously, target gradation was kept within the specification bands, but coarse gradation was allowed to lay outside these limits, as far as the variations are kept within the range required by the operational tolerances in Table 2.2.

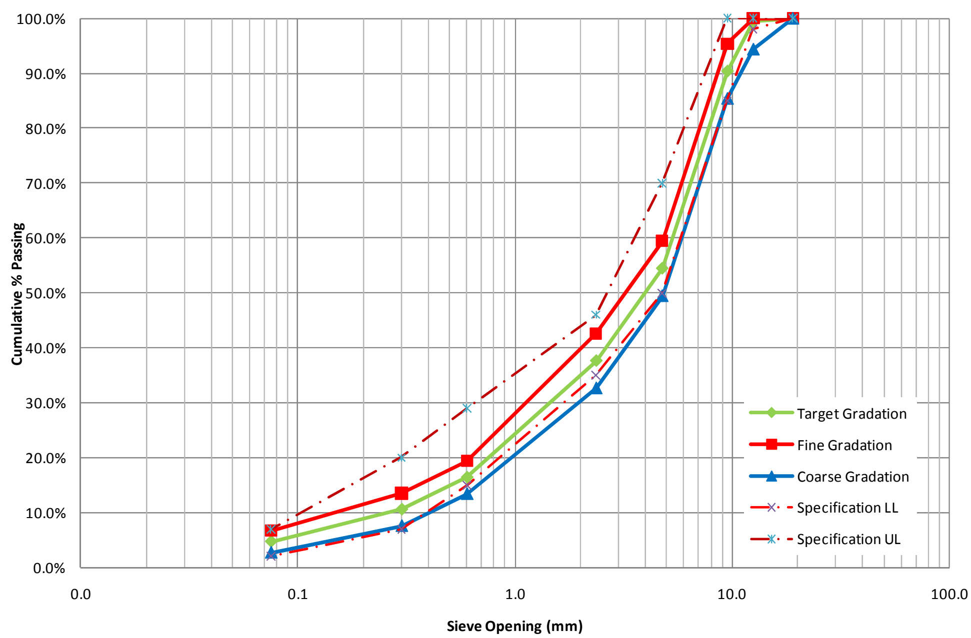


Figure 4.5: Gradation Curves. Gravel Type D mixture.

Twelve mixes were prepared out of the original gravel Type D design, corresponding to an experimental design with one aggregate type, three gradation levels (target, fine, and coarse), and four asphalt contents (5.3, 5.6, 5.9 (OAC) and 6.3%). The combined gradations corresponding to the different gradation levels (i.e., target, fine, and coarse) for gravel Type D mixture are shown in Appendix A.

4.6 Specimen Preparation

For each combination, 110 lbs. of mixture were produced. Each mix was prepared and quartered in order to obtain representative material for performing theoretical maximum specific density (Rice density) tests, Hamburg wheel-tracking test, overlay test, bending beam fatigue life test, and a volumetric control using the Superpave Gyratory Compactor, in accordance with these procedures: Tex-227-F, Tex-242-F, Tex–248–F, AASHTO T 321 -03, and Tex-241-F, respectively.

From each mix, two slabs were compacted using the Automatic Vibratory Compactor (AVC) to obtain four beams to be tested under the Beam Fatigue Test, two more specimens were molded to be tested in the HWTD, and three more specimens to perform the Overlay Testing. The remaining material was used to determine maximum theoretical densities (Rice densities).

Chapter 5. Results

The data collected from the different tests performed on the various mixtures are summarized in this chapter in the form of tables and graphs. In addition, the different parameters and relationships calculated from these test results are presented and the relationships are interpreted and discussed.

The volumetric properties of compacted SGC samples and cut specimens for beam fatigue test (BFT) were determined according to TxDOT standard procedures. Aggregate gradation was determined according to Tex-200-F: "Sieve Analysis of Fine and Coarse Aggregates." The bulk specific gravity of compacted samples was determined according to Tex–207–F: "Determining Density of Compacted Bituminous Mixtures." Rice gravity was determined according to Tex-227-F: "Theoretical Maximum Specific Gravity of Bituminous Mixtures." The hand-held glass pycnometer method was used for Limestone Type C and Type D mixtures, and the metal vibratory pycnometer was used for SMA D, Gravel Type C, and Gravel Type D for determining the Rice gravity.

5.1 Beam Fatigue Test Results

For each mix type, two tensile strain levels were selected within the range suggested by the test procedure (AASHTO T 321-03). The maximum strain value was chosen so that the specimen would not fail before applying 10,000 load cycles. The minimum value was set so that the test would be completed in a reasonable time (i.e., between 24 to 72 hours). For the mixes tested, this range was 300 to 700 microstrains. The applied load frequency was 10 Hz and the failure criterion was set according to AASHTO T 321-03: "Determining the Fatigue Life of Compacted Hot-Mix Asphalt (HMA) Subjected to Repeated Flexural Bending." This test procedure defines failure as the load cycle at which the specimen exhibits a 50% reduction in stiffness relative to the initial stiffness (S_0). S_0 was measured at the 50^{th} cycle. The results obtained from the BFT for all the asphalt mixtures are shown in Figure 5.1 to Figure 5.5, where the number of cycles to failure for the different asphalt contents is plotted as a function of the tensile strain level.

The logarithm of the number of cycles to failure was used for the statistical analysis. This is done because fatigue life under the bending beam follows a log-normal distribution. Figure 5.1 to Figure 5.5 also present the regression lines obtained from the statistical analysis with the independent variables air voids (AV) and initial stiffness (S_0) calculated as the average of air voids and initial stiffness after the 50^{th} load cycle, respectively, for each gradation level: Target, Fine, and Coarse. Thus, only asphalt content (AC) and tensile strain level (Strain) were the independent variables, and the logarithm of number of cycles to failure was the dependent variable. The fine and coarse gradations were modeled as "dummy" variables, mutually exclusive, that adopt the value of 0 or 1 depending on the mixture gradation. The regression and ANOVA results are shown in Table 5.1 to Table 5.5. In general, there is significant correlation between the dependent and independent variables for all the mixes. This is represented by the large value of the F-statistic.

All of the mixtures tested present a similar behavior, according to what was expected. The particular values of change in number of cycles to failure or laboratory fatigue life depend on the mixture type under analysis but the following general trends were observed:

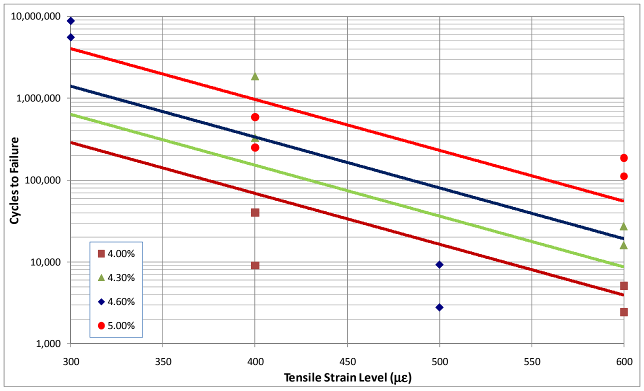
- The number of cycles to failure decrease with an increase in tensile strain level, and
- The number of cycles to failure increase with an increase in asphalt content.

5.1.1 Limestone Type C

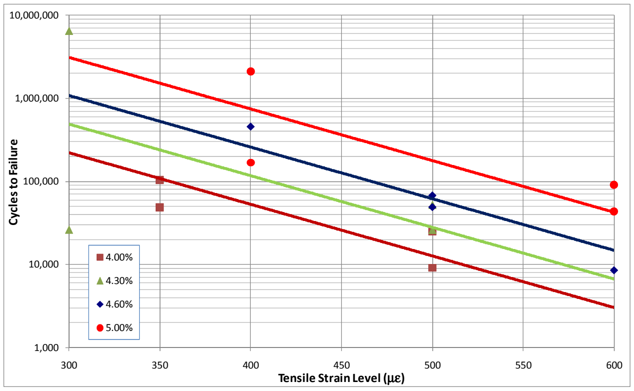
The slope of the regression lines in Figure 5.1 is negative and has a value of -0.0062. This means that, assuming a linear relationship between the logarithm of the number of load cycles and the applied tensile strain, if the strain level increased by 100 microstrains, the laboratory fatigue life would decrease to 24% of the initial life ($10^{-.62} = 0.24$).

Laboratory fatigue life increases as the asphalt content increases. For this mixture, an increase of 1% in asphalt content produces an increase in laboratory fatigue life in the order of 10.8 times ($10^{1.0337} = 10.8$). Graphically, this is represented by the vertical offset between the curves corresponding to 4.0% and 5.0% asphalt content in Figure 5.1.

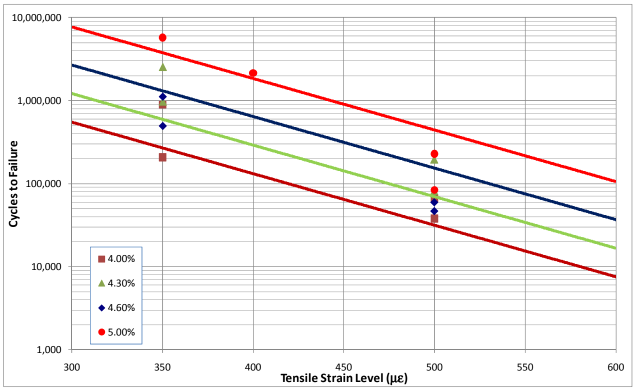
The independent variable asphalt content (AC) and tensile strain (Strain) are both statistically significant, as shown by the p-value in Table 5.1. In practical terms, this means that their effect cannot be ignored because they will significantly affect the fatigue life of this type of mixture.



a) Target Gradation



b) Fine Gradation



c) Coarse Gradation.

Figure 5.1: Tensile Strain vs. Cycles to Failure (limestone Type C)

Table 5.1: BFT regression analysis and ANOVA for limestone Type C

Regression Statistics	
Multiple R	0.82
R Square	0.68
Adjusted R Square	0.63
Standard Error	0.56
Observations	48

ANOVA

	df	*SS*	*MS*	*F*
Regression	6	26.39	4.40	14.20
Residual	41	12.70	0.31	
Total	47	39.08		

	Coefficients	*Standard Error*	*t Stat*	*P-value*
Intercept	1.8584	1.9475	0.95	0.35
AC	103.3734	24.9696	4.14	0.00
Fine	-0.3006	0.2721	-1.10	0.28
Coarse	0.1638	0.2131	0.77	0.45
AV	6.5239	9.7416	0.67	0.51
Strain	-0.0062	0.0009	-6.98	0.00
So	0.0002	0.0002	1.50	0.14

Limestone Type C:

$$Log\ N_f = 1.86 + 103AC - 0.301Fine + 0.164Coarse + 6.52AV - 0.0062Strain + 0.0002S0$$

Where:
Nf = number of cycles to failure. Failure is defined as a reduction in initial stiffness of 50% or more.
Fine, Coarse = mutually exclusive variables that receive the value of 0 or 1 depending on the gradation level of the material.
AC = asphalt content, %.
Strain = tensile strain, με (micro strain).
AV = air void content, %.
S0 = initial flexural stiffness at the 50th load cycle, MPa.

The effect of gradation (target, fine or coarse), air-void content, and initial stiffness are not statistically significant at the 5% level (Table 5.1); however, their average effects for the specimens tested can be described as follows.

For the same asphalt content, by moving the gradation curve towards the fine side of the operational tolerance band, the limestone Type C mixture experiences a reduction in fatigue life to 50% of that of the mixture with the target gradation ($10^{-.3006} = 0.50$). The opposite effect occurs as the mixture shifts to the coarse side of the gradation operational tolerance, where limestone Type C undergoes an increase in fatigue life of 45% ($10^{.1638} = 1.45$).

For the specimens tested, an increase in the mixture stiffness of 1,000 MPa will translate into the fatigue life of the specimen increasing by 58%. Once again, this represents the average effect on the mixes tested but it is a statistically insignificant change because of the typical variability of the test.

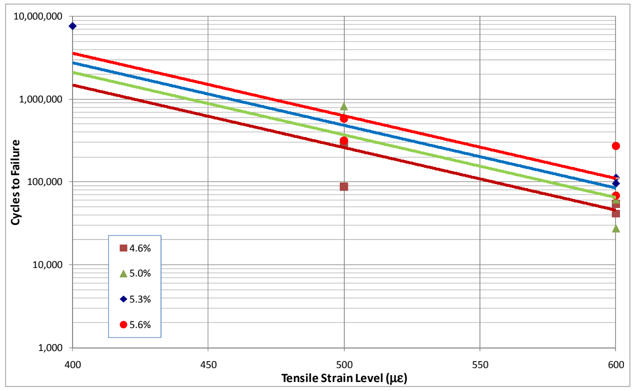
5.1.2 Limestone Type D

The slope of the regression curves is negative and has a value of -0.0076. This means that, assuming a linear relationship between the log of the number of cycles to failure and the tensile strain, if the strain level were to be increased by 100 micro strains, the fatigue life of limestone Type D will decrease to 17% of the original life (Figure 5.2).

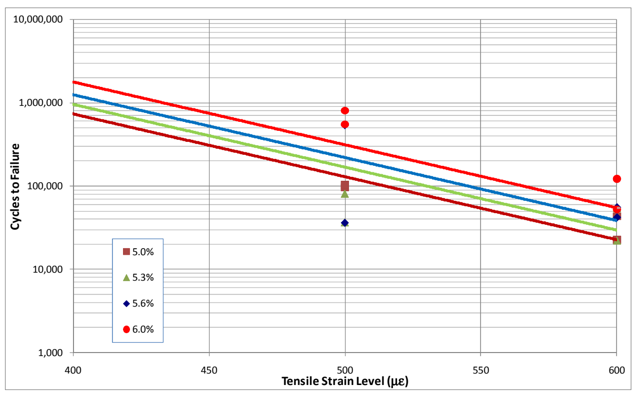
Laboratory fatigue life also increases as the asphalt content increase. An increase of 1% in asphalt content produces an increase in laboratory fatigue life in the order of 2.4 times. Graphically, this value is represented by the vertical offset between the curves corresponding to 4.6% and 5.6% asphalt content.

The independent variables tensile strain and initial stiffness are both statistically significant, as shown by the p-value in Table 5.2. In practical terms this means that their effect cannot be ignored because they will significantly affect the fatigue life of limestone Type D mixture.

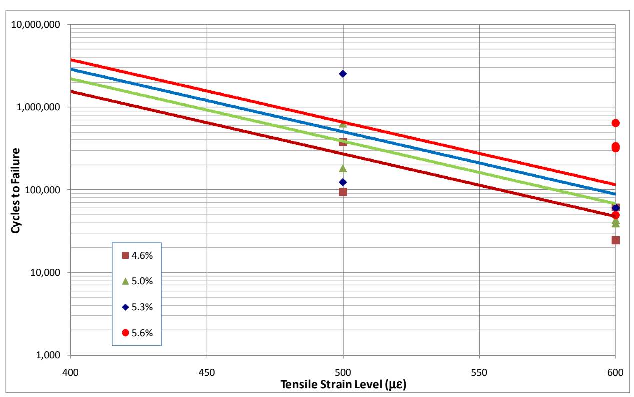
In practical terms this means that their effect cannot be ignored because they will significantly affect the fatigue life of limestone Type D mixture.



a) Target Gradation



b) Fine Gradation



c) Coarse Gradation

Figure 5.2: Tensile Strain vs. Cycles to Failure (limestone Type D)

Table 5.2: BFT regression analysis and ANOVA for limestone Type D

Regression Statistics	
Multiple R	0.86
R Square	0.74
Adjusted R Square	0.70
Standard Error	0.33
Observations	48

ANOVA

	df	*SS*	*MS*	*F*
Regression	6	12.58	2.10	19.50
Residual	41	4.41	0.11	
Total	47	16.99		

	Coefficients	*Standard Error*	*t Stat*	*P-value*
Intercept	9.5440	2.5913	3.68	0.00
AC	38.0564	29.8861	1.27	0.21
Fine	-0.2189	0.1853	-1.18	0.24
Coarse	0.0719	0.1249	0.58	0.57
AV	-9.7864	9.2322	-1.06	0.30
Strain	-0.0076	0.0008	-9.03	0.00
So	-0.0004	0.0002	-2.37	0.02

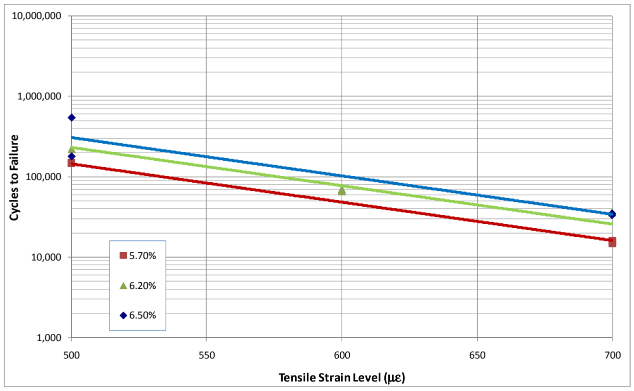
Limestone Type D:

$$Log\ N_f = 9.54 + 38.1AC - 0.22Fine + 0.072Coarse \pm 9.79AV - 0.0076Strain - 0.0004S0$$

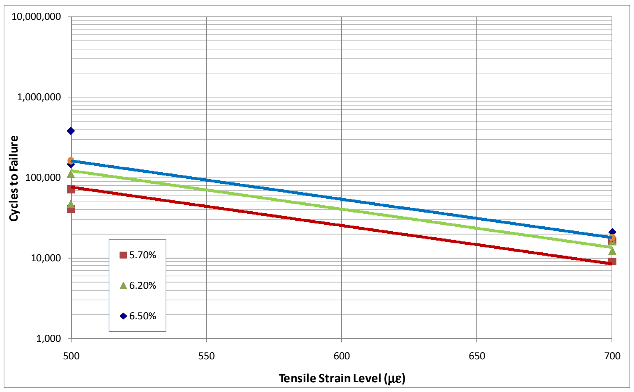
The effect of gradation (target, fine or coarse), air-void content, and asphalt content are not statistically significant at the 5% level (Table 5.1); however, the average effects can be calculated as before. For the samples tested, these effects were not statistically significant.

5.1.3 Limestone SMA-D

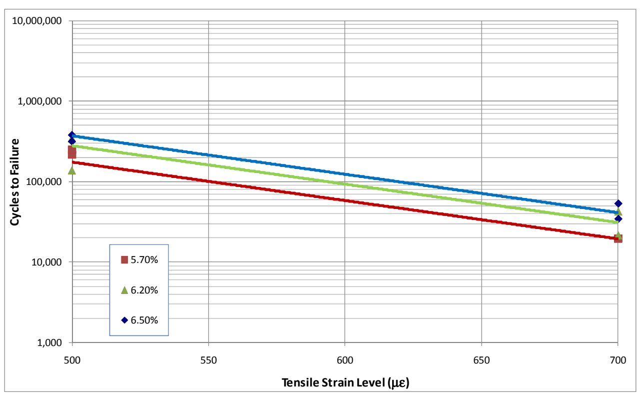
The slope of the regression curves is negative and has a value of -0.0048. This means that if the tensile strain were to be increased by 100 micro strains, the fatigue life of limestone SMA-D mixture would be reduced to 33% of the original life. The same slope is observed for target, fine, and coarse gradation (Figure 5.3).



a) Target Gradation.



b) Fine Gradation.



c) Coarse Gradation.

Figure 5.3: Tensile Strain vs. Cycles to Failure (limestone SMA-D)

Fine gradation, asphalt content, gradation (fine) and tensile strain are statistically significant, as shown by the p-value in Table 5.3. By moving the gradation curve towards the fine side of the operational tolerance band, limestone SMA-D experiences a reduction of 35% in fatigue life. An increase of 1% in asphalt content produces an increase in laboratory fatigue life on the order of 2.6 times. The independent variables coarse gradation, air-void content, and initial stiffness are not statistically significant at 95% confidence level (Table 5.3).

Table 5.3: BFT regression analysis and ANOVA for limestone SMA-D

Regression Statistics	
Multiple R	0.96
R Square	0.92
Adjusted R Square	0.90
Standard Error	0.16
Observations	36

ANOVA

	df	*SS*	*MS*	*F*
Regression	6	8.400	1.400	55.951
Residual	29	0.726	0.025	
Total	35	9.126		

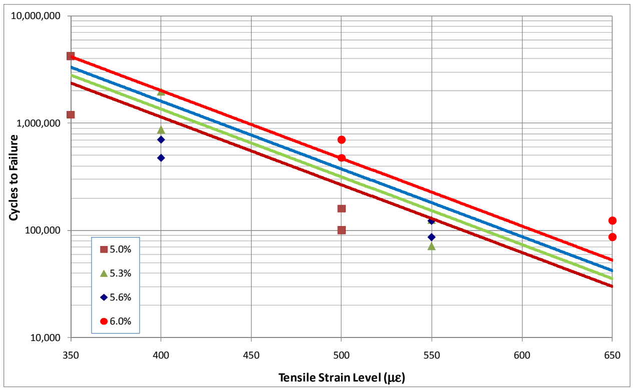
	Coefficients	*Standard Error*	*t Stat*	*P-value*
Intercept	5.525	0.765	7.22	0.00
AC	40.836	8.182	4.99	0.00
Fine	-0.190	0.082	-2.32	0.03
Coarse	-0.076	0.110	-0.69	0.49
AV	1.941	2.251	0.86	0.40
Strain	-0.005	0.000	-14.77	0.00
So	0.000	0.000	-1.50	0.15

Limestone SMA-D:

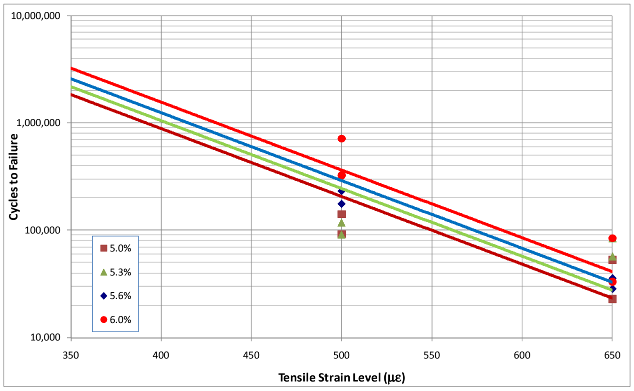
$$Log\ N_f = 5.52 + 40.84AC - 0.19Fine - 0.076Coarse + 1.94AV - 0.0048Strain - 0.0002S_0$$

5.1.4 Gravel Type C

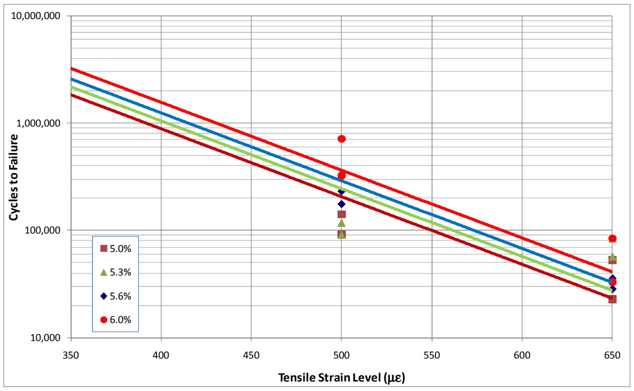
The slope of the regression curves is negative and has a value of -0.0063. This means that if the strain level were to be increased by 100 microstrains, the fatigue life of limestone Type C mixture would be reduced to 23% its original value. The same slope is observed for target, fine, and coarse gradation (Figure 5.4).



a) Target Gradation.



b) Fine Gradation.



c) Coarse Gradation.

Figure 5.4: Tensile Strain vs. Cycles to Failure (gravel Type C)

In this case, the average laboratory fatigue life also increases as the asphalt content increases. This change is not statistically significant at the 5% level. On average, an increase of 1% in asphalt content produces an increase in laboratory fatigue life in the order of 1.8 times. Graphically, this is represented by the vertical offset between the curves corresponding to 5.0% and 6.0% asphalt content.

The independent variables tensile strain and initial stiffness are both statistically significant, as shown by the p-value in Table 5.4. It means that their effects cannot be ignored because they will significantly affect the fatigue life of Gravel Type C mixture. The effect of gradation (target, fine, coarse), air-void content, and asphalt content are not statistically significant at 95% level (Table 5.4).

Table 5.4: BFT regression analysis and ANOVA for gravel Type C

Regression Statistics	
Multiple R	0.93
R Square	0.87
Adjusted R Square	0.85
Standard Error	0.22
Observations	48

ANOVA

	df	*SS*	*MS*	*F*
Regression	6	12.69	2.11	45.43
Residual	41	1.91	0.05	
Total	47	14.59		

	Coefficients	*Standard Error*	*t Stat*	*P-value*
Intercept	10.2173	2.4177	4.23	0.00
AC	24.6209	15.6335	1.57	0.12
Fine	-0.0705	0.1045	-0.68	0.50
Coarse	0.0723	0.0893	0.81	0.42
AV	-12.9535	7.9091	-1.64	0.11
Strain	-0.0063	0.0006	-10.43	0.00
So	-0.0006	0.0003	-2.08	0.04

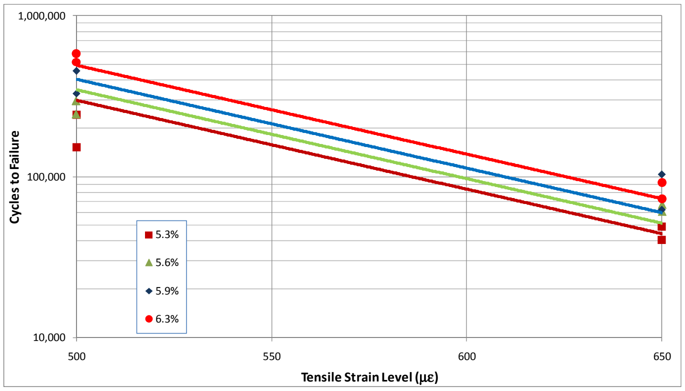
Gravel Type C:

$$Log\ N_f = 10.2 + 24.6AC - 0.071Fine + 0.072Coarse - 12.9AV - 0.0063Strain - 0.0006S0$$

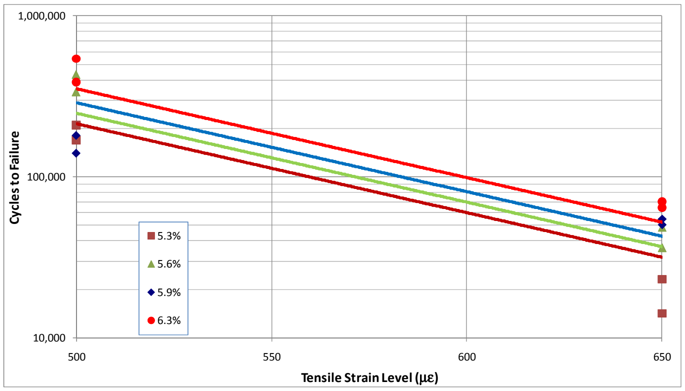
5.1.5 Gravel Type D

The slope of the regression curves is negative and has a value of -0.0055. This means that if the tensile strain were to increase 100 microstrains, the fatigue life of gravel Type D mixture would decrease to 28% of its original life (Figure 5.5).

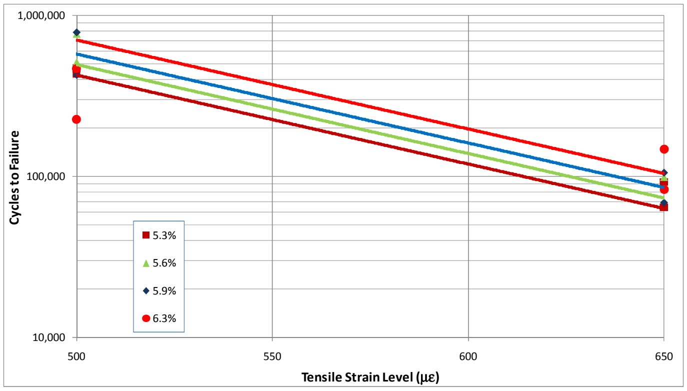
As it was for the Type C gravel mixture, the average laboratory fatigue life also increases as the asphalt content increases; however, this increment is not statistically significant at a 5% level. An increase of 1% in asphalt content produces an increase in laboratory fatigue life in the order of 1.6 times. This represents the vertical offset between the curves at 5.3% and 6.3% asphalt content. Tensile strain level and initial stiffness are both statistically significant, as shown by the p-value in Table 5.5. The effect of gradation, asphalt content, and air-void content are not statistically significant.



a) Target Gradation.



b) Fine Gradation.



c) Coarse Gradation.

Figure 5.5: Tensile Strain vs. Cycles to Failure (gravel Type D)

Table 5.5: BFT regression analysis and ANOVA for gravel Type D

Regression Statistics	
Multiple R	0.95
R Square	0.91
Adjusted R Square	0.90
Standard Error	0.14
Observations	48

ANOVA

	df	*SS*	*MS*	*F*
Regression	6	8.09	1.35	70.66
Residual	41	0.78	0.02	
Total	47	8.88		

	Coefficients	*Standard Error*	*t Stat*	*P-value*
Intercept	8.9902	1.8540	4.85	0.00
AC	21.4555	14.6012	1.47	0.15
Fine	-0.0848	0.0645	-1.31	0.20
Coarse	0.0241	0.0649	0.37	0.71
AV	-7.8966	6.1854	-1.28	0.21
Strain	-0.0055	0.0003	-17.51	0.00
So	-0.0003	0.0001	-2.84	0.01

Gravel Type D:

$$Log\ N_f = 8.99 + 21.5AC - 0.085Fine + 0.024Coarse - 7.9AV - 0.0055Strain - 0.0003S0$$

5.2 Hamburg Wheel-Tracking Device Test Results

The total deformation after 20,000 passes of the HWTD was used as the dependent variable to perform a regression analysis. The independent variables were gradation (target, fine or coarse) and asphalt content (AC). Similar to BFT regression analysis, fine and coarse gradations were represented by dummy variables that adopt the mutually exclusive values of 0 or 1, depending on the aggregate gradation. The results of this analysis are presented in Tables 5.6 to 5.10.

All the tested mixtures present a similar behavior, according to what was expected: the rut depth after 20,000 repetitions of the HWTD increases as asphalt content (AC) increases. The particular values of change in permanent deformation as a function of AC depend on the mixture type under analysis. The following changes were observed for the specimens tested.

5.2.1 Limestone Type C

The slope of the regression curve was obtained by pooling the data of the three gradations together. Its numerical value is 360 as shown in Table 5.6. Because AC is expressed in percentage, this means that for this mixture, an increase of 1% in asphalt content will produce an increase in permanent deformation of 3.6 mm after 20,000 wheel passes, as evaluated with the HWTD test.

For limestone Type C mixture, the specimens with the target gradation experienced the highest permanent deformation as evaluated by the HWTD test. Both alternative gradations, fine and coarse, experienced less deformation after 20,000 cycles (Figure 5.6). The difference is given by the regression coefficients in Table 5.6. On average, the fine gradation rutted 1.58 mm less than the target gradation and the coarse gradation rutted 1.42 mm less than the target gradation. It should be noted that these differences are not significant at 5% level; therefore, it cannot be concluded that, in general, this will be the trend.

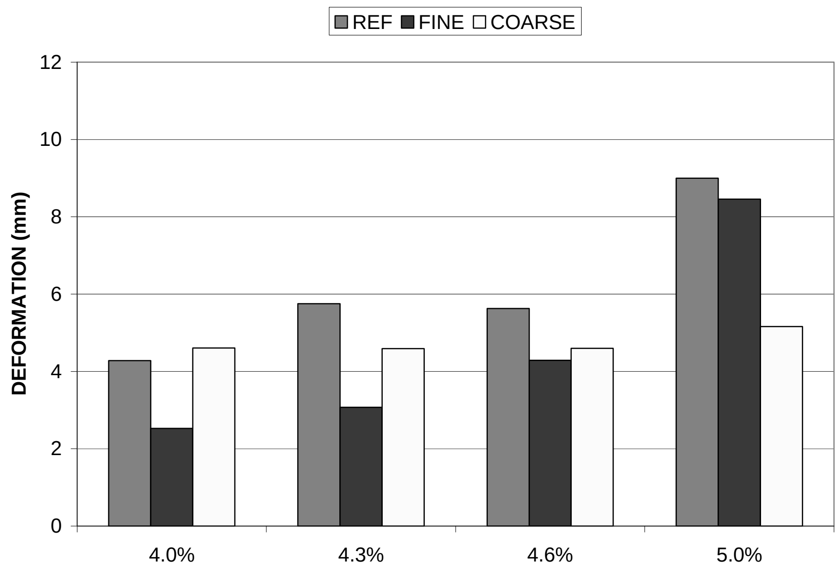


Figure 5.6: Deformation after 20,000 HWTD Passes (limestone Type C)

Table 5.6: HWTD regression analysis and ANOVA for limestone Type C

Regression Statistics	
Multiple R	0.83
R Square	0.69
Adjusted R Square	0.57
Standard Error	1.25
Observations	12

ANOVA

	df	*SS*	*MS*	*F*
Regression	3	27.38	9.13	5.81
Residual	8	12.56	1.57	
Total	11	39.94		

	Coefficients	*Standard Error*	*t Stat*	*P-value*
Intercept	-9.96	4.42	-2.25	0.05
FINE	-1.58	0.89	-1.78	0.11
CORASE	-1.42	0.89	-1.61	0.15
AC	360.35	97.78	3.69	0.01

The regression equation that describes the permanent deformation measured in HWTD after 20,000 passes for limestone Type C is as follows:

$$Def_{@20.000} = -9.96 - 1.58Fine - 1.42Coarse + 360AC$$

5.2.2 Limestone Type D

The slope of the regression curve obtained by pooling the data of the three gradations together (Table 5.7). Its numerical value is 243 and is statistically significant as reflected by its p-value. As AC is expressed in percentage, this means that an increase of 1% in asphalt content will produce an increase in permanent deformation of 2.4 mm after 20,000 wheel passes, as evaluated with the HWTD test.

For limestone Type D mixture, the specimens with the fine gradation experienced significantly higher permanent deformation as evaluated by the HWTD test. Target and coarse gradation levels showed similar deformation after 20,000 cycles, as shown in Figure 5.7. On average, the fine gradation deformed approximately 4.0 mm more than the target gradation. The effect of fine gradation in rutting is statistically significant at 5% level of significance.

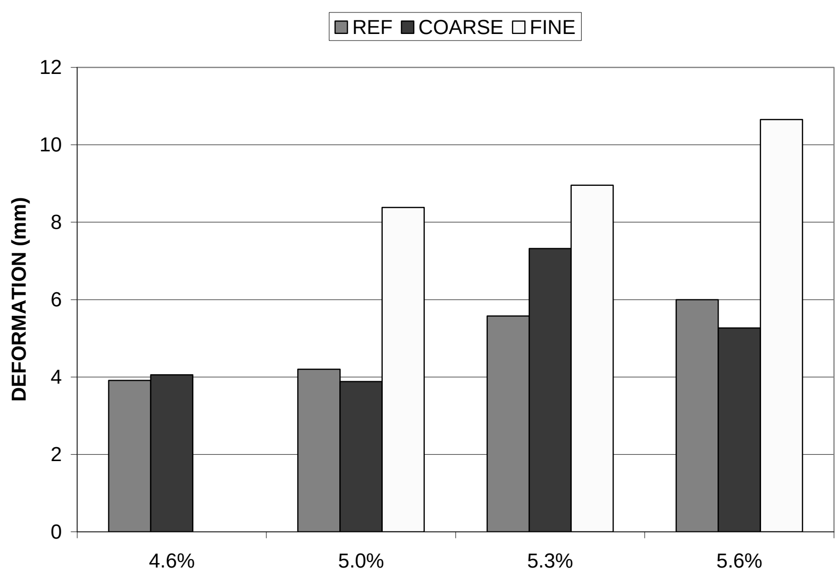


Figure 5.7: Deformation after 20,000 HWTD Passes (limestone Type D)

Table 5.7: HWTD regression analysis and ANOVA for limestone Type D

Regression Statistics	
Multiple R	0.94
R Square	0.89
Adjusted R Square	0.84
Standard Error	0.92
Observations	11

ANOVA

	df	*SS*	*MS*	*F*
Regression	3	47.98	15.99	18.89
Residual	7	5.93	0.85	
Total	10	53.90		

	Coefficients	*Standard Error*	*t Stat*	*P-value*
Intercept	-7.52	4.20	-1.79	0.12
COARSE	0.21	0.65	0.32	0.76
FINE	3.98	0.72	5.55	0.00
AC	242.76	81.48	2.98	0.02

The regression equation that describes the permanent deformation measured in HWTD after 20.000 passes for limestone Type D is as follows:

$$Def_{@20.000} = -7.52 + 3.98Fine + 0.21Coarse + 243AC$$

5.2.3 Limestone SMA-D

For the limestone SMA-D mixture, the slope of the regression line was 423 and is statistically significant as reflected by its p-value (Table 5.8). This slope means that, for this mixture, on average an increase of 1% in asphalt content will produce an increase in permanent deformation of 4.2 mm after 20,000 wheel passes of the HWTD.

The specimens with the fine gradation experienced the highest permanent deformation followed by the target and then the coarse mix, which experienced the lowest deformation after 20,000 cycles (Figure 5.8). On average, fine gradation rutted 1.65 mm more than the target gradation, and the coarse gradation, rutted 0.35 mm less than the target gradation. It should be noted that these differences are not significant at 5% level of significance; therefore, it cannot be concluded that the effect of the tested gradations is significant.

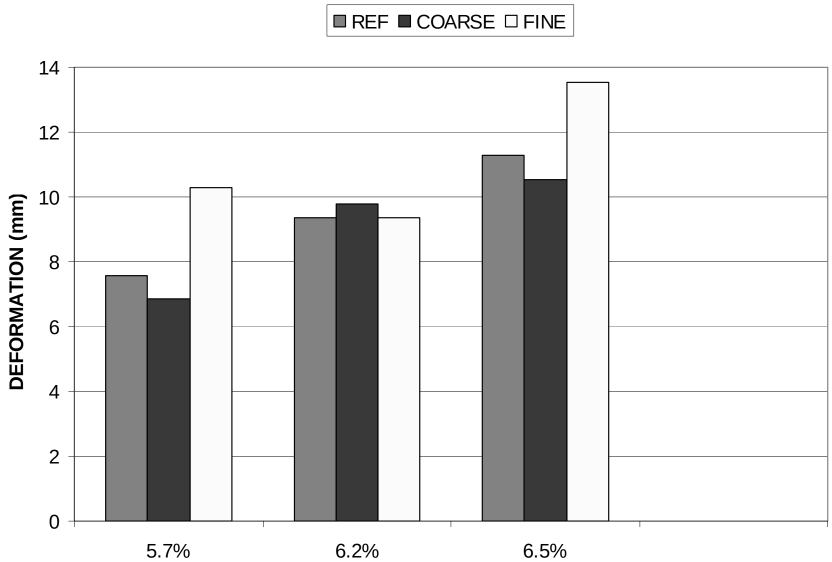


Figure 5.8: Deformation after 20,000 HWTD Passes (limestone SMA-D)

Table 5.8: HWTD regression analysis and ANOVA for limestone SMA

Regression Statistics	
Multiple R	0.89
R Square	0.79
Adjusted R Square	0.66
Standard Error	1.14
Observations	9

ANOVA

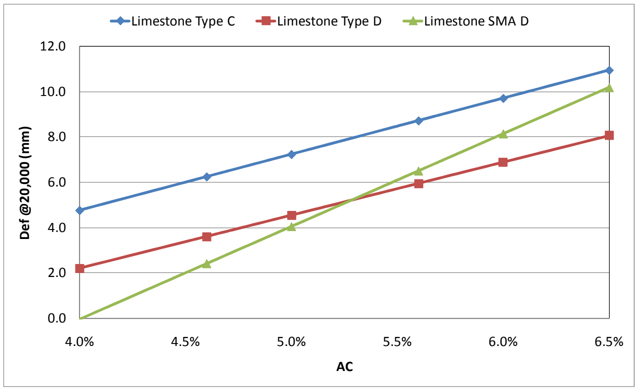
	df	*SS*	*MS*	*F*
Regression	3	24.401	8.134	6.279
Residual	5	6.477	1.295	
Total	8	30.877		

	Coefficients	*Standard Error*	*t Stat*	*P-value*
Intercept	-16.571	7.082	-2.3	0.07
COARSE	-0.347	0.929	-0.4	0.72
FINE	1.650	0.929	1.8	0.14
AC	423.435	114.967	3.7	0.01

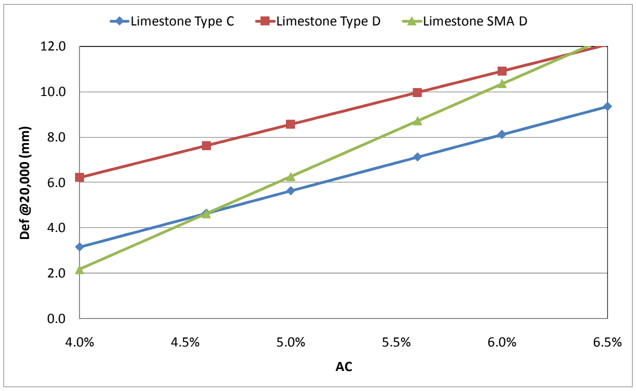
The regression equation that describes the permanent deformation measured in HWTD after 20,000 passes for limestone SMA-D is as follows:

$$Def_{@20.000} = -16.57 + 1.65Fine - 0.35Coarse + 423AC$$

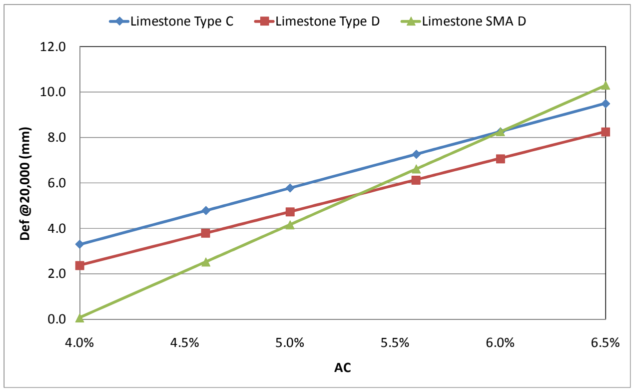
Figure 5.9 presents the summarized regression results for the different asphalt mixtures tested in this study, at the three gradation levels. It can be observed that the limestone Type C and Type D mixtures were less sensitive to changes in the asphalt content. However, it should also be noted that the optimum asphalt content of the dense mixture was significantly lower than that of the SMA mixture.



a) Target Gradation



b) Fine Gradation



c) Coarse Gradation

Figure 5.9: Comparison of limestone mixture after HWTD testing

5.2.4 Gravel Type C

The slope of the regression curve for the gravel Type C mixtures was 437 as shown in Table 5.9. Because AC is expressed in percentage, this means that an increase of 1% in asphalt content will produce an increase in permanent deformation of 4.4 mm after 20,000 wheel passes.

Because the total deformation was low, this mixture can accommodate more asphalt binder and meet the HWTD criterion while at the same time improving its fracture and cracking resistance.

For gravel Type C mixture, the specimens with the target gradation experienced the lowest permanent deformation while both alternative gradations, fine and coarse, experienced more deformation after 20,000 cycles (Figure 5.10). The difference is given by the regression coefficients in Table 5.9. It should be noted that these differences are not significant at 5% level; therefore, it cannot be concluded that, in general, this will be the trend.

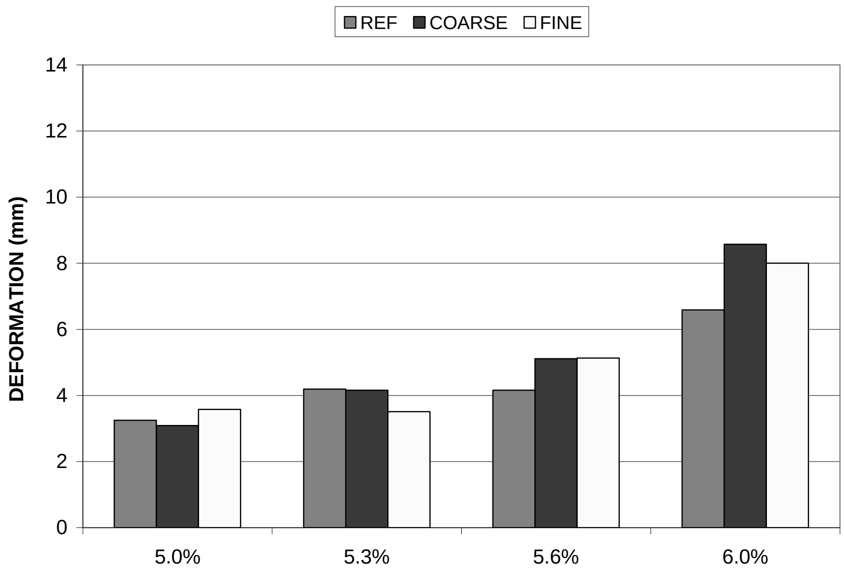


Figure 5.10: Deformation after 20,000 HWTD Passes (gravel Type C)

Table 5.9: HWTD regression analysis and ANOVA for gravel Type C

Regression Statistics	
Multiple R	0.93
R Square	0.87
Adjusted R Square	0.82
Standard Error	0.79
Observations	12

ANOVA

	df	*SS*	*MS*	*F*
Regression	3	32.32	10.77	17.39
Residual	8	4.96	0.62	
Total	11	37.28		

	Coefficients	*Standard Error*	*t Stat*	*P-value*
Intercept	-19.36	3.39	-5.72	0.00
COARSE	0.69	0.56	1.23	0.25
FINE	0.51	0.56	0.91	0.39
AC	436.62	61.42	7.11	0.00

In general, when gravel was use, the deformations were low probably due to the use of a hard and durable aggregate. The regression equation that describes the permanent deformation measured in HWTD after 20,000 passes for limestone Type C is as follows:

$$Def_{@20.000} = -19.4 + 0.51Fine + 0.69Coarse + 437AC$$

5.2.5 Gravel Type D

The slope of the regression (Table 5.10) for this mixture was 252 and is statistically significant. This means that an increase of 1% in asphalt content will produce an increase in permanent deformation of 2.5 mm after 20,000 wheel passes. Because the average deformation for this mixture was approximately 6.4 mm at the end of the test, this particular mixture can accommodate some more binder to increase its resistance to fatigue cracking without the HWTD criterion.

For the gravel Type D mixture, the specimens with the target gradation experienced more permanent deformation as compared with the fine and coarse gradation (Figure 5.11 and Table 5.10). The effect of the different gradations was statistically significant at 5% level of significance.

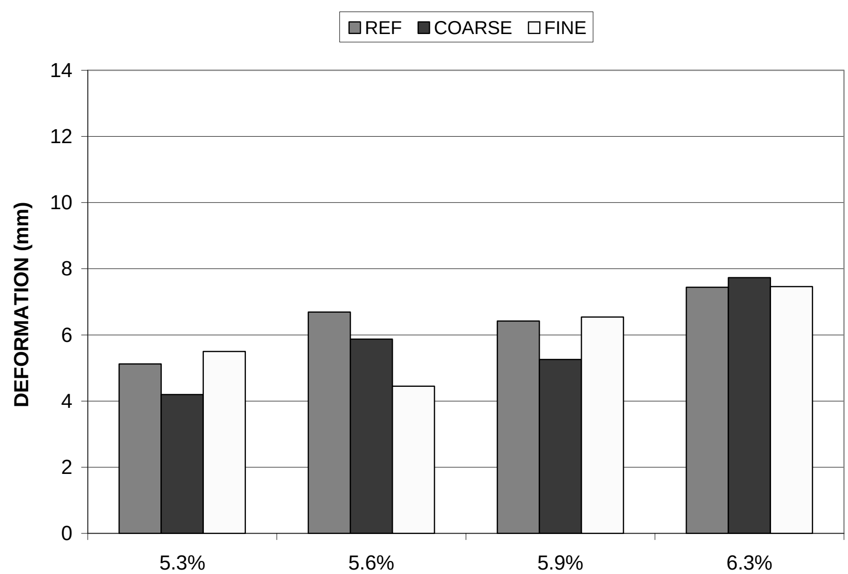


Figure 5.11: Deformation after 20,000 HWTD Passes (gravel Type C)

Table 5.10: HWTD regression analysis and ANOVA for gravel Type D

Regression Statistics	
Multiple R	0.86
R Square	0.74
Adjusted R Square	0.64
Standard Error	0.71
Observations	12

ANOVA

	df	*SS*	*MS*	*F*
Regression	3	11.33	3.78	7.53
Residual	8	4.01	0.50	
Total	11	15.34		

	Coefficients	*Standard Error*	*t Stat*	*P-value*
Intercept	-8.15	3.21	-2.54	0.03
COARSE	-0.65	0.50	-1.30	0.23
FINE	-0.43	0.50	-0.86	0.42
AC	252.18	55.24	4.57	0.00

The regression equation that describes the permanent deformation measured in HWTD after 20.000 passes for limestone Type D is as follows:

$$Def_{@20.000} = -8.15 - 0.65Fine - 0.43Coarse + 252AC$$

5.3 Overlay Tester Results

Three replicates were tested for each particular mixture combination produced. The number of cycles to failure was averaged and used as the representative number of cycles to failure for that mix in particular. The results obtained from the Overlay Tester (OT) for all the asphalt mixtures are shown in the Appendix D.

Figure 5.12 to Figure 5.16 show graphically the results by plotting the number of cycles to failure versus asphalt content (AC) for the three gradation levels: target, coarse, and fine. Regression analyses were conducted for each of the mixtures tested with the following independent variables: asphalt content (AC) and gradation level: target, fine, and coarse. Fine and coarse gradation are “dummy” variables, mutually exclusive, that adopt the value of 0 or 1 depending on the mixture gradation. The number of cycles to failure was the dependent variable.

Regression analyses and ANOVA results are shown in Table 5.11 to Table 5.15. In general, none of the selected variables show statistical significance at a 5% level. In some cases, the effect of asphalt content and gradation is significant at the 10% level. On average, all the tested mixtures present a similar behavior, according to what was expected: the number of cycles to failure increases with an increase in asphalt content. This effect was not statistically significant at the 5% level. The particular values of change of the number of cycles to failure versus asphalt content depend on the mixture type under analysis. The following sections present the results per mix type.

5.3.1 Limestone Type C

The slope of the relationship between the number of cycles to failure and the binder content was calculated from the data of the three gradations for each mixture. For the limestone Type C mixture the slope parameter was 2,815 but it was not statistically significant at the 5% level, as it is reflected by its p-value of 0.17 (Table 5.11). Assuming a linear variation of AC with respect to the number of cycles to failure, this result suggests an average increase of 28 cycles per every 1% increase in AC.

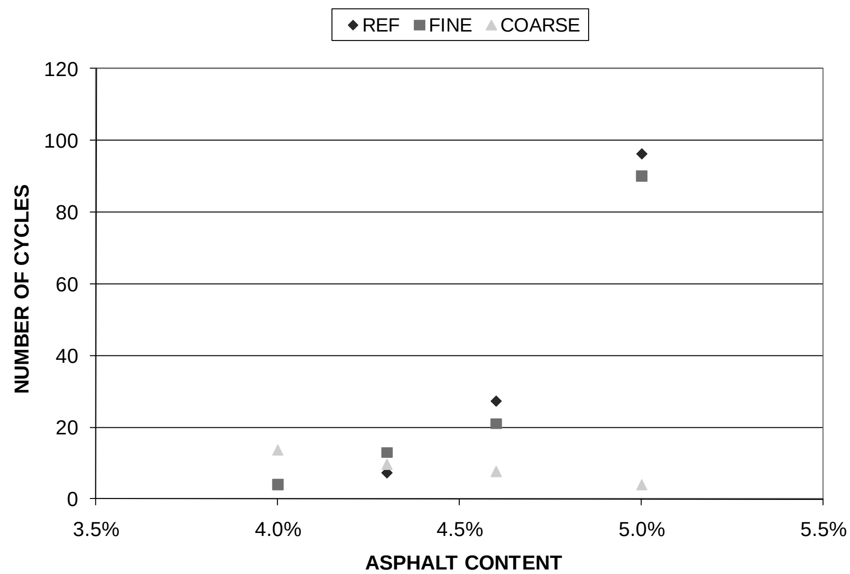


Figure 5.12: Overlay Tester results for limestone Type C

For limestone Type C, target gradation was the gradation that resulted in the best resistance to cracking, as evaluated by the OT. Fine and coarse gradation levels have statistically similar values of number of cycles compared to target gradation.

Table 5.11: OT regression analysis and ANOVA for limestone Type C

Regression Statistics	
Multiple R	0.62
R Square	0.39
Adjusted R Square	0.16
Standard Error	23.69
Observations	12

ANOVA

	df	*SS*	*MS*	*F*
Regression	3	2859.55	953.18	1.70
Residual	8	4489.12	561.14	
Total	11	7348.67		

	Coefficients	*Standard Error*	*t Stat*	*P-value*
Intercept	-92.21	83.56	-1.10	0.30
FINE	-23.25	16.75	-1.39	0.20
COARSE	-25.00	16.75	-1.49	0.17
AC	2814.81	1848.34	1.52	0.17

The regression equation that describes number of cycles to failure in OT for limestone Type C is as follows:

$$NC = -92 - 23Fine - 25Coarse + 2815AC$$

5.3.2 Limestone Type D

The slope parameter of the regression analysis of the data in Figure 5.13 was calculated from the data of the three gradations. Its numerical value is 7333 and is statistically significant at the 10% level, as reflected by its p-value of 0.09 (Table 5.12). Assuming a linear variation of AC respect to the number of cycles to failure, the regression analysis indicates an average increase of 73 cycles per every 1% increase in AC.

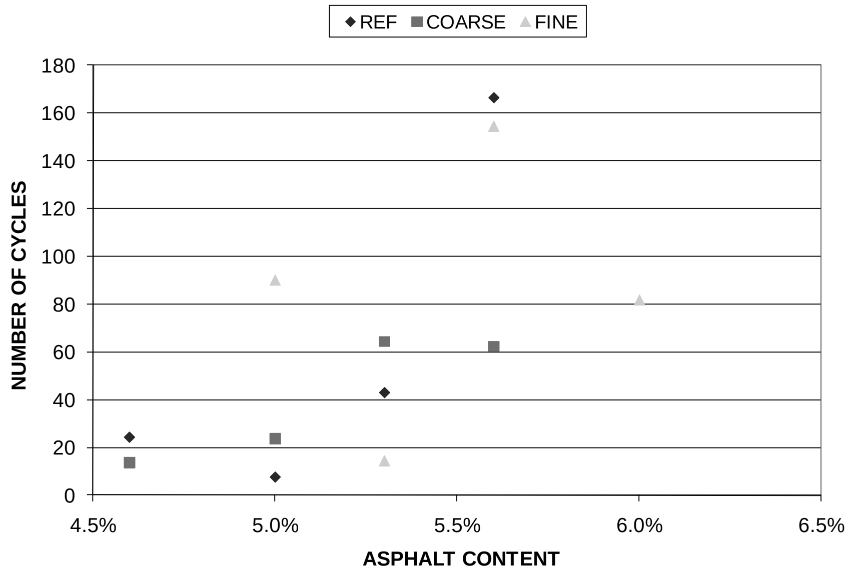


Figure 5.13: Overlay Tester results for limestone Type D

For limestone Type D, target gradation is the gradation level that presents the higher resistance to cracking as evaluated by the OT. Both fine and coarse gradation levels have statistically similar values of number of cycles to failure; however, on average, the coarse gradation will resist 19 fewer cycles while the fine gradation will fail one cycle before. Statistically these mixtures cannot be differentiated.

Table 5.12: OT regression analysis and ANOVA for limestone Type D

Regression Statistics	
Multiple R	0.64
R Square	0.41
Adjusted R Square	0.18
Standard Error	48.29
Observations	12

ANOVA

	df	*SS*	*MS*	*F*
Regression	3	12738.02	4246.01	1.82
Residual	8	18658.53	2332.32	
Total	11	31396.55		

	Coefficients	*Standard Error*	*t Stat*	*P-value*
Intercept	-315.47	194.63	-1.62	0.14
COARSE	-19.33	34.15	-0.57	0.59
FINE	-0.91	36.61	-0.02	0.98
AC	7332.83	3768.26	1.95	0.09

The regression equation that describes number of cycles to failure in OT for limestone Type D is as follows:

$$NC = -315 - 0.91Fine - 19Coarse + 7333AC$$

5.3.3 Limestone SMA-D

The slope of the regression analysis was calculated by pooling all data from the three gradations together (Figure 5.14). Its numerical value is 12,714 and is not statistically significant, as reflected by its p-value of 0.17 (Table 5.13). For this mix, on average, a one percent increase in AC will result in an increase of 127 cycles before failure. The coarse gradation is the gradation level that presents the higher resistance to cracking as evaluated by the OT (74 more cycles than the target gradation). On average, the fine gradation will fail some 7 cycles before the target gradation.

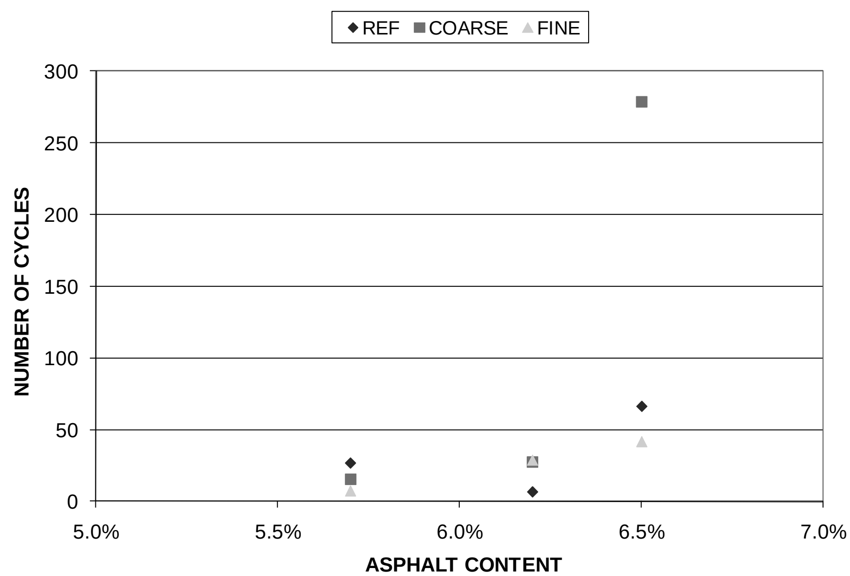


Figure 5.14: Overlay Tester results for limestone SMA-D

Table 5.13: OT regression analysis and ANOVA for limestone SMA-D

Regression Statistics	
Multiple R	0.69
R Square	0.48
Adjusted R Square	0.16
Standard Error	78.50
Observations	9

ANOVA

	df	*SS*	*MS*	*F*
Regression	3	27986.889	9328.963	1.514
Residual	5	30812.889	6162.578	
Total	8	58799.778		

	Coefficients	*Standard Error*	*t Stat*	*P-value*
Intercept	-746.70	488.47	-1.53	0.19
COARSE	74.00	64.10	1.15	0.30
FINE	-7.33	64.10	-0.11	0.91
AC	12714.29	7929.91	1.60	0.17

The regression equation that describes number of cycles to failure in OT for limestone SMA-D is as follows:

$$NC = -747 - 7.33Fine + 74Coarse + 12714(AC)$$

5.3.4 Gravel Type C

The slope of the regression analysis of the data in Figure 5.15 was 6,865 and was not statistically significant at a 5% level, as reflected by its p-value of 0.09 (Table 5.14). Assuming a linear variation of AC respect to the number of cycles to failure, this represents an average increase of 69 cycles per every 1% increase in AC.

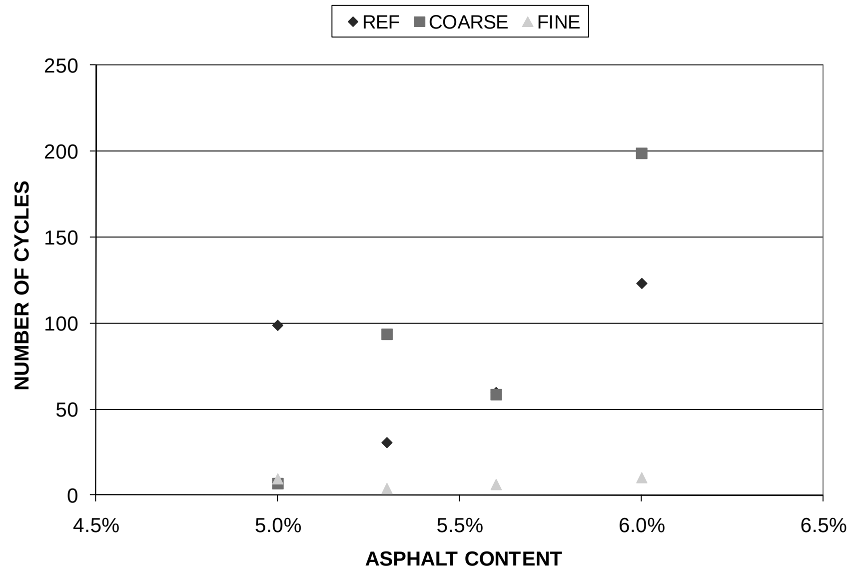


Figure 5.15: Overlay Tester results for gravel Type C

For the gravel Type C mixture tested, the coarse gradation is the gradation level that presents the higher resistance to cracking as evaluated by the OT, while the fine gradation showed the lowest resistance. On average, the coarse gradation resisted 11 more cycles while the fine gradation failed 70 cycles sooner.

Table 5.14: OT regression analysis and ANOVA for gravel Type C

Regression Statistics	
Multiple R	0.76
R Square	0.58
Adjusted R Square	0.42
Standard Error	46.21
Observations	12

ANOVA

	df	***SS***	***MS***	***F***
Regression	3	23434.26	7811.42	3.66
Residual	8	17081.40	2135.17	
Total	11	40515.66		

	Coefficients	***Standard Error***	***t Stat***	***P-value***
Intercept	-297.86	198.75	-1.50	0.17
COARSE	11.33	32.67	0.35	0.74
FINE	-70.42	32.67	-2.16	0.06
AC	6865.04	3605.49	1.90	0.09

The regression equation that describes number of cycles to failure in OT for limestone Type C is as follows:

$$NC = -298 - 70Fine + 11Coarse + 6865AC$$

5.3.5 Gravel Type D

The test results for the gravel Type D mixture are represented in Figure 5.16, where each point represents the average of three specimens. The regression analysis indicates that the slope parameter is 6,193 but it is not statistically significant, as reflected by its p-value of 0.38 (Table 5.15). On average, every 1% increase in AC would result in an increase in the number of cycles to failure of 62.

For the gravel Type D mixture, the fine gradation showed the highest resistance to cracking as evaluated by the OT; on average, it resisted 16 cycles more than the target gradation. The coarse gradation, on the other hand, resisted fewer cycles. It should be noted that these differences were not statistically significant.

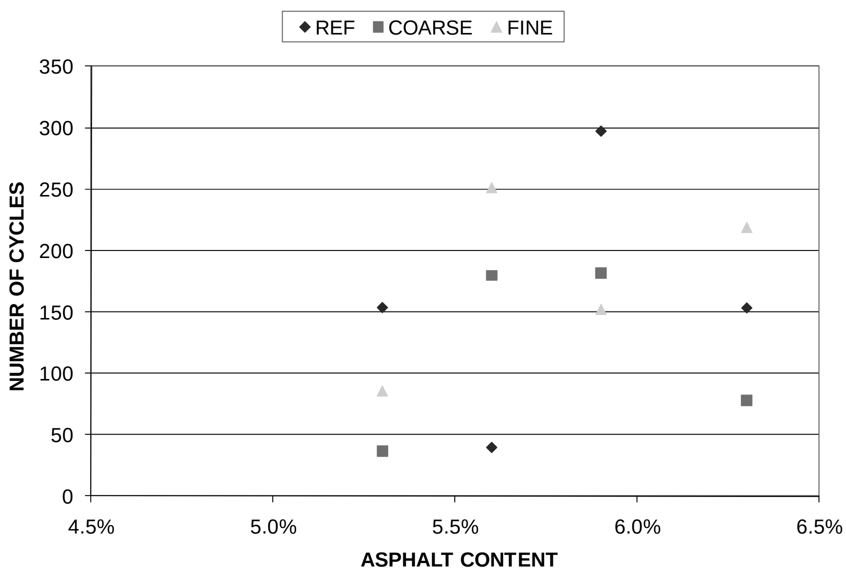


Figure 5.16: Overlay Tester results for gravel Type D

Table 5.15: OT regression analysis and ANOVA for gravel Type D

Regression Statistics	
Multiple R	0.43
R Square	0.18
Adjusted R Square	-0.12
Standard Error	86.15
Observations	12

ANOVA

	df	*SS*	*MS*	*F*
Regression	3	13422	4474	0.60
Residual	8	59368	7421	
Total	11	72790		

	Coefficients	*Standard Error*	*t Stat*	*P-value*
Intercept	-196.97	390.56	-0.50	0.63
COARSE	-41.67	60.91	-0.68	0.51
FINE	16.17	60.91	0.27	0.80
AC	6192.80	6721.70	0.92	0.38

The regression equation that describes number of cycles to failure in OT for limestone Type D is as follows:

$$NC = -197 + 16Fine - 42Coarse + 6193AC$$

Chapter 6. Research Findings and Recommendations

6.1 Final Comments

The development of high performing mixtures is the major objective of HMA design. The reason that most HMA design methods are based on volumetric is that volumetrics there is some correlation with performance. This correlation is by no means perfect, thus, as the HMA design methods become more sophisticated they make use of laboratory performance-related tests. In turn, predictions based on performance tests are not ideal but allow a more confident estimation of performance. The ideal situation would consist of building experimental sections and closely monitoring the short- and long-term performance of the mixtures. This solution is costly and time consuming because the research findings become available only several years after the sections have been built, particularly if the fatigue cracking performance of the mixtures is of interest. For this reason, since the mid 1990s, the Unites States experienced an unprecedented increase in the number of Accelerated Pavement Testing (APT) facilities, which have the ability to simulate field conditions closely under full size traffic loading. The most active APT facilities are currently operational in Alabama, California, Florida, Illinois, Kansas, Louisiana, Minnesota, Mississippi, New Hampshire, Ohio and Washington (DC).

In Texas, HMA design is primarily based on a volumetric approach; that is, some key relationships between the volume and mass of the mixture components (asphalt binder, aggregate, and air voids) are tightly controlled as they are believed to determine the performance of the mixtures in the field. This volumetric approach is complemented with a number of additional laboratory tests that are aimed at accepting or rejecting a particular mixture type for a particular application. These series of tests include Indirect Tensile Strength (ITS), Hamburg Wheel Tracking Device (HWTD), and the Texas Overlay Tester (OT).

No matter how sophisticated the HMA design method is and how extensive and tight the quality control/quality assurance (QC/QA) plan is, deviation from the original job mix formula will always be present. For this reason, states agencies have implemented operational tolerances and pay adjustment factors that allow some flexibility around the mixture design and optimum binder content. Current operational tolerances accepted by the Texas Department of Transportation are given in the 2004 *"Standard Specification for Construction and Maintenance of Highways, Streets and Bridges."* Concerns about the effect that these tolerances have on the performance of the asphalt mixtures gave origin to this research project. Therefore, the primary objective of this research project was the laboratory evaluation of the influence of TxDOT's operational tolerance on the performance of hot-mix asphalt concrete in the laboratory.

Three performance-related tests were selected to assess the performance of five typical TxDOT mixtures as their binder content, gradation and densities were changed within the ranges allowed by the current operational tolerances. The five mixes consisted of three limestone mixtures (Type C, Type D, and SMA-D) and two gravel mixtures (Type C and Type D). The performance tests utilized in this study were the Beam Fatigue Tests (BFT), Hamburg Wheel Tracking Device, and Overlay Tester.

The three tests were run following the latest TxDOT or AASHTO specification. In all cases, the specified densities of the test specimens have to be between 92 and 94% of the maximum theoretical density (i.e. 7 ± 1% air void content). This range of densities proved to be insufficient for quantifying the effect of density of the performance of the mixtures tested. There is enough evidence in the literature that lower densities will negatively affect performance. This is also supported by engineering knowledge; however, laboratory tests outside the specifications will have to be performed to assess such sensitivity. This aspect was out of the scope of this study but remains subject of further research.

The effect of asphalt content on performance was captured by the three tests to different extents. The least variable and most sensitive test was the Hamburg Wheel Tracking, followed by the Beam Fatigue Test and finally the Overlay Tester. Most results of the OT did not identify any statistically significant trends. It should be noted, however, that most mixtures tested were dense-graded mixture (Items 340/341), which were not expected to perform well with the OT. In addition, the OT was initially developed to assess the resistance of the mixtures to reflection cracking and not to fatigue cracking. In order the better evaluate fatigue cracking, the strain level applied by the OT should be significantly reduced to better simulate actual traffic conditions. In addition, the loading time should also be significantly reduced to represent frequencies more closely associated with highway traffic. It is expected that these modifications have the potential to reduce the variability of the OT results.

The effect of gradation on performance was captured by some of the tests for some of the mixtures as was described in Chapter 5 of this report. Interestingly, in some cases the mixture with the target gradation performed the best while in other cases it was the coarse or the fine mixture. The conclusions cannot be generalized and should be determined on a case-by-case basis. There is some evidence to suggest that mixes to the finer side of the operational tolerances should be avoided, if possible. This conclusion cannot be generalized at this time but it deserves further research. For some of the mixes tested, the finer mixes performed worst in terms of deformation and cracking response.

One of the most important conclusions from this study is that ***the expected performance of four of the five mixtures tested could be significantly improved by increasing their binder content without radically compromising their expected deformation resistance*** in terms of HWTD. The summary statistics presented in Table 6.1 are used to illustrate this statement. For each of the mixes tested, the average maximum deformation after 20,000 passes of the HWTD is given in the table together with their rutting sensitivity expressed in mm of deformation for each additional one percent binder content.

Table 6.1: Summary performance statistics for the mixture tested

Mixture Type	*OAC (%)*	*Max. Deformation (mm)*	*Rutting Sensitivity*	*Additional binder?*	*Fatigue Sensitivity*
Limestone Type C	4.6	5.8	3.6	Yes	10.8
Limestone Type D	5.3	5.6	2.4	Yes	2.4
Limestone SMA-D	6.2	9.4	4.2	No	2.6
Gravel Type C	5.6	4.4	4.4	Yes	1.8
Gravel Type D	5.9	6.4	2.5	Yes	1.6

It can be observed that all mixes, except for the SMA-D, have the potential for increased asphalt content and still meet current HWTD criterion. By doing that, the fatigue resistance of the mixture (as measured in the laboratory) could be significantly improved as indicated in the last column by the fatigue sensitivity. The fatigue sensitivity expressed the ratio of the fatigue life, as measured by the Beam Fatigue Tests, when the binder content of the mix is increased by one percent around the optimum binder content. The only mixture that could not accept additional asphalt is the SMA-D, which already has a high binder content, i.e., 6.2%.

6.2 Specific Research Findings

In this section, some of the most important research findings after the extensive testing program are presented. These findings are restricted to the laboratory testing conditions and procedures used in this study and should not be generalized outside the range of mixtures characteristics and materials tested.

i) ***Stiffness.*** The mixes can be ranked according to average initial stiffness determined during the Beam Fatigue Test. The stiffer mixture at 20°C (68°F) was the limestone Type D (with an initial stiffness S_0 = 4,368 MPa), followed by the limestone Type C (S_0 = 3,818 MPa), then the gravel Type D (S_0 = 3,326 MPa), the SMA-D (S_0 = 2,981 MPa), and finally the gravel Type C (S_0= 2,381 MPa). This information could be useful for pavement design purposes as TxDOT is embracing the new AASHTO Mechanistic-Empirical Pavement Design Guide (MEPDG).

ii) ***Strain level.*** Fatigue life of HMA samples tested in BFT decrease with increased tensile strain. During this study, this relationship was verified and quantified. The relationship between log-number of cycles to failure and tensile strain level is assumed to be linear for the current study. Limestone Type D has the higher sensitivity to strain level (-0.76% slope), followed by gravel Type C (-0.63% slope), limestone Type C (-0.62%), gravel Type D (-0.55%), and, last of all, SMA-D with the slower rate of decay (-0.48% slope). The fatigue life results could also be used to enhance and calibrate the MEPDG for local conditions.

iii) ***Asphalt content.*** Fatigue life increases with increased asphalt content. This important aspect was quantified for five local mixtures. The most sensitive mixture to increase in asphalt content was limestone Type C. The least sensitive mixtures to changes in asphalt content are the gravel mixtures, specifically gravel Type D. This is consistent with previous studies that show the laboratory performance of gravel mixtures is less sensitive to volumetric changes.

iv) ***Gradation.*** Following the same trend observed with asphalt content, limestone Type C with fine gradation is the most sensitive mix to changes in gradation level. The less sensitive mix is gravel Type D with coarse gradation. It should be emphasized that in most cases the gradation was not found to be a statistically significant factor.

v) ***Density.*** The effect of density in performance (air voids content) was not found to be statistically significant. However, some average trends were determined. Based on these average trends, the Type C mixture (limestone and gravel) were the most sensitive to changes in air-void content, and SMA-D was the less sensitive. Mix density is an important factor that affects mixture performance. However, current test specifications allow testing to be done at 7 ± 1 % air void and it is within this range that the experiment conducted as part of this research did not find any statistical significant differences.

vi) ***Deformation.*** As expected, permanent deformation under the HWTD increased with increased asphalt content. For the tested mixtures, the critical combination was high asphalt content with the fine gradation. The limestone SMA-D was the most sensitive of the mixtures to changes in asphalt content; however, it should be noted that the optimum binder content for this mixture was the highest. The limestone Type D was the least sensitive.

6.3 Conclusions and Recommendation for Adjusting Pay Factors

The impact of the findings of this research in the development of Payment Adjustment Factors (PAF) is not straightforward. The researcher team supports the concept that PAFs should be implemented to commend high quality and a consistent product but at the same time to promote the design and construction of superior performing mixtures. It is within this context that the research team believes that ***performance-based pay adjustment factors (PB-PAF) should be developed and implemented.*** In this way, the additional cost resulting from the PAF could be offset by the additional benefits in terms of extended performance and the savings resulting from minimizing disruption and user's costs, such as the delay costs incurred during maintenance and rehabilitation activities.

The development of performance-based pay adjustment factors should be initially conducted based on the results of laboratory performance tests and later validated with short- and long-term field performance observations. The mix variables that could be considered for developing PB-PAFs should include density, gradation, and binder content. However, other performance-related variables that could be accurately and consistently determined in the laboratory and the field should be considered. For instance, the use of wave propagation velocity should be considered. This velocity can be easily be determined by non-destructive means and it is a function of performance-related properties such as density, modulus of elasticity (E), and shear modulus (G). For quality control and quality assurance practices, it is not necessary to convert velocity into modulus, which is one of the most skeptic aspects of this technology.

In the interim, and because the findings of this research indicate the important effect of binder content on performance, it is recommended that TxDOT implements some scale for pay adjustment factors to incentivize contractors that can deliver a consistent and homogeneous product and penalize those that do not. Table 6.2 provides interim recommendations that should be evaluated in conjunction with project-specific conditions and other practical mix production constraints. These adjustment factors should be applied only to the asphalt cost that should be included as a separate pay item.

Table 6.2: Interim recommendations for PAFs based on binder content

Deviation from JMF Target	Pay Adjustment Factor (PAF)
≤ 0.05	1.05
(0.05, 0.10]	1.02
(0.10, 0.15]	0.98
(0.15, 0.20]	0.92
(0.20, 0.25]	0.84
(0.25, 0.30]	0.75
> 0.30	0.00

The implementation of effective PB-PAFs cannot be general; that is, one set of pay adjustment factors cannot apply to all project conditions. This aspect poses a significant challenge and should be subjected to further research. For instance, in a region where environmental conditions are such that permanent deformation is the primary distress concern, PAF should rely heavily on the expected performance determined by a rolling wheel-type of test such as the Hamburg Wheel Tracking Device. In the case of Texas, this condition applies to South Texas. In the Panhandle region, on the other hand, cracking could be the major concern and, therefore, payment adjustment factors should be primarily based on tests such as the Beam Fatigue Test or an alternative simpler test. In general, the conditions are somewhere in the middle and a compromise decision is necessary between cracking and deformation concerns.

Unfortunately, there are some variables that affect cracking and fatigue resistance in opposite directions such as binder content: as asphalt content increases, the resistance of the mixture to fatigue increases but the resistance to permanent deformation increases. Other design variables such as density may be less problematic because for most mixtures, there is a range of optimum air voids content where both fatigue and rutting performance are near optimum while outside this range both performance types are significantly reduced.

6.4 Conclusions and Recommendation for Adjusting Operational Tolerances

During this research project three limestone mixtures (Type C, Type D, and SMA-D) and two gravel mixtures (Type C and Type D) were subjected to laboratory performance testing by means of Hamburg Wheel Tracking Device, Beam Fatigue Test, and Overlay Tester. Under these conditions, changing density and gradation within current operational tolerances was found not to have a significant effect on laboratory performance. This does not mean that density and gradation do not have an effect on performance; as a matter of fact, they do have an effect. However, based on the laboratory evaluation of laboratory prepared mixtures, as long as these mixture properties are varied within current tolerances, these effects are not statistically significant. As a result, ***this research study indicates that current operational tolerances for gradation and density appear to be adequate.***

In terms of binder content, however, the findings are different. It was found that as the binder content of a given mixture varies one percent around its optimal value, the permanent deformation and fatigue performance of the mixture are significantly affected. As expected, as the binder content increased, the maximum deformation after 20,000 passes of the HWTD significantly increased and the laboratory fatigue life with the BFT significantly increased. Most importantly, it was found that four of the five mixtures tested will be able to accommodate more binder and consequently increase their resistance to fatigue cracking without significantly compromising their rutting performance. This study did not investigate the potential effect of binder content increase on flushing.

As a result of these findings, this research recommends that the operational tolerances for the asphalt content given in the 2004 "Standard Specifications for Construction and Maintenance of Highways, Streets and Bridges" should be reviewed. This recommendation affects Table 7 Item 340, Table 9 Item 341, and Table 9 Item 346.

This research recommends that the allowable difference from current JMF target should be reduced from ± 0.3 to ± 0.2. Furthermore, it is also recommended that Note 3 should in Table 9 be reviewed and modified as follows: "Tolerance between JMF1 and JMF2 may <u>not</u> exceed ± 0.3."

These recommendations are based solely on the laboratory test results performed as part of this research project; therefore, practical considerations should be also weighed at the time of setting the final specifications. It is expected that the implementation of these recommendations will results on a more consistent and homogeneous end product.

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Appendix A. Aggregate Gradation

Table A1. Limestone: coarse aggregate gradation

Sieve Size		Cumulative Percentage Passing by Weight			
US-Standard	**Metric-Standard**	**Delta C-Rock**	**Centex D-Rock**	**Centex F-Rock**	**Delta Grade 4**
1”	25.0mm	100.0%	100.0%	100.0%	100.0%
3/4"	19.0mm	98.6%	100.0%	100.0%	100.0%
1/2"	12.5mm	25.0%	96.5%	100.0%	100.0%
3/8”	9.5mm	1.6%	71.7%	100.0%	77.0%
#4	4.75mm	0.9%	6.5%	78.2%	8.5%
#8	2.36mm	0.9%	1.5%	19.9%	0.6%
#16	1.18mm	0.8%	0.8%	6.1%	--
#30	0.60mm	0.8%	0.6%	2.4%	0.2%
#50	0.30mm	0.8%	0.5%	1.5%	0.2%
#200	0.075mm	0.6%	0.3%	0.3%	0.2%

Table A2. Limestone: fine aggregate gradation

Sieve Size		Cumulative Percentage Passing by Weight		
US-Standard	**Metric-Standard**	**Centex Manufactured Sand**	**Travis Field Sand**	**Screenings**
3/8”	9.50mm	100.0%	100.0%	100.0%
#4	4.75mm	99.9%	99.3%	98.2%
#8	2.36mm	94.0%	98.7%	86.0%
#16	1.18mm	67.5%	97.1%	71.9%
#30	0.60mm	39.4%	92.9%	59.4%
#50	0.30mm	19.9%	75.9%	49.3%
#200	0.075mm	3.8%	28.9%	28.8%

Table A3. Gravel: coarse aggregate gradation

Sieve Size		Cumulative Percentage Passing by Weight			
US-Standard	**Metric-Standard**	**Price-Galo 5/8” - 1/2”**	**Price-Galo 1/2” – 3/8”**	**Price-Galo 3/8” - 1/4”**	**Price-Galo 1/4” - 1/8”**
3/4"	19.0mm	100.0%	100.0%	100.0%	100.0%
1/2"	12.5mm	84.1%	97.5%	99.4%	100.0%
3/8”	9.5mm	12.2%	57.7%	94.7%	99.8%
#4	4.75mm	0.6%	0.9%	7.0%	67.0%
#8	2.36mm	0.4%	0.3%	0.2%	10.6%
#30	0.60mm	0.4%	0.2%	0.2%	1.8%
#50	0.30mm	0.4%	0.2%	0.2%	1.3%
#200	0.075mm	0.3%	0.1%	0.1%	0.7%

Table A4. Gravel: fine aggregate gradation

Sieve Size		Cumulative Percentage Passing by Weight		
US-Standard	Metric-Standard	Centex Manufactured Sand	Travis Field Sand	Screenings
3/8”	9.50mm	100.0%	100.0%	99.8%
#4	4.75mm	99.9%	99.3%	98.0%
#8	2.36mm	94.0%	98.7%	85.2%
#30	0.60mm	39.4%	92.9%	40.1%
#50	0.30mm	19.9%	75.9%	28.9%
#200	0.075mm	3.8%	28.9%	15.7%

Table A5. Fly ash gradation

Sieve Size		Cumulative Percentage Passing by Weight
US-Standard	Metric-Standard	Fly Ash
#8	2.36mm	100.0%
#200	0.075mm	85.0%

Table A6. Combined gradation for limestone Type C mixture

Sieve Size		Cumulative Percentage Passing by Weight				
US-Standard	Metric-Standard	Target Gradation	Fine Gradation	Coarse Gradation	Specification Lower Limit	Specification Upper Limit
3/4"	19.0mm	99.7	99.7	99.7	95.0	100.0
3/8”	9.5mm	70.3	74.2	68.9	70.0	85.0
#4	4.75mm	53.3	57.3	49.7	43.0	63.0
#8	2.36mm	37.4	41.1	36.5	32.0	44.0
#30	0.60mm	18.5	20.1	18.4	14.0	28.0
#50	0.30mm	11.9	12.6	11.8	7.0	21.0
#200	0.075mm	3.6	3.7	3.6	2.0	7.0

Table A7. Combined gradation for limestone Type D mixture

Sieve Size		Cumulative Percentage Passing by Weight				
US-Standard	Metric-Standard	Target Gradation	Fine Gradation	Coarse Gradation	Specification Lower Limit	Specification Upper Limit
1/2"	12.5mm	97.9	100.0	92.9	98.0	100.0
3/8”	9.5mm	79.3	84.3	74.3	85.0	100.0
#4	4.75mm	48.5	53.5	43.5	50.0	70.0
#8	2.36mm	36.9	39.9	33.9	35.0	46.0
#30	0.60mm	19.4	22.4	16.4	15.0	29.0
#50	0.30mm	12.0	15.0	9.0	7.0	20.0
#200	0.075mm	2.7	4.7	2.0	2.0	7.0

Table A8. Combined gradation for limestone SMA-D mixture

Sieve Size		Cumulative Percentage Passing by Weight				
US-Standard	Metric-Standard	Target Gradation	Fine Gradation	Coarse Gradation	Specification Lower Limit	Specification Upper Limit
3/4"	19.0mm	100.0	100.0	100.0	100.0	100.0
1/2"	12.5mm	88.0	93.0	83.0	85.0	99.0
3/8”	9.5mm	53.0	58.0	48.0	50.0	75.0
#4	4.75mm	25.0	30.0	20.0	20.0	32.0
#8	2.36mm	18.0	23.0	13.0	16.0	28.0
#16	1.18mm	15.6	18.6	12.6	8.0	28.0
#30	0.60mm	13.7	16.7	10.7	8.0	28.0
#50	0.30mm	12.1	15.1	9.1	8.0	28.0
#200	0.075mm	8.2	10.2	8.0	8.0	12.0

Table A9. Combined gradation for gravel Type C mixture

Sieve Size		Cumulative Percentage Passing by Weight				
US-Standard	Metric-Standard	Target Gradation	Fine Gradation	Coarse Gradation	Specification Lower Limit	Specification Upper Limit
1"	25.0mm	100.0	100.0	100.0	100.0	100.0
3/4"	19.0mm	100.0	100.0	100.0	95.0	100.0
3/8"	9.5mm	78.7	83.7	73.7	70.0	85.0
#4	4.75mm	50.8	55.8	45.8	43.0	63.0
#8	2.36mm	37.1	42.1	32.1	32.0	44.0
#30	0.60mm	16.3	19.3	13.3	14.0	28.0
#50	0.30mm	10.5	13.5	7.5	7.0	21.0
#200	0.075mm	4.7	6.7	2.7	2.0	7.0

Table A10. Combined gradation for gravel Type D mixture

Sieve Size		Cumulative Percentage Passing by Weight				
US-Standard	Metric-Standard	Target Gradation	Fine Gradation	Coarse Gradation	Specification Lower Limit	Specification Upper Limit
3/4"	19.0mm	100.0	100.0	100.0	100.0	100.0
1/2"	12.5mm	99.4	100.0	94.4	98.0	100.0
3/8"	9.5mm	90.4	95.4	85.4	85.0	100.0
#4	4.75mm	54.5	59.5	49.5	50.0	70.0
#8	2.36mm	37.6	42.6	32.6	35.0	46.0
#30	0.60mm	16.4	19.4	13.4	15.0	29.0
#50	0.30mm	10.5	13.5	7.5	7.0	20.0
#200	0.075mm	4.7	6.7	2.7	2.0	7.0

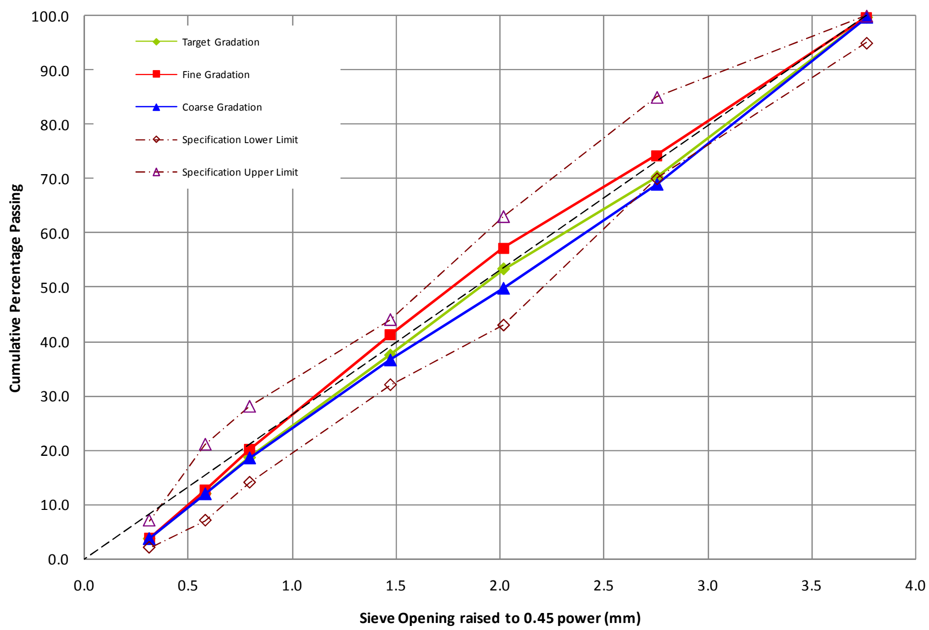

Figure A1. 0.45 Power Gradation Curves. Limestone Type C mixture

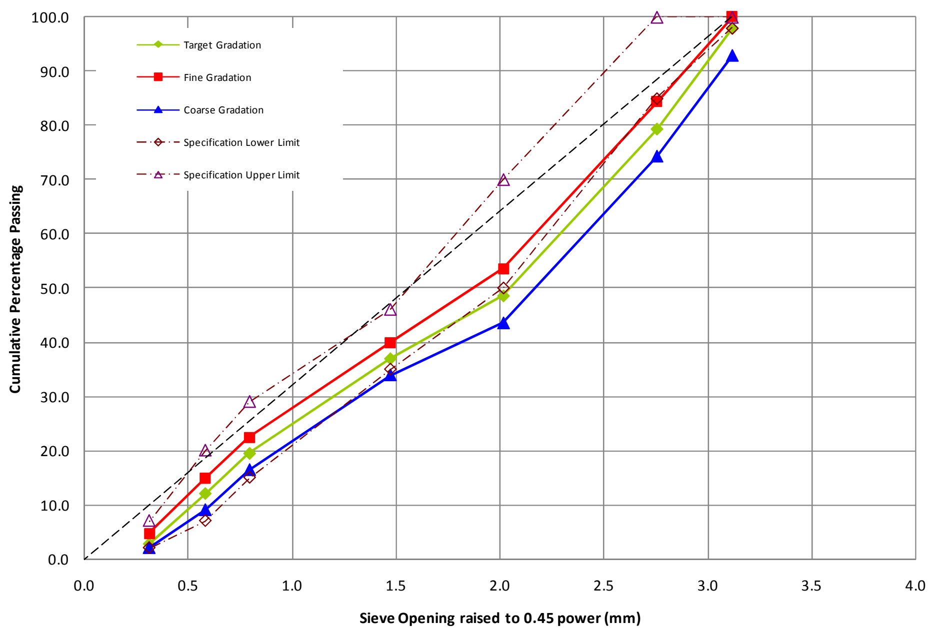

Figure A2. 0.45 Power Gradation Curves. Limestone Type D mixture

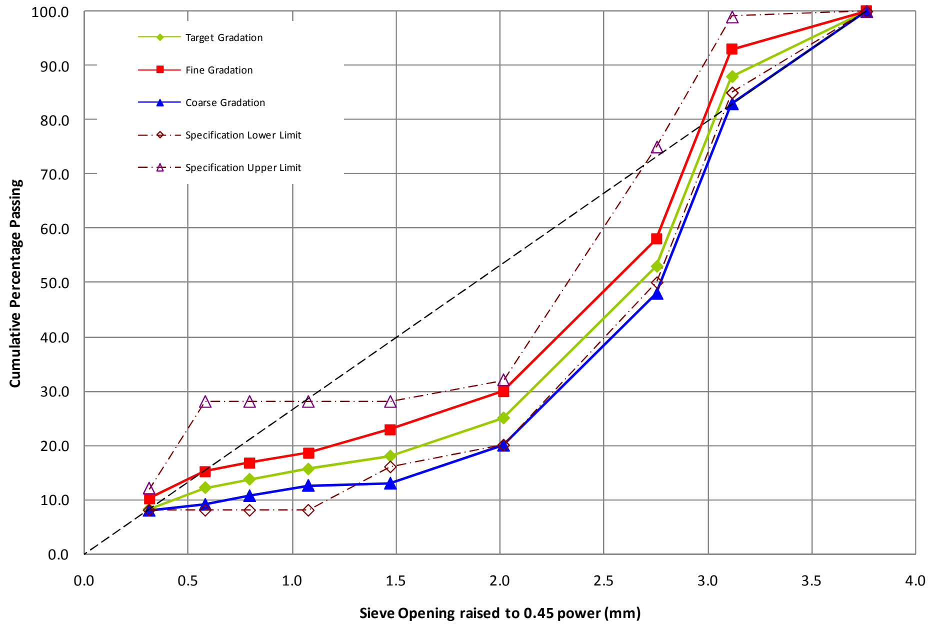

Figure A3. 0.45 Power Gradation Curves. Limestone SMA-D mixture

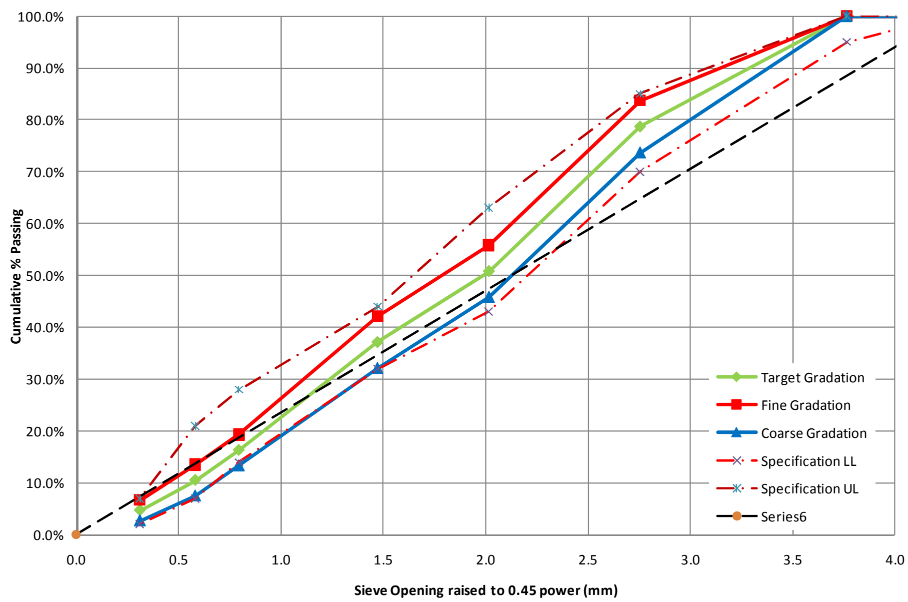

Figure A4. 0.45 Power Gradation Curves. Gravel Type C mixture

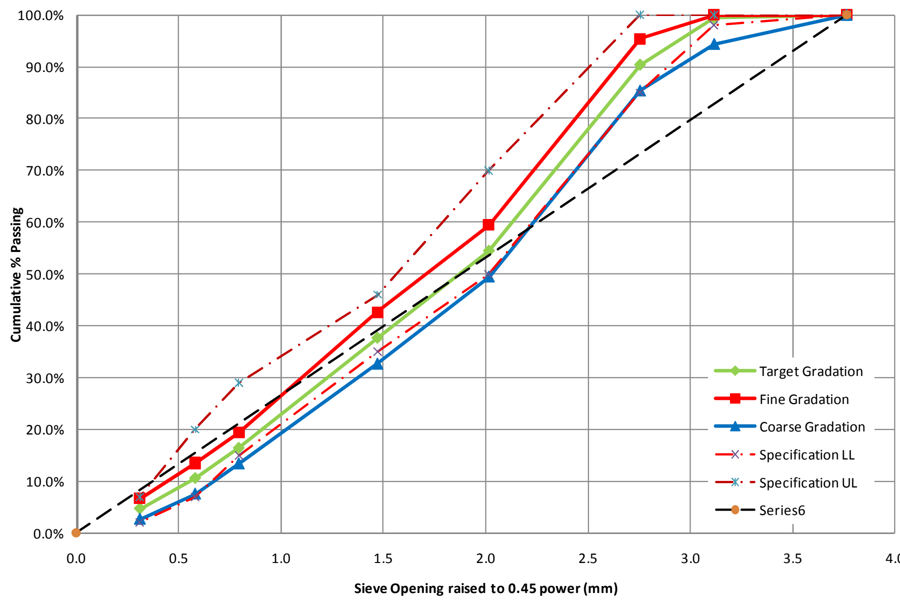

Figure A5. 0.45 Power Gradation Curves. Gravel Type D mixture

Appendix B. BFT Test Results

Table B1. Beam Fatigue Test Results for Limestone Type C

AC (%)	Gradation	Strain	Stiffness	Failure
4.0%	Target	600	3,628	5,140
4.0%	Target	600	3,527	2,440
4.0%	Target	400	3,123	9,090
4.0%	Target	400	4,567	40,220
4.3%	Target	600	2,138	27,260
4.3%	Target	600	2,415	16,010
4.3%	Target	400	2,583	1,847,840
4.3%	Target	400	3,487	327,340
4.6%	Target	500	2,822	9,360
4.6%	Target	500	2,559	2,810
4.6%	Target	300	4,576	8,799,220
4.6%	Target	300	4,594	5,551,930
5.0%	Target	600	3,988	186,200
5.0%	Target	600	4,117	111,770
5.0%	Target	400	4,294	250,220
5.0%	Target	400	4,189	591,100
4.0%	Fine	500	2,956	24,880
4.0%	Fine	500	3,609	9,140
4.0%	Fine	350	3,673	104,380
4.0%	Fine	350	4,445	48,900
4.3%	Fine	500	3,220	52,470
4.3%	Fine	500	3,616	26,260
4.3%	Fine	300	2,791	26,400
4.3%	Fine	300	4,095	6,506,280
4.6%	Fine	600	4,655	8,590
4.6%	Fine	500	5,272	68,210
4.6%	Fine	500	3,890	49,490
4.6%	Fine	400	3,700	460,050
5.0%	Fine	600	3,765	90,960
5.0%	Fine	600	4,019	43,760
5.0%	Fine	400	4,096	2,100,010
5.0%	Fine	400	4,550	168,410
4.0%	Coarse	500	4,772	38,080
4.0%	Coarse	500	4,305	66,610
4.0%	Coarse	350	4,213	894,760
4.0%	Coarse	350	4,350	206,960
4.3%	Coarse	500	4,152	194,940
4.3%	Coarse	500	3,941	81,150
4.3%	Coarse	350	4,117	973,090
4.3%	Coarse	350	4,033	2,550,820
4.6%	Coarse	500	4,003	46,940
4.6%	Coarse	500	3,420	59,710
4.6%	Coarse	350	4,344	494,450
4.6%	Coarse	350	3,865	1,113,080
5.0%	Coarse	500	3,276	83,850

AC (%)	Gradation	Strain	Stiffness	Failure
5.0%	Coarse	500	3,519	228,130
5.0%	Coarse	350	4,150	5,767,100
5.0%	Coarse	400	3,855	2,143,220

Table B2. Beam Fatigue Test Results for Limestone Type D

AC (%)	Gradation	Strain	Stiffness	Failure
4.6%	Target	600	3,558	41,380
4.6%	Target	600	3,423	54,310
4.6%	Target	500	3,795	295,390
4.6%	Target	500	3,855	87,740
5.0%	Target	600	3,559	61,480
5.0%	Target	600	3,750	27,650
5.0%	Target	500	3,186	821,340
5.0%	Target	500	3,327	622,260
5.3%	Target	350	5,084	3,503,030
5.3%	Target	400	4,567	7,624,690
5.3%	Target	600	4,139	96,360
5.3%	Target	600	4,555	112,950
5.6%	Target	600	4,468	68,190
5.6%	Target	600	4,278	272,050
5.6%	Target	500	4,381	315,310
5.6%	Target	500	4,542	583,290
5.0%	Fine	500	5,856	96,540
5.0%	Fine	500	5,597	102,250
5.0%	Fine	600	4,986	43,550
5.0%	Fine	600	5,143	22,720
5.3%	Fine	600	5,130	22,300
5.3%	Fine	600	4,655	54,390
5.3%	Fine	500	4,735	36,850
5.3%	Fine	500	5,079	80,760
5.6%	Fine	600	5,078	55,280
5.6%	Fine	600	4,913	41,870
5.6%	Fine	500	5,067	35,940
5.6%	Fine	500	5,431	535,990
6.0%	Fine	500	4,711	805,430
6.0%	Fine	500	5,135	549,520
6.0%	Fine	600	4,062	121,320
6.0%	Fine	600	4,261	52,510
4.6%	Coarse	600	3,845	24,630
4.6%	Coarse	600	3,866	61,720
4.6%	Coarse	500	4,061	94,710
4.6%	Coarse	500	4,224	377,120
5.0%	Coarse	600	3,982	38,980
5.0%	Coarse	600	4,237	42,970
5.0%	Coarse	500	3,865	628,050
5.0%	Coarse	500	4,147	181,360
5.3%	Coarse	600	4,494	60,770
5.3%	Coarse	600	4,964	59,530
5.3%	Coarse	500	4,430	123,800
5.3%	Coarse	500	3,376	2,550,740
5.6%	Coarse	600	3,960	337,190
5.6%	Coarse	600	3,818	318,560
5.6%	Coarse	600	3,689	643,830
5.6%	Coarse	600	4,374	49,180

Table B3. Beam Fatigue Test Results for Limestone SMA-D

AC (%)	Gradation	Strain	Stiffness	Failure
5.7%	Target	500	3,412	146,710
5.7%	Target	500	3,171	148,660
5.7%	Target	700	3,205	15,960
5.7%	Target	700	3,463	15,020
6.2%	Target	600	3,021	69,300
6.2%	Target	600	3,301	65,830
6.2%	Target	500	3,482	220,480
6.2%	Target	700	2,985	35,780
6.5%	Target	500	2,964	178,680
6.5%	Target	500	2,888	542,460
6.5%	Target	700	2,272	33,320
6.5%	Target	700	2,598	34,960
5.7%	Fine	500	3,790	40,490
5.7%	Fine	500	3,596	72,150
5.7%	Fine	700	3,213	9,080
5.7%	Fine	700	3,145	16,600
6.2%	Fine	500	3,829	111,640
6.2%	Fine	500	3,791	47,190
6.2%	Fine	700	3,601	12,340
6.2%	Fine	700	3,585	16,290
6.5%	Fine	500	3,704	147,250
6.5%	Fine	500	3,527	381,520
6.5%	Fine	700	3,184	17,830
6.5%	Fine	700	3,310	21,060
5.7%	Coarse	500	2,230	215,400
5.7%	Coarse	500	2,173	250,880
5.7%	Coarse	700	1,903	19,430
5.7%	Coarse	700	1,874	19,720
6.2%	Coarse	500	2,927	346,930
6.2%	Coarse	500	2,803	138,930
6.2%	Coarse	700	2,415	21,690
6.2%	Coarse	700	2,409	42,800
6.5%	Coarse	500	2,599	316,720
6.5%	Coarse	500	2,427	380,570
6.5%	Coarse	700	2,220	34,240
6.5%	Coarse	700	2,284	53,130

Table B4. Beam Fatigue Test Results for Gravel Type C

AC (%)	Gradation	Strain	Stiffness	Failure
5.0%	Target	500	2,086	100,340
5.0%	Target	500	1,848	160,000
5.0%	Target	350	2,383	4,201,990
5.0%	Target	350	2,609	1,201,450
5.3%	Target	550	2,341	71,920
5.3%	Target	550	2,346	126,160
5.3%	Target	400	2,346	1,969,010
5.3%	Target	400	2,474	870,960
5.6%	Target	550	2,211	440,740
5.6%	Target	550	2,309	258,510
5.6%	Target	400	2,820	1,421,210
5.6%	Target	400	2,648	1,218,510
6.0%	Target	650	2,456	123,430
6.0%	Target	650	2,323	86,910
6.0%	Target	500	2,683	475,980
6.0%	Target	500	2,695	703,830
5.0%	Fine	500	2,925	141,120
5.0%	Fine	500	2,882	92,140
5.0%	Fine	650	2,453	22,970
5.0%	Fine	650	2,416	52,700
5.3%	Fine	500	2,764	91,370
5.3%	Fine	500	2,487	118,040
5.3%	Fine	650	2,233	57,140
5.3%	Fine	650	2,199	84,200
5.6%	Fine	500	2,779	175,510
5.6%	Fine	500	2,575	231,090
5.6%	Fine	650	2,502	35,450
5.6%	Fine	650	2,476	28,250
6.0%	Fine	500	2,549	713,530
6.0%	Fine	500	2,628	323,440
6.0%	Fine	650	2,753	32,900
6.0%	Fine	650	2,743	84,010
5.0%	Coarse	500	2,729	293,990
5.0%	Coarse	500	2,383	1,013,710
5.0%	Coarse	650	2,096	26,090
5.0%	Coarse	650	2,071	42,130
5.3%	Coarse	500	2,444	283,160
5.3%	Coarse	500	2,378	335,900
5.3%	Coarse	650	1,981	52,410
5.3%	Coarse	650	1,981	50,860
5.6%	Coarse	500	2,204	672,120
5.6%	Coarse	500	2,192	716,780
5.6%	Coarse	650	1,831	115,620
5.6%	Coarse	650	1,833	84,520
6.0%	Coarse	500	2,140	1,010,260
6.0%	Coarse	500	2,149	499,680
6.0%	Coarse	650	1,875	227,240
6.0%	Coarse	650	2,036	96,950

Table B5. Beam Fatigue Test Results for Gravel Type D

AC (%)	Gradation	Strain	Stiffness	Failure
5.3%	Target	500	3,334	243,480
5.3%	Target	500	3,466	152,790
5.3%	Target	650	3,129	40,330
5.3%	Target	650	3,209	49,010
5.6%	Target	500	3,533	245,430
5.6%	Target	500	3,286	296,940
5.6%	Target	650	2,345	67,280
5.6%	Target	650	2,585	60,830
5.9%	Target	500	3,682	452,410
5.9%	Target	500	4,062	327,610
5.9%	Target	650	3,129	103,350
5.9%	Target	650	3,239	62,430
6.3%	Target	500	3,406	580,300
6.3%	Target	500	3,488	514,830
6.3%	Target	650	3,400	91,730
6.3%	Target	650	3,565	73,000
5.3%	Fine	500	3,485	209,910
5.3%	Fine	500	3,516	169,680
5.3%	Fine	650	3,212	23,110
5.3%	Fine	650	3,270	14,190
5.6%	Fine	500	3,047	338,470
5.6%	Fine	500	2,890	434,660
5.6%	Fine	650	3,284	36,450
5.6%	Fine	650	3,364	48,830
5.9%	Fine	500	5,044	180,910
5.9%	Fine	500	4,900	140,110
5.9%	Fine	650	4,032	54,580
5.9%	Fine	650	4,226	50,280
6.3%	Fine	500	4,267	543,270
6.3%	Fine	500	4,569	388,670
6.3%	Fine	650	4,299	70,250
6.3%	Fine	650	3,960	64,370
5.3%	Coarse	500	3,084	435,960
5.3%	Coarse	500	2,938	471,830
5.3%	Coarse	650	2,505	92,590
5.3%	Coarse	650	2,584	64,430
5.6%	Coarse	500	2,244	773,640
5.6%	Coarse	500	2,456	515,590
5.6%	Coarse	650	2,469	99,230
5.6%	Coarse	650	2,433	83,290
5.9%	Coarse	500	2,875	784,450
5.9%	Coarse	500	2,994	427,560
5.9%	Coarse	650	2,871	69,150
5.9%	Coarse	650	2,975	106,030
6.3%	Coarse	500	3,401	469,620
6.3%	Coarse	500	3,455	227,220
6.3%	Coarse	650	3,002	148,300
6.3%	Coarse	650	3,050	83,100

Appendix C. Hamburg Wheel-Tracking Device Test Results

Table C1. Hamburg Wheel Tracking Results for Limestone Type C

AC (%)	Gradation	No. of Passes		
		10,000	15,000	20,000
4.0%	Target	-2.14	-3.01	-4.28
4.3%	Target	-2.64	-4.01	-5.75
4.6%	Target	-3.05	-4.41	-5.63
5.0%	Target	-5.20	-7.61	-9.00
4.0%	Fine	-1.52	-1.80	-2.53
4.3%	Fine	-1.71	-2.12	-3.07
4.6%	Fine	-3.74	-6.04	-8.46
5.0%	Fine	-2.79	-3.43	-4.29
4.0%	Coarse	-2.09	-3.60	-4.61
4.3%	Coarse	-2.62	-3.40	-4.59
4.6%	Coarse	-2.62	-3.40	-4.60
5.0%	Coarse	-2.86	-3.66	-5.16

Table C2. Hamburg Wheel Tracking Results for Limestone Type D

AC (%)	Gradation	No. of Passes		
		10,000	15,000	20,000
4.6%	Target	-2.18	-2.94	-3.91
5.0%	Target	-2.23	-3.21	-4.20
5.3%	Target	-2.93	-4.24	-5.58
5.6%	Target	-4.15	-4.90	-6.00
5.0%	Fine	-4.03	-5.53	-8.38
5.3%	Fine	-5.92	-6.99	-8.95
5.6%	Fine	-6.58	-8.45	-10.65
6.0%	Fine	fail	fail	fail
4.6%	Coarse	-3.05	-3.63	-4.06
5.0%	Coarse	-3.21	-3.41	-3.88
5.3%	Coarse	-4.51	-5.72	-7.32
5.6%	Coarse	-4.06	-4.61	-5.27

Table C3. Hamburg Wheel Tracking Results for Limestone SMA-D

AC (%)	Gradation	No. of Passes		
		10,000	15,000	20,000
5.7%	Target	-6.25	-6.80	-7.56
6.2%	Target	-7.48	-8.36	-9.37
6.5%	Target	-8.51	-10.00	-11.27
5.7%	Fine	-8.30	-9.37	-10.27
6.2%	Fine	-8.14	-8.66	-9.34
6.5%	Fine	-11.39	-12.34	-13.54
5.7%	Coarse	-6.15	-6.67	-6.87
6.2%	Coarse	-8.77	-8.85	-9.77
6.5%	Coarse	-8.75	-9.43	-10.52

Table C4. Hamburg Wheel Tracking Results for Gravel Type C

AC (%)	Gradation	No. of Passes		
		10,000	15,000	20,000
5.0%	Target	-2.54	-2.91	-3.25
5.3%	Target	-3.47	-3.88	-4.19
5.6%	Target	-3.63	-3.79	-4.16
6.0%	Target	-5.39	-5.96	-6.59
5.0%	Fine	-2.95	-3.27	-3.58
5.3%	Fine	-2.95	-3.41	-3.51
5.6%	Fine	-4.54	-4.88	-5.13
6.0%	Fine	-6.29	-7.22	-8.00
5.0%	Coarse	-2.74	-2.74	-3.09
5.3%	Coarse	-3.58	-3.84	-4.16
5.6%	Coarse	-4.22	-4.85	-5.11
6.0%	Coarse	-7.46	-8.22	-8.57

Table C5. Hamburg Wheel Tracking Results for Gravel Type D

AC (%)	Gradation	No. of Passes		
		10,000	15,000	20,000
5.3%	Target	-4.26	-4.69	-5.12
5.6%	Target	-5.57	-6.16	-6.69
5.9%	Target	-5.07	-5.75	-6.42
6.3%	Target	-6.25	-6.65	-7.44
5.3%	Fine	-4.80	-5.09	-5.50
5.6%	Fine	-3.73	-4.10	-4.45
5.9%	Fine	-5.43	-5.94	-6.54
6.3%	Fine	-6.37	-6.92	-7.46
5.3%	Coarse	-3.67	-3.97	-4.20
5.6%	Coarse	-5.10	-5.69	-5.87
5.9%	Coarse	-4.16	-4.64	-5.26
6.3%	Coarse	-6.76	-7.33	-7.73

Appendix D. Overlay Tester Results

Table D1. Overlay Test Results for Limestone Type C

AC (%)	Gradation	Cycles to Failure		
		R1	R2	R3
4.0%	Target	4	5	3
4.3%	Target	10	3	9
4.6%	Target	9	66	7
5.0%	Target	102	141	46
4.0%	Fine	6	3	3
4.3%	Fine	2	26	11
4.6%	Fine	34	15	14
5.0%	Fine	83	32	155
4.0%	Coarse	34	3	4
4.3%	Coarse	5	6	18
4.6%	Coarse	3	6	14
5.0%	Coarse	4	4	4

Table D2. Overlay Test Results for Limestone Type D

AC (%)	Gradation	Cycles to Failure		
		R1	R2	R3
4.6%	Target	31	6	36
5.0%	Target	8	9	6
5.3%	Target	10	111	8
5.6%	Target	233	193	73
5.0%	Fine	5	258	7
5.3%	Fine	21	10	12
5.6%	Fine	168	42	253
6.0%	Fine	45	103	97
4.6%	Coarse	4	32	5
5.0%	Coarse	16	6	49
5.3%	Coarse	83	64	46
5.6%	Coarse	47	60	80

Table D3. Overlay Test Results for Limestone SMA-D

AC (%)	Gradation	Cycles to Failure		
		R1	R2	R3
5.7%	Target	58	13	9
6.2%	Target	9	5	6
6.5%	Target	64	89	45
5.7%	Fine	10	6	5
6.2%	Fine	28	28	30
6.5%	Fine	18	76	31
5.7%	Coarse	12	10	24
6.2%	Coarse	12	55	15
6.5%	Coarse	23	20	793

Table D4. Overlay Test Results for Gravel Type C

AC (%)	Gradation	Cycles to Failure		
		R1	R2	R3
5.0%	Target	26	252	18
5.3%	Target	28	51	13
5.6%	Target	93	53	33
6.0%	Target	165	88	116
5.0%	Fine	22	4	3
5.3%	Fine	5	5	2
5.6%	Fine	4	5	10
6.0%	Fine	7	7	17
5.0%	Coarse	5	6	9
5.3%	Coarse	223	30	28
5.6%	Coarse	143	22	10
6.0%	Coarse	215	273	108

Table D5. Overlay Test Results for Gravel Type D

AC (%)	Gradation	Cycles to Failure		
		R1	R2	R3
5.3%	Target	181	197	82
5.6%	Target	33	67	18
5.9%	Target	390	291	210
6.3%	Target	304	80	75
5.3%	Fine	129	61	67
5.6%	Fine	346	104	303
5.9%	Fine	206	76	174
6.3%	Fine	181	283	192
5.3%	Coarse	69	8	33
5.6%	Coarse	127	53	359
5.9%	Coarse	374	55	116
6.3%	Coarse	139	61	34